HYDROABRASION BY HIGH-SPEED SEDIMENT-LADEN FLOWS IN SEDIMENT BYPASS TUNNELS

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presented by
MICHELLE FRANZISKA MÜLLER-HAGMANN
MSc Civil Eng, ETH Zurich

born on 07.10.1985
citizen of Sevelen SG, Glarus Süd GL

accepted on the recommendation of
Prof. Dr. Robert Michael Boes
Dr. Ismail Albayrak
Prof. Dr. Eugen Brühwiler
Dr. Frank Jacobs
Dr. Christian Auel

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Reservoir dams impound flowing water to balance a fluctuating water supply and demand. All reservoirs situated on natural watercourses are subjected to sediment inflow. Water-carried sediments continuously settle in the quiescent environment of a reservoir resulting in accumulation, negatively affecting their functions. Sediment bypass tunnels (SBTs) are an effective measure against reservoir sedimentation by diverting sediment-laden water past the dam and allowing for re-establishing the natural sediment continuity. However, high-speed sediment-laden flows in SBTs can cause severe invert abrasion, putting SBT operation at risk and provoking high maintenance costs. Mitigation of hydroabrasion problems requires a better understanding of governing parameters, namely flow characteristics, particle and invert material properties, sediment transport and their interactions. Despite a large number of small-scale laboratory studies on hydroabrasion, upscaling of their results to prototype scale is questionable due to a lack of prototype data. Therefore, this study aims at filling this research gap by in-situ investigation of hydroabrasion in combination with laboratory tests.

Various invert materials, i.e. concretes, granite, steel and cast basalt, were installed in the Solis, Pfaffensprung and Runcahez SBTs in Switzerland. The abrasion patterns and rates, flow conditions, sediment transport rate and sediment properties in the rivers, reservoirs and SBTs were determined and their interrelations were analyzed with an emphasis on hydroabrasion, bypass efficiency and operating regime of the SBTs and respective reservoirs.

The results of the abrasion measurements showed that hydroabrasion is a self-intensifying process, triggered by discontinuities and constructional weaknesses resulting in characteristic material-dependent abrasion patterns. In addition to this, the abrasion pattern was found to be affected by 3D flow structures induced by either tunnel bends or low tunnel width to flow depth ratios. The resulting ratio of maximum to spatially averaged abrasion depths ranged from 1.6 to 2.2, which is of prime importance regarding the dimensioning of the invert material thickness.

Invert material selection and dimensioning requires cost-effectiveness analysis, which is a function of abrasion rate, material cost, abrasion conditions and target life time. For the first, three existing abrasion prediction models were evaluated and calibrated based on the present field data. The resulting abrasion coefficients generally agreed with literature data. However, application of material-specific abrasion coefficients was found to significantly enhance the
model prediction accuracy and is therefore recommended. Based on that, present cost-effectiveness analysis revealed that hard rock and (ultra-)high-strength concretes are more suitable for long-term application under severe abrasion conditions, whereas less expensive materials are more economical for moderate to low abrasion conditions or short-term application, despite the lower abrasion resistance.

The present study revealed that the bypass efficiency of a SBT strongly depends on the location of the SBT intake and the operational regime of both the reservoir and the SBT. To achieve an optimum bypass efficiency, the SBT and reservoir operations need to be coordinated and optimized, which requires a continuous and real-time monitoring of the hydraulic and sediment transport conditions in the river, reservoir and SBT.

Overall, this study provides new insights into the flow and sediment transport characteristics in SBTs and reservoirs and advances the understanding of hydroabrasion processes in high-speed sediment-laden open channel flows. The main results are the applicable recommendations with a focus on hydroabrasion and bypass efficiency, contributing to a sustainable and efficient design and operation of SBTs.
Talsperren stauen Fließgewässer auf zum Ausgleich zwischen Wasserbedarf und Wasserverfügbarkeit. Stauseen an natürlichen Fließgewässern unterliegen einem stetem Sedimenteintrag. Die im Fluss mitgeführten Sedimente lagern sich unter den beruhigten Fließbedingungen im Stausee ab und frühen so zu einer zunehmenden Verlandung, was vielfältige Probleme verursacht. Sedimentumleitstollen (Englisch Sediment Bypass Tunnels, SBTs) stellen eine effizierte Massnahme dar, um der Stauraumverlandung entgegenzuwirken. Sie leiten sedimentreiche Abflüsse um die Talsperre herum ins Unterwasser ab und stellen so die natürliche Sedimentdurchgängigkeit wieder her. Die hohen Fließgeschwindigkeiten in Kombination mit den hohen Sedimenttransportraten können allerdings schwerwiegende Abrasionsschäden an der Stollensohle verursachen, die teure Unterhaltsarbeiten erfordern und die Betriebssicherheit gefährden. Um der Hydroabrasionsproblematik entgegen zu wirken, sind ein vertieftes Verständnis massgebender Einflussfaktoren, namentlich der Strömungsverhältnisse, der Eigenschaften des Auskleidungsmaterials und des Sediments, des Sedimenttransports sowie deren Wechselwirkungen unerlässlich. Trotz zahlreicher Laborstudien zur Hydroabrasion blieb die Skalierbarkeit der gewonnen Erkenntnisse auf Prototypmassstab aufgrund fehlender Prototypdaten ungeklärt. Diese Forschungslücke soll mit der vorliegenden Arbeit mittels Feld- und Laborversuchen reduziert werden.


Die Abrasionsuntersuchungen zeigten, dass Hydroabrasion ein selbstverstärkender Prozess ist, der durch Diskontinuitäten und Inhomogenitäten ausgelöst wird und so zu materialspezifischen Abrasionscharakteristiken führt. Darüber hinaus wird das Abrasionsmuster auch durch dreidimensionale Strömungsstrukturen beeinflusst, die durch Tunnelkrümmungen und enge Abflussquerschnitte induziert werden. Das Verhältnis zwischen maximalen und gemittelten Abrasionstiefen ist von besonderer Bedeutung für die Dimensionierung der
Materialschichtdicke und betrug zwischen 1,6 bis 2,2 unabhängig von Auskleidungsmaterial, SBT-Geometrie oder Betriebszustand.


Schließlich liefert diese Studie neue Einblicke in die Strömungs- und Sedimenttransportprozesse in SBTs und Stauseen und fördert das Verständnis von Hydroabrasionsprozessen in sedimentführenden schiessenden Freispiegelabflüssen. Das Hauptide des dieser Studie sind die Konstruktions- und Bemessungsempfehlungen für SBTs mit Schwerpunkt auf Hydroabrasion und Umleiteffizienz, die zu einer effizienten und nachhaltigen Gestaltung von SBTs sowie deren Betrieb beitragen.
1 Introduction

1.1 Motivation

Thousands of dams have been built worldwide within the last century in order to cope with the variability of river runoff. The purposes of reservoirs are multifold. They provide storage capacity for energy production, flood control, water supply and irrigation and serve for navigation, recreation and fishing. However, the sustainability of many reservoirs is threatened by sedimentation (UNESCO 2011). Since the sedimentation volumes both in Switzerland and worldwide exceed the increase of reservoir capacity, the net storage capacity is expected to decrease in the near future (Schleiss et al. 2010, Boes and Hagmann 2015). Under the strong impact of climate change, the sedimentation rates as well as runoff variabilities are also expected to rise in the future. Therefore, long-term sediment management strategies are required to maintain reservoir safety and sustainability.

Sediment Bypass Tunnels (SBTs) are an efficient and holistic measure against reservoir sedimentation. They divert sediment around the reservoir, re-establish natural sediment connectivity between upper and lower river reaches and hence contribute to a sustainable, and ecofriendly reservoir sediment management. To ensure sufficient transport capacity and to avoid transition to pressurized flow, most SBTs are operated at supercritical flow conditions. However, high-speed sediment-laden flows in SBTs can cause severe invert abrasion, putting SBT operation at risk and provoking high maintenance costs.

An impressive example is the Palagnedra SBT in Ticino, Switzerland. After a flood event with an over 100-year return period in August 1978, more than $2\times10^6$ m$^3$ of sediments were flushed through the SBT and caused a several meter deep incision channel along the tunnel invert shown in Figure 1.1a. In the winters from 2011 to 2013 expensive rehabilitation works were performed and a new abrasion-resistant lining was implemented, which is shown in Figure 1.1b.

On average, annual maintenance cost of SBTs may amount to 1% of their investment costs (Auel 2014). Optimized hydraulic design, operation regime and invert material selection can substantially reduce the hydroabrasive wear of inverts and hence the maintenance and refurbishment costs. This results in an increase of the overall energy production efficiency at hydropower reservoirs and a general enhancement of the cost-effectiveness of those facilities. Although hydroabrasion has been investigated in the past, most studies were based on
laboratory experiments. However, the transferability of those results to field applications is difficult, due to potential scale and model effects. Existing field investigations mainly focus on hydroabrasion as a landscape-shaping process, acting on larger temporal and spatial scales. There are only few prototype studies analyzing hydroabrasion, covering a narrow spectrum of operating conditions and invert materials. Furthermore, no guideline for the design and operation of SBTs is available (Harada et al. 1997, Sumi et al. 2004b, Auel and Boes 2011). Therefore, the Laboratory of Hydraulics, Hydrology and Glaciology (VAW) of ETH Zurich initiated the presented PhD project.

The main goal of the project is to fill the above described knowledge gaps by \textit{in-situ} investigation of hydroabrasion in combination with laboratory tests. To this end the behavior and abrasion resistance of various invert materials implemented in three Swiss SBTs were investigated. A former PhD project dealt with the flow characteristics, sediment transport and invert abrasion processes and their interrelations in a Froude-scaled SBT model at VAW and enhanced the state-of-the-art-abrasion prediction model by Sklar and Dietrich (2004) (Auel 2014). The findings of both the former and the present project - in combination with recent results reported in the literature - contribute to a sustainable design and operation of SBT as well as other hydraulic structures facing hydroabrasion, and potentially to minimize the loss of reservoir volume due to sedimentation (Auel et al. 2015b).
1.2 Objectives

The present study aims at mitigating hydroabrasion problems at SBTs and other hydraulic structures exposed to hydroabrasion. The following specific objectives were established to this end:

- Determination of abrasion resistance of various invert materials, such as high performance concretes, steel, cast basalt and granite, used in existing SBTs by means of laboratory and prototype investigations;
- Quantification of suspended sediment and bedload transport rates and hydraulic conditions in SBTs;
- Determination of the correlation between hydraulic conditions, sediment transport, material properties and hydroabrasion;
- Examination of the transferability of laboratory abrasion results to prototype scale;
- Calibration of the abrasion prediction model of Auel et al. (2017b), Sklar and Dietrich (2004) and Ishibashi (1983) for concrete and granite;
- Determination of the cost-effectiveness of the studied invert materials;
- Recommendations for the design and operation of SBTs with a focus on the bypass efficiency and the invert abrasion.

To achieve these objectives, *in-situ* experiments were conducted at the Solis, Pfaffensprung and Runcahez SBTs in Switzerland, and specimens of the invert materials from Solis and Pfaffensprung SBTs were tested in the laboratory of the Institute of Construction Materials of the Technical University Dresden, Germany. The data basis was extended by surveys of other facilities mainly located in Switzerland and Japan.

1.3 Thesis outline

The presented study is organized as follows. Chapter 2 gives an overview of reservoir sedimentation and summarizes the state-of-the-art on SBTs. Chapter 3 provides a theoretical background including hydraulics, sediment transport, hydroabrasion as well as hydroabrasion prediction models. The test setups and methodology are documented in Chapter 4, and the results from the different sites and test series are presented in the Chapters 5 to 9. Chapter 10 deals with bypass efficiency, abrasion modelling and material cost-effectiveness. Finally,
design and operation recommendations for engineering application are summarized in Chapter 11, while Chapter 12 includes the conclusions and an outlook on further research.

1.4 Research methodology

Hydroabrasion problems can be investigated by means of analytical approaches, numerical simulations, physical scale model tests in the laboratory and prototype experiments or a combination of them. Analytical and numerical methods require prototype measurements for validation purposes, and scale effect in laboratory experiments is still a topic of ongoing research. Therefore, the present study aims at investigating hydroabrasion by means of prototype as well as laboratory experiments. The following approach was adopted:

- Acquisition of abrasion data using
  - Prototype experiments conducted in three Swiss SBTs (requiring novel instrumentation and invert test fields)
  - Laboratory experiments conducted by the Technical University Dresden, Germany
- Acquisition of sediment transport data by means of
  - Indirect measurements of suspended sediment and bedload transport at Solis SBT using (I) turbidimeters calibrated based on bottle samples and (II) a Swiss Plate Geophone System (SPGS) calibrated in the laboratory and in-situ
  - Sediment transport estimates in the studied rivers and SBTs using various formulas described in literature and site-specific suspended sediment concentration - discharge rating curves
- Determination of the abrasion resistance of the studied invert materials based on the prototype and laboratory test results
- Validation / calibration of existing state-of-the-art abrasion models
- Economical assessment of the lift-cycle costs of the studied invert materials based on net present values accounting for investment cost, abrasion resistance, target life time abrasion conditions and resulting maintenance cost
- Evaluation of the bypass efficiency of the prototype SBTs
- Summarizing the results and providing design and operation recommendations for SBTs
2 Reservoir sustainability and sediment bypass tunnels

This chapter provides background information on reservoir sedimentation including physical processes, consequences and countermeasures. The focus of the latter is on SBTs, for which detailed information on existing SBT with regard to the bypass efficiency, the invert abrasion and the eco-morphological impacts are presented.

2.1 Reservoir sedimentation

Dams impound rivers to create a storage volume to regulate the fluctuations of water demand and the natural variability of runoff in rivers. Thus, river water is stored and released as is needed for water supply, irrigation, energy generation or flood protection aspects. Reservoirs are also used for fishing, navigation, leisure and recreation purposes. The need for reservoirs to fulfill these multiple functions increases with population growth and the progress of developing countries. Moreover, the demand for energy production from renewable resources such as hydropower has gained growing interest after the nuclear accidents in Chernobyl and Fukushima in 1986 and 2011, respectively.

It was common to design and operate reservoirs in the past based on the “dead storage” concept, i.e. storing sediment for a determined lifespan of 50 to 100 years followed by replacement (Vischer 1981, Kantoush and Sumi 2010). This concept leaves decommissioning of the reservoirs to future generations and does not incorporate the long-term costs of reservoir storage loss into decision-making (Kondolf 1997, Palmieri et al. 2001, Annandale 2014). Although reservoir sedimentation was not previously identified as a problem at most Alpine reservoirs, Vischer (1981) already pointed out that the future generations will have to deal with it. Since then, the sedimentation problems have increased and endangered the sustainable use of reservoirs. Therefore, the conventional concept of “dead storage” is no longer acceptable and a new concept based on active sediment management is required (Kondolf 1997). This topic receives globally increasing attention, which has been demonstrated by the large number of studies conducted in the last three decades (Wilcock et al. 1996, Kondolf 1997, Vischer et al. 1997, Boillat et al. 2000a, Boillat et al. 2000b, ÖWAV 2000, Sumi 2000, Palmieri et al. 2001, Schleiss and Oehy 2002, Hartmann 2004, Sumi 2005, Boes and Reindl 2006, DWA 2006, Knoblauch et al. 2007, Annandale 2009, Basson 2009, Hargrove et al. 2010, Kantoush and...
The knowledge of sediment transport, erosion and deposition is required to establish efficient and holistic sediment management techniques. The sediment transport processes can be idealized as a system consisting of three parts: (1) the zone of surface erosion and sediment production in the mountain runoff, (2) the zone of transport through the downstream river system and (3) the zone of ultimate deposition in the ocean, or intermediate stagnant water bodies like lakes and reservoirs (Kondolf 1997). More specific descriptions of these three parts of the sediment transport system are given as follows:

(1) Sediment production mainly takes place in steep headwaters that cause rapid erosion and is expressed as the specific erosion rate defined as the sediment mass per year and unit area. Erosion varies due to climate conditions, hydrology, meteorology, vegetation, land use, land cover, geologic materials and petrography (Walling and Webb 1996, Inoue 2009, Graf et al. 2010, Savi et al. 2016). The sediment yield typically ranges from 5 to 100 to/(yr·km²), but can reach values between 500 and 7000 to/(yr·km²) in mountainous areas with weak geological formations, active tectonic zones, volcanic activity, semi-arid climate zones and seasonally humid tropics (Walling and Webb 1983, Bogen 1989, 2008, Hinderer et al. 2013). In glacier-covered areas, sediment production is on average an order of magnitude higher than for glacier-free basins (Hallet et al. 1996, Hinderer et al. 2013) due to plucking and abrasion processes beneath the glaciers (Bogen 2008) and freeze-thaw cycles affecting bare ground (Lenzi and Marchi 2000, Izumiyama et al. 2010, Izumiyama et al. 2012). Therefore, downstream river reaches of mountainous areas like the Alps, Andes or Himalayas are often dominated by glacier sediments. Given the forecasted climate change and temperature increase, retreating glaciers will expose vast pro-glacial fields with unconsolidated sediment and steep erodible moraines leading to a further increase of sediment supply (Walling and Webb 1996, Syvitski 2003, Anselmetti et al. 2007, KOHS 2007, Bogen 2008, Schleiss et al. 2010, Wisser et al. 2013).

(2) Sediment transport is site-specific and strongly fluctuating. The bulk of the total annual load is frequently discharged within a single flood event (Haimann et al. 2014). Generally, the sediment transport capacity is larger than the sediment transport rate limited by the available mobile sediment supply (Rickenmann et al. 2008). The sediment transport is furthermore increasingly affected by the anthropogenic impacts. Gravel mining and dams disturb sediment continuity. The latter may massively reduce the sediment load in the catchment areas. For
instance, the original sediment inflow to Lake Geneva, Lake Brienz and in the Isar River in Munich were reduced to 33%, 43% and 11%, respectively (Loizeau and Dominik 2000, Anselmetti et al. 2007, Hinderer et al. 2013), and 24 of the 33 world’s largest deltas are retreating due to human impact (Syvitski et al. 2009). For example, the Mississippi delta lost 4800 km² due to the construction of multiple dams reducing the sediment supply by 48% (CPRA 2012). A further well documented example is the Shanghai wetland, the area of which decreased by 19% from 2001 to 2004 due to the construction of the Three Gorges Dam retaining approx. 60% of the sediments (Yang et al. 2006).

(3) Flow velocities decrease as river water flows into an impounded reach triggering the settling process. The coarsest particles settle first and build a delta with a shallow topset and a steep foreset separated by the pivot point shown in Figure 2.1. The particle size of the settling material continuously decreases along the flow path with the finest particles settling near the dam. This process causes the characteristic grain size distribution (GSD) from coarse to fine along the aggradation (Vischer 1981, Morris and Fan 1998, Schleiss et al. 2008).

A simple parameter to describe the reservoir sedimentation is the reservoir trap efficiency, defined as the mass ratio of the retained sediment to the total sediment inflow (Gill 1979). It depends on various parameters, i.e. particle settling velocity, residence time in the reservoir, capacity-inflow-ratio, reservoir topography, type of outlet structures, operation regimes and reservoir age (Brune 1953). Due to progressive reservoir sedimentation, the effective reservoir capacity and thus the trap efficiency decreases consecutively (Brune 1953, Vischer 1981). Different empirical approaches for the estimation of long-term trap efficiency (TE) and thus the reservoir service time exist (Morris and Fan 1998, Lewis et al. 2013). A widely applied and well proven empirical approach based on the capacity-inflow-ratio (CIR), defined as the ratio between capacity (CAP) and mean annual runoff (MAR), i.e. CIR = CAP/MAR, is Brune’s (1953) criterion, which is originally provided as a diagram. Gill (1979) introduced the
corresponding empirical solutions for coarse (Equation (2.1)), medium (Equation (2.2)) and fine sediments (Equation (2.3)) as follows:

\[ TE = \frac{CIR^2}{0.994701 \cdot CIR^2 + 0.006297 \cdot CIR + 3 \times 10^{-6}} \quad [-] \quad (2.1) \]

\[ TE = \frac{CIR}{0.012 + 1.02 \cdot CIR} \quad [-] \quad (2.2) \]

\[ TE = \frac{CIR^3}{1.02655 \cdot CIR^3 + 0.02621 \cdot CIR^2 - 1.33 \times 10^{-4} \cdot CIR + 1.0 \times 10^{-6}} \quad [-] \quad (2.3) \]

Brune (1953) did not specify the particle size but only distinguished between “coarse”, “medium” and “fine” sediment. Since his investigations focused mainly on suspended sediment monitoring data, this classification may presumably distinguish clay (Equation (2.3)), silt (Equation (2.2)) and sand (Equation (2.1)).

Another parameter to determine reservoir sedimentation is the sedimentation rate, defined as the ratio between accumulated volume and initial reservoir storage capacity. The mean global estimated annual sedimentation rates vary from 0.5 to 2% (Table 2.1) resulting in an infill time of 50 to 200 years. However, sedimentation is strongly site-dependent and sedimentation rates can reach values of around 20%, which are reported for some Chinese reservoirs (ICOLD 2009).

Table 2.1 Mean global reservoir sedimentation rates

<table>
<thead>
<tr>
<th>Author</th>
<th>Annual sedimentation rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>White (2001)</td>
<td>0.48%</td>
</tr>
<tr>
<td>Sumi and Kantoush (2011)</td>
<td>0.5%</td>
</tr>
<tr>
<td>Wisser et al. (2013)</td>
<td>0.55%</td>
</tr>
<tr>
<td>Schleiss et al. (2010)</td>
<td>0.8%</td>
</tr>
<tr>
<td>ICOLD (2009)</td>
<td>1%</td>
</tr>
<tr>
<td>Schleiss and Oehy (2002)</td>
<td>1-2%</td>
</tr>
</tbody>
</table>

2.2 Consequences of reservoir sedimentation

Reservoir sedimentation is a continuous process affecting mainly inactive storage volume at the early stage of the reservoir life time and thus tends to be ignored (Morris and Fan 1998). However, reservoir sedimentation causes numerous problems during the service life time of a
Reservoir sustainability and sediment bypass tunnels

The loss of water storage capacity is of prime importance for drinking water reservoir since fresh water is essential for (human) life. The overall rate of water consumption must be less than the rate of regeneration to ensure sufficient fresh water supply (Annandale 2014). Considering the replenishment rate of available fresh water resources (1400 years for ground water and 18 days for river water), river water exhibits a much higher potential for sustainable water supply than ground water (Shiklomanov and Rodda 2003). However, river runoff is characterized by daily, seasonal or even year-by-year discharge fluctuations, which have to be balanced. Therefore, storage capacity is required to compensate the effect of the hydrological variability (Annandale 2014).

The global reservoir storage capacity is estimated in the range of 6800 km³ and 8000 km³ (White 2001, ICOLD 2009, Wisser et al. 2013). As shown in Figure 2.2, the net storage capacity and the per capita storage volume decrease due to reservoir sedimentation, even though new reservoirs are built and existing ones are enhanced. Considering the estimated population growth and increasing per capita consumption, the occurrence of water supply shortage in the near future becomes inevitable (White 2001, Kundzewicz et al. 2008, Annandale 2013, 2014, Wisser et al. 2013). The situation will be more dramatic in the future due to the impacts of climate change, requiring for larger storage capacities.

Reservoir sedimentation furthermore causes a loss of hydroelectric power production, a deterioration of water quality, an endangerment of safe and reliable water supply and irrigation,
Reservoir sustainability and sediment bypass tunnels

A reduced flood protection function and an increase of wear at hydraulic structures. Furthermore it can endanger the operating safety or even the dam stability if the aggradation body reaches the dam. The accumulated sediments cause a bed level rise at the reservoir head, which increases flood risk. A sediment deficit, termed “hungry water” by Kondolf (1997), occurs downstream of the reservoir. As a consequence, river bed incisions, endangerment of embankment stability, scour of hydraulic structures and degradation of the river bed may arise (Wilcock et al. 1996, Kondolf 1997, Morris and Fan 1998, Kantoush and Sumi 2010, Kantoush et al. 2010, Kondolf et al. 2014). The altered runoff hydrographs further affect the downstream sediment transport, and the lower discharges reduce the downstream ground water recharge leading to an undesirable lowering of the ground water level (Hartmann 2004). The lower ground water level promotes vegetation growth, reducing the discharge capacity of the river channel and provoking higher flood risk (Wilcock et al. 1996, Kondolf 1997, Fruchart and Camenen 2012). Moreover, the eco-morphology becomes impaired and the biodiversity impoverished, because invertebrates habitats and fish spawning areas disappear (Surian and Rinaldi 2003, Cajot et al. 2012, Fukuda et al. 2012, East et al. 2015, Facchini et al. 2015, Martin et al. 2015). The disturbed natural river characteristics cause even negative impacts on the coastal areas. Due to the reduction of sediments and nutrients normally carried in river water leads to coastal erosion and to a decline of fish population, respectively (Yang et al. 2006, Syvitski et al. 2009, Kantoush and Sumi 2010, Kondolf et al. 2014).

Over all, the total global costs (including loss water supply, hydroelectric power production, refurbishment, wear at hydraulic structures and downstream damages such as deterioration of the eco-morphology and habitat quality) due to reservoir sedimentation are approx. $1.7 \times 10^9$ US$ per year, but only a small share of this is actually spent (Basson 2009). In fact, the costs are mainly postponed to future generations while the current generation benefits from reservoirs, leading to an intergeneration unfairness (Brundtland 1987). A fair, cost-effective and sustainable use of reservoir requires for holistic sediment management.

2.3 Overview of countermeasures

A variety of measures to counteract reservoir sedimentation exists and is summarized in Appendix Figure A.1. Sumi (2005) and Kondolf et al. (2014) gave overviews on those measures and detailed explanations have been provided in many publications (Sumi 2000, Palmieri et al. 2001, White 2001, Schleiss and Oehy 2002, Sumi et al. 2004b, Knoblauch et al. 2007, Sumi et al. 2009, Annandale 2011, Auel and Boes 2011, Kantoush et al. 2011, Fruchart and Camenen...
Reservoir sustainability and sediment bypass tunnels

2012, Kondolf et al. 2014, Wang and Kondolf 2014, Boes and Hagmann 2015, Auel et al. 2016c). As shown in Figure 2.3, reservoir sedimentation countermeasures can be divided into four categories according to their purpose: (1) sediment yield reduction, (2) sediment routing, (3) sediment removal and (4) optimizing the location and design of the dam and the reservoir. Although providing a larger dead storage capacity for sedimentation is sometimes regarded as a possible countermeasure (Kondolf 1997, Morris and Fan 1998, Schleiss and Oehy 2002), this method only postpones the decommissioning but not efficiently solves sedimentation problems. Therefore, this option is not considered as a sustainable reservoir sedimentation countermeasure herein.

![Figure 2.3](image)

Figure 2.3 Schematic overview of the reservoir sedimentation countermeasure concepts: (1) sediment yield reduction in the catchment, (2) sediment routing around the reservoir, (3) sediment removal from the reservoir, (4) optimized location and design of reservoirs to avoid reservoir sedimentation

Sediment yield reduction is a preventive concept that often requires long-term and expensive measures over large areas. Its effectiveness has not been clearly demonstrated. The disadvantages of this concept are (I) the sediment transport continuity is not restored and (II) it can lead to a release of an enormous and harmful sediment pulse in the event of a failure (Annandale 2011, Wang and Kondolf 2014, Auel et al. 2016c).

In contrast to sediment yield reduction, sediment routing acts in real time. The inflowing sediments are diverted into the tailwater without settling and thus not only prevent reservoir sedimentation but also restore the native sediment transport in downstream river reaches (Morris and Fan 1998). The main flaw of this concept is that sediment routing through the outlet structures, which are usually not designed for such applications, can cause severe abrasion damage and hence endangers structural stability and operating safety (Auel and Boes 2011). SBTs are a part of this concept and are described in detail in Chapter 2.4.

The removal of the settled sediments from the reservoir is a retroactive measure, which is only economical for small reservoirs (Boes et al. 2014). Its effect is limited in time and space and
the released water for flushing can exhibit undesirably high sediment concentration causing detrimental ecological impacts on the downstream river reach (Lai and Shen 1996). Moreover, a significant financial loss with a potentially long lasting refilling is provoked by the reservoir drawdowns.

The fourth concept concerns the optimal location and design of the dam and the reservoir. This concept has a preventative character since it acts from the onset of the design through the construction and beyond. It is also less restricted by law in most cases (Müller and De Cesare 2009, Boes 2011) and therefore a preferred measure for facilities in the planning stage.

### 2.4 Sediment bypass tunnels

#### 2.4.1 General remarks

SBTs avoid or at least reduce reservoir sedimentation by diverting sediment-laden inflows with rates and timing similar to pre-dam conditions around the reservoir (Fukuroi 2012, Boes et al. 2014, Kondolf et al. 2014, Hagmann et al. 2016). As a consequence, they restore the sediment continuity and hence enhance the quality and quantity of the affected benthic habitats in the downstream river sections (Facchini et al. 2015, Auel et al. 2017c, Martin et al. 2017). Moreover, SBTs increase the outflow capacity and enhance the operating safety of dams, which is not sufficient anymore at many schemes due to climate and hydrological changes (Trucco 1981, Bruschin et al. 1982, Kundzewicz et al. 2008, Wisser et al. 2013, Annandale 2014).

SBTs are generally operated at discharges above a certain threshold value correlating with significant sediment concentrations in the inflow. Bypassing depends on the runoff regime, the hydrology and topography and takes place for a few hours during flood events or for several days, weeks or even months during high-flows or rainy seasons (Vischer et al. 1997, Kantoush and Sumi 2010, Auel and Boes 2011).

The first SBTs in the world were built at the beginning of the 20th century, whereas the majority of SBTs was commissioned within the past 50 years. However, the global number of existing SBTs is still low due to the high costs for construction and maintenance as well as a lack of knowledge regarding design and selection of suitable invert materials. Nevertheless, a major effort on SBTs has been made mainly in Switzerland and Japan. Other SBTs are in operation, under construction or planned in Ecuador, France, Iran, United States and Taiwan, amongst others (Knapp 2008, Auel and Boes 2011, De Cesare et al. 2011, Fukuda et al. 2012, Sumi et al. 2012, Tomita et al. 2012, Carlioz and Peloutier 2014, Wang and Kondolf 2014, Baumer and
Reservoir sustainability and sediment bypass tunnels


Figure 2.4 shows the general design of SBTs including a guiding structure, an intake structure, the tunnel itself and an outlet structure. These structures are briefly described in the following. More detailed information is reported in literature (Delley 1988, Vischer and Chervet 1996, Harada et al. 1997, Kashiwai et al. 1997, Vischer et al. 1997, Auel et al. 2010, Kantoush et al. 2011, Sumi et al. 2012). The guiding structure directs sediment-laden inflows to the SBT intake, where the water-sediment mixture is discharged through the tunnel around the dam and released by the outlet structure into the tailwater. Depending on the intake location, two different types of inflow occur: (A) free-surface flow when the intake is located at the reservoir head (Figure 2.4a) and (B) pressurized flow when the intake is situated at the pivot point of the accumulation body downstream of the reservoir head (Figure 2.4b). Type (A) SBTs protect the whole storage volume from reservoir sedimentation. However, the length of the tunnel may become long depending on the reservoir geometry and topography causing high investment cost. Type (B) SBTs require reservoir water level lowering to ensure that inflowing sediments reach and enter the SBT intake. This results in the release of otherwise productive water and hence revenue losses. Moreover, the operation of the SBT needs to be carefully coordinated with the hydraulic conditions in the reservoir and the inflow, which is challenging and can lead to lower bypass efficiencies (Boes et al. 2018). Therefore, most existing bypass tunnels are of type (A) featuring an intake at the reservoir head.
2.4.2 Existing sediment bypass tunnels

Data of existing SBTs are compiled from the literature and questionnaires completed by the facility owners or operators (Brechtbühl 1994, Delpino et al. 2008, Knapp 2008, Amberg 2009, Axpo 2011, Capatt 2011, Radogna 2011, Züger 2011, Wehrli 2012, ILF 2013, Sonderegger and Kroulik 2013, Auel et al. 2014a, Auel et al. 2017d). Key data such as the year of the commissioning, SBT length $L$ and slope $S$, catchment area, reservoir storage capacity $CAP$, capacity-inflow ratio $CIR$, design discharge $Q_d$ and its flood return period as well as 1 and 100 years flood discharges $HQ_1$ and $HQ_{100}$, respectively, are listed in Table 2.2. Furthermore, the hydraulics, sediment properties and hydroabrasion data of the SBTs are listed in Table 2.3 (in subsection 2.4.4), while a compilation of the geometries is given in the Appendix Table B.1. It is noted that although some facilities are by definition flood bypass tunnels, they are also considered as SBTs herein due to the high sediment transport rates conveyed during flood peaks.

The SBT tunnel length is generally shorter than 2 km and the slope varies between 1 and 4%. Only the Jotty SBT with $S = 0.006$ and the Zengwen SBT with $S = 0.0532$ as well as the Rizzanese SBT with $S = 0.069$ have a shallower and steeper bottom slope, respectively. The SBT design discharge varies between 38 and 1000 m$^3$/s, corresponding to a flood discharge with a return period of a few years.

Regarding the reservoir volume, i.e. the storage capacity $CAP$, SBTs are typically installed at small to medium size reservoirs impounding up to a few million m$^3$. There are only few larger reservoirs with a storage capacity of a few dozen million m$^3$. Figure 2.5a shows the relation between the capacity and the mean annual runoff $MAR$ of the existing SBTs. The average residence time, determined by the capacity-inflow ratio, varies around $CIR = 0.01$ yr. This means that most SBTs are constructed at daily or weekly reservoirs.

The reservoir life span as defined by the ratio of capacity to mean annual sediment inflow $CAP/MAS$ is shown in Figure 2.5b. It is generally less than 50 years, indicating a high sedimentation rate of more than $1/50 = 2\%$, whereas only a few exceptions with a life span of more than 100 years exist.

In summary, SBTs are mainly installed at daily or weekly reservoirs with a sedimentation rates beyond 1%. Reasons for that are technical and in particular economic limits for tunnel dimensions and feasible design discharge.
Table 2.2  Key data of existing SBTs at dam reservoirs

<table>
<thead>
<tr>
<th>Country</th>
<th>Name</th>
<th>Completion</th>
<th>$L^1$ [m]</th>
<th>$S^2$ [%]</th>
<th>Cutslope Area [km$^2$]</th>
<th>CeP [10$^6$m$^3$]</th>
<th>CIR</th>
<th>$Q_{di}$ [m$^3$/s]</th>
<th>Return period of $Q_{di}$ [yr]</th>
<th>$H_{Q_{di}}$ [m]</th>
<th>$H_{Q_{max}}$ [m$^3$/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH</td>
<td>Egschi</td>
<td>1976</td>
<td>20/360</td>
<td>21/2.6</td>
<td>109</td>
<td>0.51</td>
<td>0.0067</td>
<td>50</td>
<td>&gt;1</td>
<td>15</td>
<td>ns</td>
</tr>
<tr>
<td>CH</td>
<td>Hintersand</td>
<td>2001</td>
<td>25/1050</td>
<td>10/1.2</td>
<td>35</td>
<td>0.11</td>
<td>0.0014</td>
<td>38</td>
<td>~5</td>
<td>ns</td>
<td>60</td>
</tr>
<tr>
<td>CH</td>
<td>Palagnedra</td>
<td>1978</td>
<td>50/1760</td>
<td>29.6/2</td>
<td>138</td>
<td>4.8</td>
<td>0.019</td>
<td>220</td>
<td>3</td>
<td>ns</td>
<td>1000</td>
</tr>
<tr>
<td>CH</td>
<td>Pfaffensprung</td>
<td>1922</td>
<td>25/282</td>
<td>35/3</td>
<td>390</td>
<td>0.17</td>
<td>0.0003</td>
<td>220</td>
<td>1</td>
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<td>ns</td>
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<tr>
<td>CH</td>
<td>Rempen</td>
<td>1986</td>
<td>22/470</td>
<td>25/4</td>
<td>25</td>
<td>0.48</td>
<td>0.003</td>
<td>80</td>
<td>&gt;100</td>
<td>&gt;30</td>
<td>&gt;50</td>
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<tr>
<td>CH</td>
<td>Runcahez</td>
<td>1962</td>
<td>85/572</td>
<td>25/1.4</td>
<td>56</td>
<td>0.44</td>
<td>0.006</td>
<td>110</td>
<td>2</td>
<td>40</td>
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2.4.3 Bypass efficiency

The concept of SBTs is to divert inflowing sediment to the downstream river reach without deposition in the reservoir. Its performance can be quantified by the bypass efficiency (defined as the ratio of bypassed sediment volume to inflow sediment volume) or by the reservoir lifetime enhancement (Auel et al. 2016c).

A thoroughly investigated and documented case study is the Asahi SBT in Japan (Kataoka 2000, 2003, Sumi et al. 2004b, Mitsuzumi et al. 2009, Sumi and Kantoush 2011, Akiyama 2012, Fukuda et al. 2012, Fukuroi 2012, KEPCO 2012). The sediment inflow was estimated using a one-dimensional numerical model, which was calibrated with both bathymetric measurements and bed elevation survey data downstream of the dam. Figure 2.6 shows the annual and cumulative sediment inflow and accumulation of the Asahi reservoir between 1989 and 2013. Since the commissioning of the Asahi SBT the estimated bypassed sediment averaged 77% of the inflowing sediment load prolonging the remaining reservoir life span by approx. 3.3 times from 190 to 636 years (Auel et al. 2016c). When the Typhoon Talas struck Japan in 2011, it caused severe landslides and flooding in the Asahi catchment. During this typhoon, still 66% of the sediment load were bypassed (Auel et al. 2016c), and a part of the infilled reservoir volume was re-established after the typhoon in 2012 as indicated by the negative accumulation volume in 2012.

The bypass efficiency of two other Japanese SBTs, i.e. the Miwa and the Nunobiki SBT show comparable bypass efficiencies of 80% and 81-95%, respectively. The coarse sediment at the
Miwa SBT was efficiently trapped behind a check dam, while up to 60% of the fine sediments were bypassed around the reservoir (Sumi et al. 2004b, Sumi et al. 2012). The bypass efficiency at the Nunobiki SBT is 81% and reaches 95% if the effect of the Sabo dams is included (Sumi et al. 2004a, b, Auel et al. 2016c). Hence the remaining reservoir life time $RL$ of the Nunobiki reservoir (commissioned in 1900) was extended by approx. 24 times from 52 years to 1256 years by the commissioning of the SBT in 1908 (Auel et al. 2016c).

In contrast to Japan, the aggradation and sediment transport monitoring at Swiss reservoirs is limited to bathymetric observations and flushing monitoring. The Egschi SBT, located in the Swiss Alps, has almost stopped reservoir sedimentation (Vischer et al. 1997). The Palagnedra reservoir situated in the south of Switzerland had suffered an annual storage loss of 100,000 m$^3$ before a SBT was constructed 20 years after the dam commissioning (Martini 1981, Vischer et al. 1997). It was reported that the SBT significantly reduced the annual sedimentation and overcame a severe flood event, which occurred just after its construction (Vischer et al. 1997).

### 2.4.4 Invert abrasion

Since SBTs divert sediment-laden discharges at high velocities, high hydroabrasive impact on the tunnel invert is expected. Although various invert materials have been used to the date, severe wear has been unavoidable (Hagmann et al. 2012a, Hagmann et al. 2012b, Auel et al. 2015b). Figure 2.7 shows examples of different types of concrete with and without reinforcement, cast basalt and granite linings exposed to hydroabrasion. The damages clearly
show that hydroabrasion can be a major issue at hydraulic structures. Annual refurbishment costs can reach up to 1% of the construction costs of SBTs (Auel 2014).

Figure 2.7 Hydroabrasion examples: a) up to 2 m deep abrasion at the Val d’Ambra SBT into the concrete lining and underlying rock (M. Müller-Hagmann), b) several decimeter deep incision channels at the reinforced concrete lining of the Asahi SBT (Kansai 2012), c) abraded cast basalt plates in the Pfaffensprung SBT (M. Müller-Hagmann) and d) abraded granite lining of the Pfaffensprung SBT (VAW)

Hydroabrasion is not only present in Alpine regions at SBTs but also in lowland regions, with lower hydroabrasive impact. In Germany, 50% of all weirs, observed by the Federal Waterways and Shipping Authority suffer from hydroabrasion damage, and at 20% of those weirs, the serviceability or stability is endangered (Spörel et al. 2015). Application of suitable abrasion-resistant materials can reduce such damages. However, there is no guideline available and invert material selection is often based on trial and error field experiences. Since the damage potential is high and willingness of taking a risk by applying novel materials is limited, the spectrum of implemented materials is narrow. Table 2.3 lists the invert materials and their properties used in existing SBTs. The most common invert material is concrete. Its 28-day compressive strength varies from 22 to 101 MPa with a majority lying between 50 and 70 MPa (Martini 1981, Delley 1988, Vischer et al. 1997, Jacobs et al. 2001, Hagmann et al. 2014). The second most widely
used material, particularly in Switzerland, is rock such as granite and cast basalt, whereas steel is rarely and only locally implemented at highly stressed tunnel sections for economic reasons.

Table 2.3 also lists information on hydraulic operation conditions, i.e. maximum flow velocity $U_{\text{max}}$, flow velocity at the outlet $U_{\text{out}}$, uniform flow velocity $U_{\text{u}}$ and Froude number at the outlet $F_{\text{out}}$, sediment properties, operation duration and abrasion depths. The design flow regime in the existing SBTs is generally supercritical free-surface flow, while some of them can be also operated under pressurized conditions. The design discharges range from a few dozen to a few hundred m$^3$/s. The design velocities are between 4 and 15 m/s and corresponding Froude numbers are $F = 1 - 4$, defined for rectangular cross-sections as:

$$F = \frac{U}{\sqrt{gh}} \quad [-] \quad (2.4)$$

where $U =$ mean flow velocity, $g =$ gravitational acceleration and $h =$ mean flow depth. Exceptions are the not yet commissioned Nanhua and Zengwen SBTs with design velocities of 24.5 and 18 - 30 m/s, respectively (Sumi and Auel 2016, VAW 2016b).

At the listed SBTs, the annual sediment supply reaches between a few thousand up to several 100'000 m$^3$, containing different quartz contents and various grain size distributions (GSD) (characterized by the mean and 90-percentile grain diameter $d_m$ and $d_{90}$, respectively). The GSD depends on geology, transport distance and petrography and varies from some millimeters up to some decimeters. The abrasion rate is strongly site-specific and spreads over a large scale. The abrasion rates $a$ vary from 1 up to 22 millimeters per year. However, during extraordinary events, abrasion rates can be an order of magnitude higher. Any initial local irregularity on the invert causes self-intensifying abrasion leading to high material losses and weakening of the surrounding area, which in return promotes the abrasion process.

The tunnel axis has a significant influence on the sediment transport distribution and hence the abrasion pattern across the tunnel width. The 505 m long and 2.74 m wide Mud Mountain SBT is an excellent example (VAW 2016a). This SBT has a left hand bend after the inlet and right hand bend upstream of the outlet (compare plan view in Table B.1 in the Appendix). The tunnel was lined in 1995 with a 2.54 cm thick steel plate (Knapp 2008). Its remaining thicknesses and abrasion depths were determined in 2013 (ILF 2013). Figure 2.8a shows the abrasion depths at the center line ($y = 1.37$ m), along the right ($y = 0.15$ m) and the left wall ($y = 2.59$ m), respectively. This demonstrates that the abrasion pattern is affected by the plan view geometry of the tunnel. The deepest abrasion depths in the left hand bend occurred along the left tunnel wall. After the bend, the abrasion pattern was still affected by the strong bending effect.
(explained in Chapter 3.1.3) until \( x \approx 130 \) m. The steel lining was partially removed and several decimeter deep incision channels had scoured the underlying material along the left tunnel wall (Auel et al. 2017d). In the following transition zone between \( x \approx 130 - 200 \) m, the effect of the bend diminished with increasing distance to the left hand bend, and the maximum abrasion depths occurred along the tunnel center line between \( x \approx 200 - 450 \) m. The abrasion pattern is dominated by the hydraulic and sediment transport conditions of the straight section. In the right hand bend at the outlet, the maximum abrasion depths shifted from the center line to the right tunnel side. The sections I, II and III marked in Figure 2.8a represent the sections affected by the left hand bend, straight section and right hand bend, respectively. The longitudinally averaged cross section profiles along these sections are shown in Figure 2.8b and confirm the strong effect of tunnel plan view geometry. Bends result in strong abrasion concentration on the inner side of the curvature and affect downstream sections as well.

Figure 2.8  a) Longitudinal abrasion profiles of the Mud Mountain SBT for the right side, left side and center line, gray marked the sections used to compute b) the section wise averaged cross section abrasion profiles affected by bended and straight sections

In general, hydroabrasion problems arise if sediment has a considerable quartz content due to its high hardness (Vischer et al. 1997, Sumi et al. 2004b). However, existing hydroabrasion prediction models (Sklar and Dietrich 2004, Lamb et al. 2008, Sklar and Dietrich 2012, Auel et al. 2016a) do not account for sediment hardness as well as extraordinary hydraulic and sedimentological operating conditions occurring in SBTs. To address this gap of knowledge further data on various invert materials, sediment properties, operating conditions and hydroabrasive damages is required.
Table 2.3  Invert material, hydraulic conditions for design discharge, sediment parameters and mean annual abrasion depths of existing SBTs

<table>
<thead>
<tr>
<th>Name</th>
<th>Material</th>
<th>$f_c$</th>
<th>$Q_d$</th>
<th>$U_{\text{max}}$</th>
<th>$U_{\text{out}}$</th>
<th>$U_u$</th>
<th>$F_{\text{out}}$</th>
<th>Sediment inflow</th>
<th>Bypassed sediment</th>
<th>$d_{m}$</th>
<th>$d_{90}$</th>
<th>Quartz content</th>
<th>Operation</th>
<th>$a$</th>
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2.4.5 Eco-morphological effects

SBT operation also has an impact on river bed development, water level elevations, eco-morphology, as well as aquatic and riparian habitats.

An investigation carried out downstream of the Asahi SBT showed that SBT operation improved the eco-morphology of the river, aquatic and riparian habitats and restored the natural grain size distribution to pre-dam conditions (Fukuda et al. 2012). As a result, sand bars, which had been disappearing since the dam commissioning, revived the impaired morphology progressively. Finally, the river system recovered and increased the habitat quality in the river reach (Akiyama 2012, Kondolf et al. 2014).

Another success of the Asahi SBT was the improvement of the water quality downstream of the dam by an increase of the reservoir’s water residence time by 6. This decreased the mean sediment concentration from 16 to 4 mg/l (Akiyama 2012) and the duration of long time turbid water from 50 to 9 days on average (Fukuroi 2012). Moreover, the nutrient transport to coastal zones has been re-established and the presence of red tide in the freshwater has been decreased (Akiyama 2012).

The results of systematic numerical simulations on the downstream morphological effects of SBTs by Facchini (2017) indicate that sediment feeding by SBTs changes the downstream riverbed slope towards an equilibrium (where the slope of the upstream river stretch is the reference) and the riverbed surface sediment composition towards that of the feeding material. However, riverbed level and riverbed surface composition evolve on different time scales, the latter being much faster, so that even after only a few SBT operations the riverbed surface composition is already close to the equilibrium configuration and is then reworked at each operation. The lower the ratio of the bedload feed rate, the more operations are needed to completely change the fractions of the single grain classes in the active riverbed layer. In contrast, the steepening of the riverbed slope (1D morphological effect) is slow, requiring thousands of SBT operations until an equilibrium is reached. Sediments released from typical SBT operations behave like sediment pulses, i.e. they translate, disperse or have a mixed behavior as function of the hydraulic conditions and sediment feed characteristics.
3 Theoretical background

This section provides a physical background and state of the art on the flow characteristics in open channels, sediment transport over movable and fixed beds, and hydroabrasion. Afterwards, a summary of abrasion prediction models accounting for the physical processes causing hydroabrasion in SBTs is provided.

3.1 Hydraulics

While most SBTs are gate-controlled at the intake, so that the flow regime downstream of the gate is free-surface, some SBTs are controlled at their downstream end, e.g. due to spatial restrictions hindering the arrangement of the gate at the intake. For downstream control, the flow regime may change from free-surface via transitional to pressurized, mainly depending on the gate opening, the bed slope and the available head at the intake. Because the flow regime has a considerable effect both on the sediment transport and the transport of air being potentially entrained at the intake, the hydraulic design is crucial for the safe and reliable operation of SBTs (Boes et al. 2017). Herein, focus is put on free-surface flows as these are most common and should be aimed at in SBT design.

Knowledge of the characteristics of free-surface or open channel flow are relevant for the understanding of hydraulic engineering problems, particularly for flow-sediment interaction, channel abrasion and sediment erosion, transport and deposition. There are many investigations on the mean and turbulence characteristics of open channel flow enabling quantification of important flow parameters, e.g. distribution of time-averaged velocities, turbulence intensities and bed shear stresses (Nikuradse 1933, Kline et al. 1967, McQuivey and Richardson 1969, Blinco and Partheniades 1971, Grass 1971). A comprehensive overview, in particular for supercritical open channel flows, which are typical for SBTs is provided by Auel et al. (2014b). In the following, definitions of the most important parameters are provided first, followed by a summary of the formulas used to determine the flow velocity in the SBTs and in the rivers. Finally, the secondary currents occurring in SBT and their effects on the sediment transport are treated.
3.1.1 Parameter definition

Widely applied parameters regarding flow characteristics are the dimensionless Froude number \( F \), Reynolds number \( R \), the bed shear stress \( \tau_b \) and the friction velocity \( U_* \) defined as:

\[
F = \frac{U}{\sqrt{g \cdot A / b_{ws}}} \quad [-] \quad (3.1)
\]

\[
R = \frac{4UR_h}{\nu} \quad [-] \quad (3.2)
\]

\[
\tau_b = \sqrt{\rho \cdot g \cdot R_h \cdot S_e} \quad [\text{Pa}] \quad (3.3)
\]

\[
U_* = \sqrt{g \cdot R_h \cdot S_e} = \sqrt{\tau_b / \rho} \quad [\text{m/s}] \quad (3.4)
\]

where \( U \) = mean flow velocity, \( g \) = gravitational acceleration \( A \) = wetted cross section, \( b_{ws} \) = water surface width, \( R_h \) = hydraulic radius and \( \nu \) = kinematic viscosity, \( \rho \) = fluid density and \( S_e \) = energy slope. For rectangular shaped channels \( A / b_{ws} \) can be replace by \( h \) = mean flow depth. For uniform flow, \( S_e \) equals the bottom slope, whereas it can be computed by means of backwater calculations for non-uniform flow.

The flow in SBTs is typically supercritical and turbulent, namely characterized by the Froude number \( F > 1 \) and by the Reynolds number \( R > 2300 \), respectively. River flows are mostly turbulent, and are sub- as well as supercritical depending on the discharge (Jirka and Lang 2009).

3.1.2 Flow resistance

From the mixing length theory suggested by Prandtl (1933) the universal logarithmic velocity distribution known as “log law” results:

\[
\frac{u(z)}{U_*} = \frac{1}{\kappa} \ln \left( \frac{z}{\delta} \right) + \text{const.} \quad (3.5)
\]

where \( u(z) \) = time-averaged velocity at level \( z \), \( U_* \) = friction velocity (Equation (3.4)), \( \kappa \) = van Karman constant (usually \( \kappa = 0.41 \), e.g. Bose and Dey 1986), and a constant, for which a boundary condition is needed. Equation (3.5) applies on hydraulic smooth wall conditions \( (z/h < 0.2 \) near bed), but is also commonly used as approximation for transitionally rough beds, (Nezu and Nakagawa 1993). Nikuradse (1933) proposed the logarithmic law for transitionally and complete rough bed conditions as:
\[
\frac{u(z)}{U_*} = \frac{1}{\kappa} \ln \left( \frac{z}{z_0} \right)
\]

or
\[
\frac{u(z)}{U_*} = \frac{1}{\kappa} \ln \left( \frac{zU_*}{\nu} \right) + A - \Delta U^+
\]

with the zero-velocity level \( z_0 = k_0/30 \) for hydraulic rough conditions, as existing in SBTs, \( k_0 \) = equivalent sand roughness, \( A \) = integral constant and \( \Delta U^+ \) = roughness shift. Based on open channel flow experiments over a transitionally rough bed, Auel et al. (2014b) revealed that Equation (3.6) fits well with the experimental data in particular in the inner region \((z/h < 0.2)\), whereas the velocity profile deviates from the log-law in the outer region \((z/h > 0.2)\) due to free-surface and non-uniform flow effects. The roughness shifts of their experimental data followed Colebrook’s function given by Hama (1954) as
\[
\Delta U^+ = 2.46 \ln \left( k_0 U_*/\nu + 3.3 \right) - 2.92
\]
and the integral constant was \( A = 5.29 \), which is in a good agreement with literature data (Nikuradse 1933, Colebrook 1939, Bergstrom et al. 2001, Schultz and Flack 2003, Bigillon et al. 2006, Shockling et al. 2006, Schultz and Flack 2007, Langelandsvik et al. 2008). By adding the wake function introduced by Coles (1956) the adapted log-law describes the entire velocity profile except for the viscous sublayer over a smooth bed (Nezu and Rodi 1986):
\[
U^+ = \frac{1}{\kappa} \ln \left( z^+ \right) + A + \frac{2 \Pi}{\kappa} \sin^2 \left( \frac{\pi z}{2h} \right)
\]

The wake parameter, \( \Pi \) expresses the strength of the wake function and increases monotonically with the Reynolds number, starting at 0.10 and asymptotically reaching 0.55 (Coles 1956, Cebeci and Smith 1974, Nezu and Rodi 1986).

The mean flow velocity \( U \) is obtained by integrating the log-law over flow depth \( h \). The Darcy-Weisbach flow equation, a common approach used in hydraulic engineering application, follows as:
\[
U = \sqrt{\frac{8g}{\lambda} R_0 S_{10}} \quad [\text{m/s}]
\]

The friction coefficient \( \lambda \) depends on the Reynolds number and bed roughness and can be expressed as (von Karman 1930, Nikuradse 1932, Prandtl 1932, Nikuradse 1933, Colebrook 1939):
\[ \lambda = \frac{64}{R} \quad [-] \quad \text{Laminar flow for } R < 2300 \quad (3.9) \]

\[ \frac{1}{\sqrt{\lambda}} = -2 \log \left( \frac{k_s / D}{3.71} + \frac{2.51}{R \sqrt{\lambda}} \right) \quad [-] \quad \text{Turbulent flow for } R \geq 2300 \quad (3.10) \]

where \( D = 4R_h \) = hydraulic diameter for open channels. Solving Equation (3.8) for \( h_f = S_e \cdot L \) describes the friction loss in an open channel flow along a flow reach length \( L \), expressed as:

\[ h_f = \frac{\lambda \cdot LU^2}{8R_h g} \quad [m] \quad (3.11) \]

A further widely applied empirical flow equation is the Gauckler-Manning-Strickler equation (Strickler 1923):

\[ U = k_{St} \cdot R_h^{2/3} \cdot S_e^{1/2} \quad [m/s] \quad (3.12) \]

with \( k_{St} \) = Strickler’s value for bed roughness, which is crucial for the calculation of mean streamwise flow velocity. For common natural and artificial surfaces \( k_{St} \)-values are given in literature (Naudascher 1992). It can also be determined using a characteristic sediment particle diameter. Literature provides a wide range of relation between particle size and \( k_{St} \) (Marriott and Jayaratne 2010). However, a widely applied and well accepted relationship was introduced by Strickler (1923) and confirmed by Jäggi (1984) and follows as:

\[ k_{St} = \frac{21.1}{d_{s0}^{1/6}} \quad [m^{1/3}/s] \quad (3.13) \]

### 3.1.3 Secondary currents

Cellular secondary currents caused by either interaction between side-wall and free-surface, turbulence anisotropy or bed forms create zones of alternating up- and downflows with corresponding low and high bed shear stresses (Nezu and Rodi 1985, Nezu and Nakagawa 1993, Rodriguez and Garcia 2008, Albayrak and Lemmin 2011). The secondary currents re-distribute the turbulence intensities and Reynolds shear stresses in the water column and bed shear stresses in spanwise direction and hence are of particular interest concerning sediment transport and invert abrasion (Auel et al. 2014b).

Prandtl (1949) defined two types of secondary currents. The **first type of Prandtl’s secondary current** occurs in curved channels and meandering rivers. A spiral flow driven by centrifugal forces cause a concentration of sediment transport at the inner side of the curve (Figure 3.1a). This phenomenon is termed bending effect. The intensity of secondary currents increase with
decreasing curvature radius, increasing deflection angle and decreasing aspect ratio $b/h$ with $b =$ flow width and $h =$ flow depth (Kashyap et al. 2012).

The second type of Prandtl’s secondary current occurs in straight channels due to non-homogeneity and anisotropy of turbulence. The entire flow cross section is affected in narrow open channel flow with low aspect ratios of $b/h < 4 - 5$, and the maximum streamwise flow velocity is located below the water surface, termed velocity dip phenomenon (Rajaratnam and Muralidhar 1969, Nezu et al. 1985, Nezu and Rodi 1985, Nezu and Nakagawa 1993, Yang et al. 2004). Moreover, the bed shear stresses are 20 - 50% higher along the walls compared to the cross-sectional mean values. These effects are shown in Figure 3.1b, where the normalized streamwise flow velocities and the corresponding normalized spanwise bed shear stress distribution are plotted for $F = 2.0$, $b/h = 2.8$ (Auel 2014). The effect of secondary current remains strong at the side walls, but their effect on the flow field decreases with increasing aspect ratio until a two-dimensional flow is established at the channel center (Nezu and Nakagawa 1993, Albayrak and Lemmin 2011, Auel et al. 2014b).

Figure 3.1  a) Flow field, sediment transport and bed shear stress distribution in open channel flows at curvatures, and b) cellular secondary currents, normalized streamwise velocity distribution and bed shear stress distribution in a straight narrow open channel flow for $F = 2.0$ and $b/h = 2.8$ (Auel et al. 2014b)

Despite higher Froude numbers and the absence of bed forms Auel’s (2014) results are in a good agreement with the results of former investigations on plane beds (Knight et al. 1984, Nezu et al. 1985, Tominaga et al. 1989, Nezu 2005) and fixed or immobile alluvial beds (Gosh and Roy 1970, Nezu and Nakagawa 1984, McLelland et al. 1999, Albayrak and Lemmin 2011).
3.2 Sediment transport

Sediment transport capacity is defined as the potential sediment volume or mass per unit time transported in uniform open channel flow without erosion or deposition. It represents the upper limit of sediment transport rate, which is a parameter of particular interest regarding hydroabrasion. Numerous investigations have been conducted to determine sediment transport capacities (Meyer-Peter and Müller 1948, Einstein 1950, Pedrol 1963, Ackers and White 1973, Novak and Nalluri 1975, Fernandez Luque and Van Beek 1976, Haenger 1979, Smart and Jaeggi 1983, Novak and Nalluri 1984, Gomez and Church 1989, Rickenmann 1990, Mayerle et al. 1991, Yang and Wan 1991, D’Agostino and Lenzi 1999, Cheng 2002, Abrahams 2003, Wong and Parker 2006, Chatanantavet et al. 2013). In addition, the type of particle motion is decisive regarding the abrasion process. Hence, in the following, criteria for the initiation of particle motion and different particle motion types are introduced, followed by a summary of sediment transport formulas.

Widely applied parameters regarding sediment transport investigations are the dimensionless particle diameter $D_*$, the Shields parameter $\theta$, which is also called non-dimensional shear stress, the particle Reynolds number $R_p$ and the transport stage $T^*$ defined as:

\[
D_* = d^3 \frac{(s-1)g}{v^2} [-] \quad (3.14)
\]

\[
\theta = \frac{U_*^2}{(s-1)gd} = \frac{R_pS_c}{(s-1)d} [-] \quad (3.15)
\]

\[
R_p = \frac{U_*d}{v} [-] \quad (3.16)
\]

\[
T^* = \left( \frac{U_*}{U_{*c}} \right)^2 = \frac{\theta}{\theta_c} [-] \quad (3.17)
\]

where $d =$ particle diameter and $s = \rho_d/\rho =$ ratio of solid to fluid density, $\theta_c =$ critical Shields parameter (Chapter 3.2.1), $U_{*c} =$ critical friction velocity for the initiation of particle motion and $U_* =$ friction velocity defined by Equation (3.4). Note that although the ‘transport stage’ $T^*$ and the ‘excess transport stage’ defined as $T^*-1$ are different, the latter is sometimes also represented by $T^*$ (Auel et al. 2017a).
3.2.1 The initiation of particle motion

Sediment transport is governed by the flow-induced bed shear stress and begins if the critical shear stress is exceeded. Based on laboratory flume experiments with uniform bed material Shields (1936) introduced the non-dimensional Shields parameter (Equation (3.18)), which is widely used for particle motion investigations. As soon as the non-dimensional Shields parameter exceeds a critical value, i.e. $\theta > \theta_c$, particle motion is initiated.

$$\theta = \frac{R_h S}{(s-1) d} [-] \quad (3.18)$$

This equation holds for relatively high flow depths defined as the ratio between hydraulic radius and grain size $R_h/d > 25$. The critical Shields parameter is illustrated as a function of particle Reynolds number in the widely used Shields diagram shown in Figure 3.2.

![Critical Shields parameter as a function of particle Reynolds parameter](image)

An overview on the initiation of particle motion studies in the 20th century was given by Buffington and Montgomery (1997) and recently by Chatanantavet et al. (2013). Considering Swiss torrents and SBT examples the critical Shields parameters for movable beds and for plane or bedrock beds are of interest. For the former $\theta_c = 0.03 - 0.086$ is suggested (Meyer-Peter and Müller 1948, Fernandez Luque and Van Beek 1976, Buffington and Montgomery 1997). For the latter $\theta_c = 0.002 - 0.006$ are generally accepted values (Smart and Jaeggi 1983, Hu and Hui 1996a, Chatanantavet et al. 2013, Auel 2014, Beer and Turowski 2015), whereas Abbott and Francis (1977) suggested $\theta_c = 0.05$, which is an order of magnitude higher. In the present study $\theta_c = 0.047$ for movable beds and $\theta_c = 0.005$ for plane beds are used for sediment transport calculations in rivers and SBT, respectively, following Meyer-Peter and Müller (1948) and Auel et al. (2017a), who particularly investigated sediment transport in SBTs.
3.2.2 Armor layer

The mobility of grains is non-homogeneous for a given discharge, leading to a selective erosion process. At relatively low and medium discharge levels, only the finer portion of the bed material is transported, whereas the coarser part remains immobile and builds a stable armor layer. This layer exhibits a higher erosion resistance compared to the normal bed material, protects the substrate against scour and increases the threshold for initiation of motion. There are several criteria for the development of an armor layer. Widely applied criteria are listed in Table 3.1 with different percentiles of the grain size (noted by the indices):

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gessler (1965)</td>
<td>$d_{84}/d_{50} &gt; 2.0$</td>
</tr>
<tr>
<td>Little and Mayer (1976)</td>
<td>$(d_{84}/d_{16})^{1/2} &gt; 1.3$</td>
</tr>
<tr>
<td>Chin (1985)</td>
<td>$d_{84}/d_{50} &gt; 1.5$ &amp; $d_{\text{max}}/d_{50} &gt; 1.8$</td>
</tr>
<tr>
<td>Schöberl (1992)</td>
<td>$(d_{84}/d_{16})^{1/2} &gt; 1.35$, $d_{90}/d_{50} &gt; 1.55$ &amp; $d_{m}/d_{50} &gt; 1.05$</td>
</tr>
<tr>
<td>Günter (1971)</td>
<td>$(d_{84}/d_{16})^{1/2} &gt; 1.4$ - 1.6</td>
</tr>
</tbody>
</table>

To account for the increased erosion resistance due to the armor layer, Günter (1971) suggested to adapt the critical Shields parameter, representing the threshold for initiation of bedload transport. The critical Shields parameter is adapted based on both the mean grain size of the armor layer $d_{m,\text{armor}}$ and of the substrate $d_{m,\text{substrate}}$ as follows:

$$\theta_{s}^{'} = \theta_{c} \left( \frac{d_{m,\text{armor}}}{d_{m,\text{substrate}}} \right)^{2/3} [-] \quad (3.19)$$

Sediment transport occurs either as the armor layer is scoured exposing the substrate, or if sediment is supplied from upstream reaches and hence is transported over the intact armor layer, or if suction removal of sediments from between the blocks of the armor layer happens. The occurrence of the latter depends on the grain sizes of the armor layer relative to the base sediment material. Based on laboratory flume experiments, Sumer et al. (2001) reported threshold Shields parameters $\theta_{su}$ as a function of the ratio between sediment and armor grain size $d/d_a$ for suction removal of sediment. They found the empirical relation:

$$\theta_{su} = 0.3 + 3 \left( \frac{d}{d_a} \right)^{-0.15} e^{-7.5 \frac{d}{d_a}} [-] \quad \text{for } 0.002 \leq d/d_a \leq 1.0 \quad (3.20)$$

The described criteria of armor layer building and suction removal were applied to determine the initiation of particle motion and hence of bed load transport.
3.2.3 Sediment transport mode

Depending on the Shields parameter, solid particles with different sizes are transported in different modes: sliding, rolling, saltation or suspension. When the Shields parameter exceeds the critical value, particles are transported in rolling or sliding motion in permanent contact with the bed (Bagnold 1973, Abbott and Francis 1977). With increasing Shields parameter the transport changes from rolling / sliding to saltation and further on to suspension. As shown in Figure 3.3, saltation is specified as a series of particle hops with regular trajectories accelerated downwards between the upward impulses sustained while in contact with the bed (Abbott and Francis 1977, Parsons et al. 2015). On contrary, suspension exhibits large trajectories, including upward directed vertical accelerations impaired by turbulence structures between two subsequent impacts. The trajectories show wavy and quasi-horizontal paths only limited by the flow depth. In contrast, sliding, rolling and saltation, i.e. bedload transport, only takes place near the bed up to a few grain diameters high. However, since the transition from one to another transport mode is smooth the determination of the particle motion threshold is quite challenging and moreover is affected by the definition.

Figure 3.3 Typical grain trajectories for (1) saltation and (2) suspension transport mode (adopted from Francis (1973))

Abbott and Francis (1977) investigated the probability of rolling, saltation and suspension in a laboratory flume. Figure 3.4a shows the observed percentage of the transport modes as a function of $U_\ast/U_{\ast c}$. In fact, different transport modes may occur simultaneously. The saltation mode is strongly present over a wide range of $U_\ast/U_{\ast c}$, whereas the share of suspension and rolling motion increases and decreases with increasing $U_\ast/U_{\ast c}$, respectively. Bose and Dey (2013) investigated particle motion and defined critical Shields parameters for no motion, bedload transport and suspension depending on particle Reynolds number $R_p$, shown in Figure 3.4b.
Auel et al. (2017a) experimentally investigated sediment transport characteristics in supercritical open channel flow over a fixed bed and revealed that particles were dominantly transported in saltation with minor parts in rolling mode and some small particles in suspension. This is an agreement with literature (Hu and Hui 1996a, Bose and Dey 2013). Based on their own data set including Hu and Hui’s (1996) set, Auel et al. (2017a) defined the rolling probability $P_R$ as:

$$P_R = 1.84 \left( \frac{\theta}{\theta_c} - 1 \right)^{-0.94} = 1.84 \cdot \left( T^* - 1 \right)^{-0.94} \text{ [-]}$$

The boundary between saltation and suspension is difficult to determine and depends on the definition. Hence, there is no strict boundary, rather a smooth transition from bedload to suspension. Several researchers reported different threshold values or equation for the suspension threshold (Bagnold 1941, Laursen 1958, Bagnold 1966, Smith and Hopkins 1972, Celik and Rodi 1984, Van Rijn 1984a, b, Sumer 1986, Nino et al. 2003, Eastwood et al. 2012, Bose and Dey 2013). Kresser (1963) developed a simple formula distinguishing between bedload and suspended load in natural rivers in terms of a threshold grain diameter:

$$d = \frac{U^2}{360g} \text{ [m]}$$

Suspension is driven by the downward particle settling velocity $V_s$ and the upward flow velocity associated with turbulent-induced fluctuations of the vertical velocity. Assuming a thin bed layer, the upward velocities can be scaled with friction velocity (Bose and Dey 2013). Hence, the ratio $V_s/U_*$ is suitable to define the boundary between saltation and suspension. Consequently, $V_s/U_*$ is often applied for suspension investigations (Eastwood et al. 2012) along...
with the Rouse number $P_{Rouse} = \frac{V_s}{\kappa U^*}$, which additionally includes the van Karman constant $\kappa = 0.41$ (Bose and Dey 2013). Since the definition of suspension motion is not distinct, different threshold values for $V_s/U^* = 0.3 - 2.5$ and $V_s/U^* = 0.3 - 1$ are provided for the initiation of suspension and for fully developed suspension, respectively (Bagnold 1941, Laursen 1958, Bagnold 1966, Smith and Hopkins 1972, Van Rijn 1984b, Nino et al. 2003, Eastwood et al. 2012). Instead of a fix value, Bose and Dey (2013) determined the suspension probability $P_{su}$ based on the ratio $V_s/U^*$:

$$P_{su} = \frac{1}{16} \left( 16 - \frac{V}{U^*} - \frac{V^2}{U^*^2} \right) \exp \left( \frac{V}{U^*} \right) \quad [-]$$

(3.23)

This equation is easily applicable, but can result in a wide range of suspension thresholds due to the non-objective selection of critical suspension probability.

### 3.2.4 Particle trajectories

Saltation particle trajectory and velocity are related to invert abrasion and bed rock incision problems since they determine the total energy transfer per unit invert area. Figure 3.5 schematically shows a particle trajectory, which is defined by hop length $L_p$, hop height $H_p$, and particle velocity $V_p$. The particle velocity is the resulting velocity of vertical and longitudinal particle velocity $W_p$ and $U_p$, respectively:

$$V_p = \sqrt{U_p^2 + W_p^2} \quad [\text{m/s}]$$

(3.24)

![Figure 3.5 Particle trajectory and corresponding parameters](image)

(2004, 2012) summarized and analyzed the literature data for subcritical and supercritical \((F \leq 2.0)\) flow regimes and proposed the following formulas for the saltation trajectories and particle velocity:

\[
U_p = 1.56\sqrt{(s-1)gd\left(T^* - 1\right)^{0.56}} \quad [\text{m/s}] \tag{3.25}
\]

\[
W_p = 0.4\sqrt{(s-1)gd\left(T^* - 1\right)^{0.18} \sqrt{1 - \left(U_*/V_s\right)^2}} \quad [\text{m/s}] \tag{3.26}
\]

\[
W_{im} = 3\frac{H_p U_p}{L_p} \quad [\text{m/s}] \tag{3.27}
\]

\[
H_p = 1.44\sqrt{T^* - 1}d \quad [\text{m}] \tag{3.28}
\]

\[
L_p = 8\left(T^* - 1\right)^{0.88}d \quad [\text{m}] \tag{3.29}
\]

where \(W_{im}\) = mean vertical particle impact velocity. To account for the transition to suspension, a non-linear function of settling velocity \(V_s\) and friction velocity \(U_*\) is introduced in Equation (3.26). This function indicates that hop length grows with increasing \(U_*/V_s\) and becomes infinite when \(U_*\) reaches \(V_s\). Particle fall velocity can be computed as follows (Ferguson and Church 2004):

\[
V_s = \frac{(s-1)gd^2}{18\nu + \sqrt{0.75(s-1)gd^3}} \quad [\text{m/s}] \tag{3.30}
\]

The reported formulas were confirmed by Chatanantavet et al. (2013) based on both, their own laboratory test results and data from the literature, and hence are applicable for the determination of particle motion in subcritical flows. However, the particle trajectories in supercritical flows, as occurring in SBTs, deviate from those in subcritical flows (Auel et al. 2017a). In contrast to the former investigations with mainly subcritical and rarely supercritical flow conditions Auel et al. (2017a, 2017b) experimentally investigated sediment transport in low to highly supercritical open channel flow with plane bed, comparable to the conditions in SBTs. Based on their results and the literature data (Ishibashi and Isebe 1968, Abbott and Francis 1977, Lee and Hsu 1994, Nino et al. 1994, Hu and Hui 1996a, Niño and García 1998, Ancey et al. 2002, Ancey et al. 2008, Lajeunesse et al. 2010, Chatanantavet et al. 2013), they proposed the following relations for particle motion in supercritical flows:
\[ V_p = 1.46 \sqrt{(T^*-1)(s-1)gd} \quad [\text{m/s}] \] (3.31)

\[ H_p = 0.27 \sqrt{(T^*-1)d} \quad [\text{m}] \] (3.32)

\[ L_p = 2.3(T^*-1)^{0.8} d \quad [\text{m}] \] (3.33)

\[ W_{in} = 0.1(T^*-1)^{0.39} \sqrt{(s-1)gd} \quad [\text{m/s}] \] (3.34)

### 3.2.5 Bedload transport on movable beds

The maximum possible sediment transport equals the sediment transport capacity. However, the effective bedload transport in a stream depends not only on transport capacity but also on sediment supply and the presence of an armor layer. If no sediment is supplied, bedload transport takes place only during high flows, which are able to scour the armor and expose the substrate to erosion. Hence, this represents the lower limit of bedload transport.

In this sub-section, different approaches to estimate the bedload transport capacity are presented. The bedload transport capacity is usually presented as the specific gravimetric bedload transport capacity \( q_s^* \) \([\text{kg/(s\cdot m)}]\), the volumetric bedload transport capacity \( q_v^* = q_s^*/\rho_s \) \([\text{m}^3/(\text{s\cdot m})]\) or as the non-dimensional volumetric bedload transport capacity \( q_{vn}^* \) [-]. The latter is introduced by Einstein (1950) as:

\[ q_{vn}^* = \frac{q_v^*}{\sqrt{(s-1)gd^3}} \quad [-] \] (3.35)

Fundamental research concerning bedload transport on movable beds was initiated by Meyer-Peter and Müller (1948) (MPM). They developed an empirical formula for the specific gravimetric bedload transport capacity based on laboratory experiments \((S = 0.0004 - 0.02, \quad d = 0.4 - 30 \text{ mm and } h = 0.01 - 1.20 \text{ m}):\n
\[ q_s^* = \frac{8}{(s-1)} \frac{g\rho_s}{k_{Str,S}} \left( \frac{k_{Str,S}}{k_{Str,r}} \right)^{1.5} R_s - \theta_c (s-1) d \quad [\text{kg/(s\cdot m)}] \] (3.36)

where \( \theta_c = 0.047, \quad k_{Str,S} \) and \( k_{Str,r} \) = the Strickler values for the bed and grain roughness, respectively, and \( R_s \) = hydraulic radius of the bed-related cross-sectional flow area in a partitioned cross-section. The formula has been verified for uniform coarse sand and gravel and is suggested to apply to gravel bed rivers with a slope \( S \leq 0.02 \). For steeper slopes, the sediment
transport capacity tends to be overestimated. By applying the non-dimensional volumetric bedload transport capacity and simplifying the roughness coefficient \((k_{Sl,S}/k_{Sl,r})^{1.5} = 1\), the formula can be re-written in a simpler form:

\[
q^*_{cm} = 8\left(\theta - \theta_c\right)^{1.5} \quad \text{[-]} \quad (3.37)
\]

Subsequent investigations showed that (MPM)’s equation overestimates bedload transport capacity. Fernandez Luque and Van Beek (1976) proposed to reduce the coefficient in Equation (3.37) from 8 to 5.7. Wong and Parker (2006) (WP) found the highest correlation with (MPM)’s data set by replacing this coefficient with 3.97 and using \(\theta_c = 0.0495\) instead of 0.047. With \(\theta_c = 0.047\) and an exponent of 1.6 (instead of 1.5) Wong and Parker (2006) alternatively proposed:

\[
q^*_{cm} = 4.93\left(\theta - \theta_c\right)^{1.6} \quad \text{[-]} \quad (3.38)
\]

Smart and Jaeggi (1983) (SJ) enlarged (MPM)’s data set by conducting laboratory experiments in a steep flume with \(S = 0.03-0.20\), \(d_m = 2 - 10.5\) mm and \(h = 0.03 - 0.09\) m. The proposed equation for \(S = 0.0005 - 0.20\) follows:

\[
q^*_{cm} = 4\left(\frac{d_{90}}{d_{30}}\right)^{0.2}S^{0.6}U\sqrt{\theta}\left(\theta - \theta_c\right) \quad \text{[-]} \quad (3.39)
\]

\[
q^*_c = 4\rho_s R U\left(\frac{d_{90}}{d_{30}}\right)^{0.2}S^{1.6}\left(\frac{\theta_c}{\theta}\right) \quad \text{[kg/(s\cdot m)]}
\]

with the critical shields parameter depending on the slope and the angle of repose \(\phi\):

\[
\theta_c = 0.05\left[\cos\left(\arctan(S)\right)\right]\left(1 - \frac{S}{\tan\phi}\right) \quad \text{[-]} \quad (3.40)
\]

Smart and Jaeggi (1983) found that the formula underestimates sediment transport rates for \(0.002 \leq S \leq 0.005\) by a factor of 2 and that the influence of the term \((d_{90}/d_{30})^{0.2}\) is weak. Therefore, they suggested to replace it by 1.05 in case of disregarding.

Rickenmann (1990) experimentally investigated the bedload transport capacity under similar conditions as Smart and Jaeggi (1983) and even enlarged the test condition range. Based on his results including data from Meyer-Peter and Müller (1948) and Smart and Jaeggi (1983) Rickenmann (2001) found the best regression with:

\[
q^*_{cm} = 2.5 \cdot F^{1.1}\sqrt{\theta}\left(\theta - \theta_c\right) \quad \text{[-]} \quad \text{for} \ 0.0004 \leq S \leq 0.20 \quad (3.41)
\]
Einstein (1950) theoretically studied bedload transport and developed an implicitly defined equation based on bedload processes, of which a simplified fit was published by Parker (1979) (P):

\[ q_{\text{em}}^* = 11.2 \left( \frac{\theta - 0.03}{\theta^3} \right)^{4.5} \quad [-] \] (3.42)

Cheng (2002) (C) empirically developed another bedload transport capacity formula based on data sets provided by Gilbert (1914), Meyer-Peter and Mueller (1948) and Wilson (1966):

\[ q_{\text{em}}^* = 13 \theta^{1.5} e^{\left( \frac{-0.05}{\theta^{1.5}} \right)} \quad [-] \] (3.43)

Note that Equation (3.43) does not imply a threshold for initiation of motion and thus applies for \( \theta \geq \theta_c \).

In order to avoid overestimation of bedload transport rates in steep channels, Cheng and Chen (2014) suggested to replace the Shields parameter by the slope-corrected Shields parameter \( \theta_S \):

\[ \theta_S = \frac{\theta}{\cos(\arctan(S)) \cdot \tan \phi - \sin(\arctan(S))} \quad [-] \] (3.44)

where \( \phi = 49 - 62^\circ \) depending on the particle shape and size distribution. Cheng and Chen (2014) did not clearly identify a slope threshold above which, their slope correction should be applied. However, according to Smart (1984) \( S = 0.03 \) is assumed to be a reasonable threshold value.

Rickenmann (2005) developed a formula with a reduced slope \( S' \) to account for energy dissipation induced by large immobile boulders in steep channels with \( S > 0.02 \):

\[ S' = S \left( \frac{0.083}{S^0.35} \left( \frac{h}{d_{90}} \right)^{0.33} \right)^{f_r} \quad [-] \] (3.45)

The macro roughness factor \( f_r \) amounts to \( f_r = 1 \) for moderate roughness.

To obtain the gravimetric \( Q_s \) or volumetric bedload transport \( Q_v \), the specific gravimetric and volumetric bedload transport rates \( q_s \) and \( q_v \), respectively, are generally multiplied by the channel width. In rivers exhibiting several flow paths at low and medium discharges, the effective bedload width does not correspond to the actual channel width (Marti 2006). To account for this phenomenon Marti suggested to replace the channel width \( b \) by the effective bedload channel width \( b' \):
\[ b' = 1.19 \cdot b \cdot \exp \left[ -0.6 \cdot b^{0.65} \cdot d_n^{0.25} \cdot S^{0.3} \cdot g^{0.18} \frac{Q^{0.36}}{Q} \right] \quad [m] \]  
\hspace{2cm} (3.46)\]

Marti (2006) developed this formula based on laboratory experiments and verified it by literature data for \(0.005 \leq S \leq 0.03\).

### 3.2.6 Bedload transport on fixed beds

While the majority of sediment transport research has focused on movable beds, there have been a few studies on sediment transport over fixed beds, as existing in SBTs (Novak and Nalluri 1984). In contrast to movable beds sediment particles are completely exposed to the approach flow over fixed beds. Therefore, most equations for bedload transport over fixed beds include a reduction of the critical Shields parameter for initiation of motion (cf. Chapter 3.2.1) in the range of one order of magnitude compared to alluvial conditions (Novak and Nalluri 1975, Chatanantavet et al. 2013). However, it is not crucial for supercritical flow conditions, where the Shields number is orders of magnitudes higher compared to the critical Shields parameter (cf. Chapter 3.2.1). The following is a summary of the equations applied to determine the bedload transport capacity in SBTs.

Pedroli (1963) (P) conducted laboratory experiments on smooth and fixed bed with \(0.001 \leq S \leq 0.02\), \(d = 1.1 - 11.1\ mm\) and \(h = 0.02 - 0.47\ m\) and found:

\[ q_s^* = 14.5 \cdot \frac{\tau_b^{8/5} \cdot d^{1/5} \cdot g^{3/5}}{\rho_s^{3/5} \cdot \nu^{1/5}} - 23.2 \cdot \rho_s \cdot v \quad [kg/(s\cdot m)] \]  
\hspace{2cm} (3.47)\]

Equation (3.47) was confirmed for bed slopes \(S \leq 0.02\) (Pedroli 1963, Mayerle et al. 1991).

Note that in the original document the exponent for the kinematic viscosity was erroneously given as \(6/5\) instead of \(1/5\) and bed shear stress is defined as \(\tau_b = \rho R_b S\) in the unit [kg/m²] instead of the normally used unit [N/m²].

A further study on bedload transport in a fixed bed channel with sand particles of \(d = 0.5 - 8.7\ mm\) was conducted by Novak and Nalluri (1975) using a flume with \(0.001 \leq S \leq 0.02\) (NN). They proposed:

\[ q_{in}^* = 11.6 \left( \frac{1}{\sqrt{d}} \right)^{-2.04} \quad [-] \]  
\hspace{2cm} (3.48)\]

Mayerle et al. (1991) confirmed this formula for subcritical and slightly supercritical flow conditions with \(F = 0.62 - 1.41\).
Smart and Jaeggi (1983) investigated sediment transport on both movable and fixed beds in laboratory experiments. They found a good correlation for their experimental data by modifying the critical Shields parameter to $\theta_c = 0.005$, the coefficient to 7.35 instead of 4, and by multiplying the non-dimensional shear stress by 1.5 in Equation (3.39):

$$q_s^* = 7.35 \cdot q \frac{\rho_s}{(s-1)} \left( \frac{d_{90}}{d_{30}} \right)^{0.2} S^{1.6} \left( 1 - \frac{\theta_c}{1.5 \theta} \right) \quad \text{[kg/(s·m)]} \quad (3.49)$$

This formula is applicable for fixed bed channels with $0.03 \leq S \leq 0.20$. The same abbreviation (SJ) is used for Smart and Jaeggi’s (1983) equations for both bedload transport on movable and fixed bed since it is basically the same equation.

Auel (2014) (A) investigated sediment transport capacity in SBTs based on laboratory experiments in a fixed and transitionally rough bed flume with $S = 0.01 - 0.04$, $d = 5.4 - 11.2$ mm and $F = 1.7 - 3.6$. He found that (P) and (SJ) underestimated the sediment transport capacities and proposed:

$$q_{\text{in}}^* = 24.0 (\theta - 0.002)^{1.5} \quad [-] \quad (3.50)$$

Additional studies have proposed different sediment transport formulae. However, they are not included herein because they are hardly applicable to engineering purposes either due to their complexity (Hänger and Zeller 1980) or due to the different operating conditions compared to SBTs (Mayerle et al. 1991) resulting in a large overestimation of the bedload transport capacity (Auel 2014).

### 3.2.7 Suspended sediment transport

The suspended sediment mass load $SSL$ is obtained by integrating the transport rate of suspended sediment $SSR$ over time $t$. The $SSR$ results from the integration of the product of the local suspended sediment concentration $ssc$ and the local flow velocity $u$. The $SSR$ at time $t$ follows as:

$$SSR(t) = \int_A ssc(t, y, z) \cdot u(t, y, z) \cdot dA = SSC(t) \cdot Q(t) \quad \text{[kg/s]} \quad (3.51)$$

where $SSC =$ homogeneous suspended sediment concentration, $U =$ cross-sectional averaged streamwise flow velocity, $A =$ cross section area and $Q =$ discharge. It was found that the relation between SSC and $Q$ follows a power law function, although the correlation with solely discharge is rather weak (Spreafico et al. 2005, Haimann et al. 2014).
The \( ssc \) is governed by the flow field strongly fluctuating in time and space. In rivers, the \( ssc \) for clay, silt and sand, i.e. particles of a size of \( d = 0 - 2 \text{ mm} \), increases with depth (Grasso et al. 2014) following the Rouse (1937) profile defined as:

\[
ssc = ssc_a \left( \frac{h - z}{z} \cdot \frac{z_a}{h - z_a} \right)^{\frac{V_z}{\nu \beta \kappa}} \left[ \text{kg/m}^3 \right] \tag{3.52}
\]

where \( ssc_a \) = reference suspended sediment concentration at the reference level \( z = z_a \) and \( \beta \) = sediment diffusion coefficient. Note that \( \beta \approx 1.0, \kappa = 0.4 \) and \( z_a \) is defined as the bedload layer thickness, corresponding to the saltation height.

A rather simple but less accurate approach to determine SSL is given by the ratio of bedload to suspended sediment load. Although this ratio is strongly site-specific and underlies a high variability, it generally lies between 1:0.5 and 1:11 for alpine regions (Sommer 1980, Lenzi and Marchi 2000, Boes 2011). Typical values for Swiss torrents vary from 1:1 to 1:2 (Rickenmann 2001, Turowski et al. 2010).

### 3.3 Hydroabrasion processes

Wear at hydraulic structures can be caused by hydroabrasion, cavitation, abrasion (contact wear without fluids), chemical and thermal impact (Kunterding 1991, Haroske 1998, Jacobs et al. 2001, Vogel 2011). Herein the focus is solely on hydroabrasion since the contribution of the other wear types in particular at SBTs is small.

Hydroabrasion is defined as continuous material loss on the surface of a solid body caused by mechanical stress from the impact of solid particles transported in flowing water (Jacobs et al. 2001). The motion type depends on the flow conditions and causes grinding (sliding motion), rolling (rolling motion) or impinging (saltation motion) impact stresses on the bed (Figure 3.6). Amongst them, saltation is the governing process causing hydroabrasion at hydraulic structures and incision of bedrock rivers (Sklar and Dietrich 2001, 2004, 2006, Lamb et al. 2008, Turowski 2009, Beer et al. 2014, Lamb et al. 2015).

![Figure 3.6 Hydroabrasion processes and typical sediment motion types (adapted from Jacobs et al. (2001))](image)

Figure 3.6 Hydroabrasion processes and typical sediment motion types (adapted from Jacobs et al. (2001))
Hydroabrasion was found to be a self-intensifying process triggered by surface irregularities and areas of structural weaknesses (Johnson and Whipple 2010, Vogel 2011, Auel 2014, Inoue et al. 2014, Jacobs and Hagmann 2015). The material loss grows laterally, vertically and in flow direction and causes irregular bed forms like incision channels or pot holes. The extent and the pattern of hydroabrasion depend on various parameters. The following is a summary of those parameters and describes their interaction with hydroabrasion.

### 3.3.1 Flow conditions

Flow conditions have a major effect on the amount and spatial distribution of the energy transmitted to the bed and hence on hydroabrasion. The abrasion rate increases with increasing energy transmitted to the bed. Particle impact in a viscous fluid can be completely damped as the kinetic energy of the moving particle is dissipated (Scheingross et al. 2014). However, this effect was found to be negligible for supercritical flows with bedload transport (Beer and Turowski 2015, Auel et al. 2017b).

The flow velocity is one important parameter concerning hydroabrasion. On the one hand, the vertical impact velocity, the sediment transport capacity, the size of mobilized particle and the abrasion potential increase with increasing flow velocity (Meyer-Peter and Müller 1948, Bania 1989, Momber and Kovacevic 1994, Haroske 1998, Horszczaruk and Kiernozycki 2000, Hu et al. 2002, 2006, Mechtcherine et al. 2012, Auel 2014). On the other hand, the saltation hop length grows with increasing flow velocity and shifts the transport mode to suspension resulting in a reduction of transmitted energy per unit area. These two effects compensate each other at a certain threshold, beyond which the specific impact energy decreases with increasing flow velocity (Auel 2014).

Moreover, the energy transmitted from the particle to the bed per unit area is not constant across the tunnel width and is affected by secondary currents. In particular for low aspect ratios, i.e. $b/h < 4 - 5$, secondary currents cause uneven bed shear stress distribution in the spanwise direction, which leads to bedload transport concentrations and higher abrasion rates (Nezu and Nakagawa 1984, McLelland et al. 1999, Albayrak and Lemmin 2011, Auel et al. 2014b). As a result, lateral incision channels occur and reveal a self-intensifying abrasion characteristic since they stabilize in return the secondary currents and amplify the sediment transport concentration (Auel 2014).
3.3.2 Sediment transport rate and properties

Theoretically, abrasion depth linearly increases with the mass of transported sediment (Helbig and Horlacher 2007, IEC 2013). This has been confirmed by several laboratory and field investigations (Bovet 1958, Horszczaruk 2009, Wu et al. 2010, Mechtcherine et al. 2012, Auel 2014, Wang et al. 2014, Winkler 2014, Nakajima et al. 2015). Consequently, abrasion rate increases with specific concentration and sediment transport rate (Bajracharya et al. 2008). However, abrasion rates peak at an intermediate level of sediment transport rate due to the burial of the invert by transient sediment deposits reducing the hydroabrasive impact (Sklar and Dietrich 1998, Sklar and Dietrich 2004, Turowski 2009, Sklar and Dietrich 2012). This effect is called cover effect. Sklar and Dietrich (2001, 2004) stated that there is a critical sediment supply rate, providing tools for abrasion while the bedrock is minimally protected by a cover.

Hydroabrasion also depends on the abrasive capacity of the supplied sediments, which is driven by (1) particle size, (2) shape and (3) hardness / mineralogy (Bogen 1989, Bajracharya et al. 2008, Winkler et al. 2010). Larger particles lead to higher abrasion rates (Bitter 1963a, b, Vogel and Müller 2009, Okita et al. 2012) due to the higher impact energy. According to Turowski et al. (2015) more than 40% of the energy transferred to the surface of a Swiss river was delivered by the biggest grain size class with \( d = 86 \) mm amounting to less than 10% of the total sediment volume. However, there is a certain particle size, i.e. not necessarily the largest particles, transmitting the highest energy to the invert (Sklar and Dietrich 2001, Auel 2014). A further increase of grain size results in a change of transport mode and finally reaches the threshold for particle motion reducing the abrasion rate to zero.

Furthermore, the local stress at an impact spot increases with decreasing contact area according to the principle of the Hertzian pressure (Hertz 1896, Popov 2010). Hence, angular shaped particles cause higher specific abrasion rates than rounded particles (Bovet 1958). The Hertzian pressure further increases with increasing relative hardness from erosive material to eroded material (Wellinger and Uetz 1955, Uetz 1986). The sediment particles are deformed or break during the impact if the sediment particle hardness is lower than the bed material hardness, whereas harder particles penetrate the material surface causing wear. For instance, Sklar and Dietrich (2001) revealed an increase of the abrasion rate of limestone by a factor of 2 by using quartzite instead of limestone gravel as erodent. It is noted that the sediment particle hardness is related to the content of hard minerals, e.g. quartz (Hertz 1896, Bajracharya et al. 2008, Popov 2010).
3.3.3 Particle impact angle

Another decisive parameter on hydroabrasion is the particle impact angle, since it affects the vertical particle impact velocity as well as the wear type. Depending on the particle impact angle, two types of hydroabrasive wear occur: (I) “cutting wear” associated with the parallel component of the kinetic energy of the particle and (II) “deformation wear” associated with the normal component of the kinetic energy of the particle (Bitter 1963a, b). Both processes occur simultaneously, while the impact angle determines their portion. Depending on the dominating harming process and impact angle, different invert material properties are controlling the abrasion resistance (Hu et al. 2004, Okita et al. 2012).

Figure 3.7a shows the relative wear of different materials as a function of the particle impact angle. Cast basalt represents brittle materials such as natural rock or concrete, and performs better under flat than steep impact angles, whereas rubber, representing ductile materials, persist better under steep than flat impact angles (Wellinger and Uetz 1955, Head and Harr 1970, Kunterding 1991). It is noted that concrete is considered a quasi-brittle material due to its low tensile strength and hence is expected to perform best at relatively flat impact angle. This is in line with the experimental results of Kunterding (1991) presented in Figure 3.7b. In his experiment concrete performed best at the lowest tested impact angle of 15°, and the highest abrasion rates were observed at an impact angle of 75°.

Figure 3.7 a) Relative abrasion rate of (1) cast basalt, (2) rubber and (3) different iron and steel types normalized by steel abrasion rates depending on impact angle (Wellinger and Uetz 1955), and b) relative abrasion rates of concrete depending on impact angle (Kunterding 1991)
3.3.4 Invert material properties

Since the most common material at hydraulic structures and particularly at SBTs is concrete, this sub-section focuses on hydroabrasion of concrete but the findings are also applicable to other quasi-brittle materials such as natural stone or cast basalt since they exhibit similar linear-elastic material properties (Heckel 1970, Rossmanith 2013).

Many studies using different setups have been conducted mainly in the laboratory to investigate the effect of various material parameters on the abrasion resistance of cementitious invert materials. A systematic investigation covering a variety of material properties and test conditions was performed by Kunterding (1991). He found that the abrasion resistance increased with (1) increasing grading factor, strength and angularity of the aggregate, which result in a force-locked structure enabling a proper force transmission, (2) decreasing water-cement ratio causing higher material density and low porosity and (3) increasing curing duration promoting a proper hydration and thus a strengthening of the cement matrix. These findings were confirmed by several other laboratory investigations (Bania 1989, Haroske 1998, Liu et al. 2006, Mechtcherine et al. 2012). There is a consensus that abrasion resistance is promoted by high tensile strength, compressive strength, elasticity and fracture energy (Bania 1989, Kunterding 1991, Haroske 1998, Jacobs et al. 2001, Sklar and Dietrich 2001, Li et al. 2006, Liu et al. 2006, Dai et al. 2012, Momber 2014). The latter is defined by the integral of the stress-deformation curve and represents the energy required to produce a crack of unit length and is given in [N/m].

In other civil engineering disciplines, fibers are admixed to concrete in order to raise the tensile strength, reduce the crack formation and hence to increase the performance of the concrete. However, for hydroabrasion purposes their effectiveness is controversial. Hu et al. (2002) found that steel fiber admixtures increase the abrasion resistance of concrete. This finding was confirmed by Wu et al. (2010) and Kryžanowski et al. (2012). On the contrary, Jacobs and Hagmann (2015) observed no significant effect of steel fiber admixtures on the abrasion resistance of concrete.

A significant enhancement of the abrasion resistance was achieved by using additions such as polypropylene, glass or carbon fibers and nanoparticles (Li et al. 2006, Liu et al. 2006, Wu et al. 2010). However, Helbig et al. (2005) found that textile reinforcements do not affect the abrasion resistance of concrete.

A further novel approach to increase the elasticity of concrete is the addition of rubber aggregate. Such concrete was tested at a stilling basin in a torrential river and showed a
considerable higher abrasion resistance compared to concrete without rubber (Kryžanowski et al. 2012).

Applying coatings is a further approach to improve the material persistence. Wang et al. (2014) investigated the behavior of flexible protective coating materials consisting of polyuria elastomers using an underwater abrasion test. Concrete specimens with coatings exhibited a 20 times slower abrasion rate compared to the specimens without coatings and the abrasion decreased with increasing coating hardness. However, after a certain point, increase of hardness leads to a loss of elasticity, and hence to a loss of abrasion resistance.

3.4 Hydroabrasion testing

Hydroabrasion strongly depends on site-specific parameters. Realistic simulation of field conditions in the laboratory is difficult, resulting in a large variety of test setups (Jacobs et al. 2001, Vogel 2011). Although all below-listed methods are capable to simulate hydroabrasion and to determine the abrasion resistance of different materials, upscaling of laboratory results to field application is questionable and still a topic of ongoing research. For a correct interpretation of the laboratory results knowledge on the test procedures and conditions is required. Therefore they are presented hereafter.

3.4.1 Underwater bedload tests

Hydroabrasion research, particularly laboratory simulation of hydroabrasion and wear prediction models, has been advanced in Germany during the last three decades. Two different test setups have been established: (I) the paddle technology invented by Bania (Bania 1989, Haroske 1998, Vogel and Müller 2009, Vogel 2011) and (II) the plate technology herein called the Dresdner Drum and shown in Figure 3.8a (Bellmann et al. 2012, Helbig et al. 2012, Mechtcherine et al. 2012). The specimens are exposed to stresses due to grinding, rolling and saltating impact stresses. By varying the rotation velocity (of the drum or paddles), particle parameters (size, shape, hardness), water-solid-ratio and amount of abrasive charge, different operating regimes can be simulated. Usually, 20 kg of an abrasive charge exhibiting a water-eroding material ratio of 1:1 is used for the Dresdner Drum. The eroding material consists of steel spheres with $d = 3$ to 32 mm.

The American Society for Testing Materials (ASTM) developed a standard underwater test method for abrasion resistance of concrete (ASTM C1138), which is an internationally accepted
method (Horszczaruk 2005, Horszczaruk 2009, Wu et al. 2010, Kryžanowski et al. 2012). The device consists of a steel cylinder containing water and eroding material (steel balls) swirled by an agitation paddle. Sklar and Dietrich (2001) designed an abrasion mill based on the ASTM C1138 standard (Figure 3.8b). In contrast to the ASTM standard test, the abrasion mill is able to simulate not only sliding and rolling, but also particle saltation. As a result, this device is expected to be more appropriate to simulate hydroabrasion caused by grinding and impinging stresses and therefore has been used in the most recent hydroabrasion studies (Sklar and Dietrich 2001, Scheingross et al. 2014, Small et al. 2015).

![Figure 3.8](image)

Figure 3.8  a) Dresdner drum with (1) abrasive charge and (2) samples (adopted from Mechtcherine et al. 2012), and b) abrasion mill with (1) motor, (2) paddle, (3) abrasive material and (4) sample (adopted from Scheingross et al. 2014)

3.4.2 Grinding abrasion tests

The Böhme disk is an easy, user-friendly, fast and therefore often applied method to determine abrasion resistance of lining materials (Kunterding 1991, Jacobs et al. 2001, Horszczaruk 2005, Kryžanowski et al. 2012). The device and its test procedure are described in detail in the standard DIN 52108. A sample is mounted horizontally in the device and abraded by an abrasive material strewn on a rotating steel disk. There are other tests causing similar grinding stresses to the sample: the ASTM C779 standard test, the ASTM C944 standard test and the sand-rubber-wheel test (Woldman et al. 2012). Although these tests determine the relative abrasion resistance due to grinding stresses, they are not able to reproduce the harming process due to impinging particles.

3.4.3 Scaled physical model tests

Hydroabrasion resistance of invert materials can be investigated in scaled physical models. To date, they are commonly applied to investigate bedrock erosion processes using fast eroding substitute materials to accelerate the processes (Finnegan et al. 2007, Johnson and Whipple...
Auel (2014) investigated the correlation between flow field, particle motion and abrasion in a scaled SBT model under various hydraulic and sediment supply conditions using the flume shown in Figure 3.9. Based on the experimental results and literature data Auel et al. (2017b) modified the abrasion model developed by Sklar and Dietrich (2004) with regard to supercritical flows in SBTs. However, in-situ investigations are required to calibrate the model for field application (Auel et al. 2016a, Auel et al. 2017a, b).

![Figure 3.9 Hydroabrasion test flume with UDS = ultra-sonic distance sensor and DLD = distance laser device, from Auel (2014)](image)

### 3.4.4 Hydroabrasion in-situ tests

Hydroabrasion affects many types of hydraulic structures. However, prototype experiments are rare and the knowledge gained from prototype experiments is largely outside the public domain (Trucco 1981, Delley 1988, Vischer et al. 1997). Two in-situ investigations were reported by Jacobs et al. (2001) and Kryžanowski et al. (2012). They investigated hydroabrasion of concretes under supercritical flow conditions with both bedload and suspended sediment load taking place in-situ under real operating conditions. The test site used by Kryžanowski et al. (2012) was in a stilling basin located at the lower Sava River in the southeast of Slovenia. The test site used by Jacobs et al. (2001) was the Runcahez SBT located in the Swiss Alps. Both investigations analyzed the abrasion resistance of different concretes in-situ and assessed different laboratory tests regarding upscaling of the laboratory results to field applications. Kryžanowski et al. (2012) revealed positive correlations between the abrasion depths in-situ and both the ASTM C1138 test and the Böhme test (Chapter 3.4.1 and 3.4.2). This indicates that the dominant harming process at the prototype stilling basin is grinding rather than...
impinging. In contrast, Jacobs et al. (2001) observed a positive correlation only between the Bania underwater bedload test and the in-situ measurements, whereas the results obtained from the Böhme test showed no correlation with the field observations. This is attributed to the fact that the Böhme test only simulates grinding but not impinging stresses, which play the major role concerning hydroabrasive wear in SBTs.

### 3.4.5 Discussion

Hydroabrasion is governed by several parameters and includes different mechanisms. Depending on invert material type, i.e. ductile or brittle material, flow field and sediment properties either deformation or cutting wear or a combination of both occurs. Due to the large bandwidth of hydraulic and sediment transport conditions under which hydroabrasion occurs, no test setup is able to completely simulate all of the factors affecting hydroabrasion. Different tests are available, but only for determining relative abrasion resistance under laboratory conditions. Particle velocity, particle impact angle and hence material-particle interaction analyzed in laboratory settings may strongly deviate from the field. There is no perfect similarity of laboratory and field data to date. Therefore, upscaling from the laboratory to field scale remains questionable.

With respect to the operating range of SBTs and other hydraulic structures, the author is aware of only two investigations dealing with the upscaling of laboratory abrasion data to field applications (Jacobs et al. 2001, Kryžanowski et al. 2012). Their findings indicate that the Böhme test may lead to appropriate results for applications with mainly grinding stresses. To simulate hydroabrasion in supercritical flows in SBTs governed by bedload, the underwater bedload drum test accounting for the particle transport mode produced reasonable results (Jacobs et al. 2001). However, there is still a lack of knowledge and the laboratory tests conditions still deviate from the field. Therefore, further in-situ investigations are required to properly determine the abrasion resistance of invert materials and gain information about upscaling from laboratory to field scale.

### 3.5 Hydroabrasion prediction

Several mechanistic models exist for abrasion prediction. The models take into account the physical process of particle impact and are derived from experimental research on particle motion. There are two types of models: (I) the bedrock incision models focusing on sub- to low supercritical flow regimes typically occurring in river systems (Sklar and Dietrich 2004, Lamb
et al. 2008, Chatantavet and Parker 2009, Beer and Turowski 2015) and (II) models for hydroabrasion of concrete mainly accounting for supercritical flow conditions (Ishibashi 1983, Huang and Yuan 2006, Helbig and Horlacher 2007, Helbig et al. 2012, Auel 2014). However, since both bedrock and concrete are brittle materials subjected to identical hydroabrasion processes, the models are applicable for both hydroabrasion at concrete structures and bedrock incision in rivers.

This chapter gives an overview of the applied abrasion models and the state-of-the-art of numerical modeling. A further abrasion model, subjected to larger prediction error due to various uncertain input parameters is documented in Appendix D. All models, except the numerical one, are one-dimensional models, simulating hydroabrasion of a unit area but do not account for the changing bed roughness due to abrasion and spatial variations in bed shear stress, particle motion and invert material strength. This must be respected for the abrasion estimation and invert material design.

3.5.1 Saltation abrasion model of Sklar and Dietrich (SAM)

Sklar and Dietrich (2004) investigated bedrock abrasion caused by bedload particles in saltation motion by using an abrasion mill (Chapter 3.4.1). They developed a physically-based model for bedrock incision depending on bed shear stress, bedrock erodibility, particle size, particle impact energy and sediment supply (Sklar and Dietrich 2004, 2012). The basic form of the so-called saltation-abrasion model (SAM) is:

$$A_r = \frac{Y_M}{k_v f_{st}^{2}} \frac{W_{in}^{2}}{L_p} q_v \left(1 - \frac{q_s}{q_v}\right) \quad [\text{m/s}] \quad (3.53)$$

where $A_r =$ vertical abrasion rate, $Y_M =$ Young’s modulus, $k_v =$ dimensionless resistance coefficient and $f_{st} =$ splitting tensile strength. Based on the findings of Clark (1966), Sklar and Dietrich (2004) stated that the variation in $Y_M$ of rocks is limited and hence they treated it as a constant, i.e. $Y_M = 50 \text{ GPa}$. This is actually a rough simplification and might be questioned regarding the large range of $Y_M$ at least for soft rocks and concretes. The term $f_{st}^{2}/(2Y_M)$ is called “elastic strain energy density” and represents the fracture energy required to detach a unit volume. $k_v$ is related to the efficiency of energy transfer from impinging particles to the invert material. Sklar and Dietrich (2004, 2012) proposed the widely accepted value of $k_v = 10^6$ for rock, which is in the same order as for concrete (Auel et al. 2017b). The second term denotes the flux of kinetic impact energy per unit area and time and the last term is related to the cover effect that considers the fraction of exposed bedrock. Expressing the vertical particle impact
velocity as $W_{im} = 3V_pH_p/L_p$ and applying their particle motion equations described in Chapter 3.2.4, Sklar and Dietrich (2004) proposed to rewrite Equation (3.53) as follows:

$$A_s = 0.08g(s-1)\frac{Y_m}{k_{st}}\cdot q_s \cdot \left(1 - \frac{q_s}{q'}\right) \left(\frac{\theta}{\theta_c} - 1\right)^{-0.5} \left(1 - \left(\frac{U_s}{V_s}\right)^2\right)^{1.5} \text{[m/s]} \quad (3.54)$$

The threshold of particle motion is accounted by $(\theta/\theta_c - 1)^{-0.5}$ and the last term is related to the mode shift from saltation to suspension. In case of hydroabrasion dominated by saltation, this term can be neglected.

Sklar and Dietrich (2001) conducted abrasion mill experiments (described in Chapter 3.4.1) to obtain gravimetric abrasion rates $A_{rs}$ of various rocks and artificial mortars. Figure 3.10 presents the obtained abrasion rates as a function of the splitting tensile strength. To measure the influence of splitting tensile strength on bedrock abrasion rates, the bed rock lithology was varied while holding constant the hydraulic conditions as well as the number, the size and the lithology of the abrasive charge (150 g and $d = 6$ mm). The abrasion rate decreases with increasing $f_{st}$ as seen in Figure 3.10a and follows

$$A_{rs} \sim f_{st}^{-2(10.1)} \quad \text{[g/h]} \quad (3.55)$$

This result was verified by experiments with different abrasive charge, i.e. one particle of 70 g and $d = 30$ mm from different lithologies. Various rock samples were abraded by a particle composed of both the same lithology (relative hardness = 1) and quartzite (relative hardness > 1), respectively, the latter being a particular strong material with Mohs hardness of 7. The results are shown in Figure 3.10b. Although the samples were approx. two times more abraded by the former than by the latter, the exponent in the power law relationship between abrasion rate and splitting tensile strength did not change. This finding confirms the SAM stating that abrasion rate scales inversely with the square of splitting tensile strength. However, the SAM does not account for the significant effect of the relative particle hardness on the abrasion rate. This might result in considerably over- or underestimation for low or high relative particle hardness, respectively.

The abrasion coefficient $k_v$ is crucial for abrasion estimation and is still a challenging issue of ongoing research. Sklar and Dietrich (2004, 2012) called it “dimensionless resistance coefficient” as well as “dimensionless proportionality constant”. However, treating $k_v$ as a constant was questioned by different studies (Beyeler and Sklar 2010, Turowski et al. 2013, Momber 2014, Scheingross et al. 2014, Lamb et al. 2015, Auel et al. 2017b), so that the term
“abrasion coefficient” is used herein. Sklar and Dietrich (2004, 2012) determined $k_v$ for various materials and proposed the widely accepted value of $k_v = 10^6$ for rock, despite large variations of $k_v$ from $1 \cdot 10^6$ to $9 \cdot 10^6$. This variation is attributed to the fact that $k_v$ depends not only on the properties of the impinging sediment such as angularity and hardness, but also on the invert material (Whipple and Tucker 1999, Beer and Turowski 2015, Small et al. 2015). Momber (2014) confirmed this hypothesis based on his laboratory study using different rocks and cementitious composites. He stated that $k_v$ is not a constant but depends on elastic strain energy density, particularly on $Y_M$ and on the material response.

Figure 3.10 Gravimetric abrasion rate as a function of splitting tensile strength from abrasion mill experiments for different rocks and artificial mortar mixtures with various water-cement ratios for a) sediment load 150 g, $d = 6$ mm, and b) one sediment particle of 70 g and $d = 30$ mm made of quartzite (solid symbols) and of the same rock as the abraded sample (open symbols); adapted from Sklar and Dietrich (2001)

Scheingross et al. (2014) replicated the abrasion mill setup of Sklar and Dietrich (2001, 2004) using polyurethane foams with tensile strengths of $f_{st} = 0.32 - 16.6$ MPa as a substitution material for rock. They converted the abrasion rates of the foams to limestone equivalents by applying the following tensile-strength scaling relationship (Equation (3.56)) proposed by Sklar and Dietrich (2004), provided $Y_M$ and $k_v$ are constant:

$$A^* = A_r \left( \frac{f_{st}}{f_{st}^*} \right)^2 \quad [m/s]$$  \hspace{1cm} (3.56)

$A^*$ and $f_{st}^*$ denote the abrasion rate and splitting tensile strength of the equivalent material, respectively. The resulting limestone-equivalents are in good agreement with Sklar and Dietrich’s (2001) limestone data, allowing for direct comparison of the results between these two materials. However, these results indicate that either $k_v$ must vary in portion to $Y_M$, or that $Y_M$ has little influence on the abrasion rate.
Theoretical background

Auel et al. (2017b) recently discussed the application of constant \( k_v \) to the abrasion models for both concrete and natural rock by analyzing data sets provided by Sklar and Dietrich (2004), Johnson and Whipple (2010), Inoue et al. (2014) and Auel et al. (2017b). They revealed that \( k_v \) increases with splitting tensile strength and tends to stabilize for hard materials with \( f_{st} > 1 \) MPa. This indicates that tensile-strength scaling according to Equation (3.56) is misleading in particular for materials with low splitting tensile strengths. Lamb et al. (2015) confirmed these findings based on the analysis of data sets provided by Sklar and Dietrich (2001), Beyeler and Sklar (2010) and Scheingross et al. (2014) and concluded that abrasion scaling is not improved by including \( Y_M \) as an independent factor. They proposed an extended scaling using the dimensional coefficient \( k_R \), which depends on Young’s modulus, material density, porosity and crystal and clast size of the eroded material without giving further details (parameters of the equivalent material denoted by the star):

\[
A_v = A^* \frac{k_R}{k_v} \left( \frac{f_{st}^*}{f_{st}} \right)^2 \quad [\text{m/s}]
\]  

(3.57)

The two coefficients, \( k_v \) and \( k_R \), are physically related by:

\[
k_R = \frac{Y_M}{k_v} \quad [\text{Pa}]
\]  

(3.58)

Beyeler and Sklar (2010) investigated various materials, including eroded disks from Sklar and Dietrich (2001), by means of a scanning electron microscope and revealed a significant effect of several material properties on the abrasion behavior. The abrasion resistance reduces with increasing porosity and increasing clast size for a given tensile strength. Beyeler and Sklar (2010) hypothesized that using tensile strength tests to determine the abrasion resistance of a material misleads due to a mismatch in the spatial scale of the fracture processes. Quasi-static tensile strength tests generally produce fractures along grain boundaries, whereas hydroabrasive wear by saltating particles typically produces silt-sized fragments suggesting erosion at the sub-grain scale in most crystalline and clastic rocks. As a result, abrasion scaling should include porosity, clast size and tensile strength instead of solely tensile strength.

To conclude, \( k_v \) was found to depend on various properties of the sediment and the invert material. Although some of these parameters have been identified, their effects on \( k_v \) have not been systematically investigated yet and require additional investigations for an accurate determination.
3.5.2 Saltation abrasion model adapted by Auel (SAMA)

Auel (2014) conducted experiments in a scaled SBT model and enlarged the existing database particularly for highly supercritical flows. Moreover, Auel’s (2014) investigation encompassed direct particle impact measurements with a high-speed camera instead of using particle impact characteristics. Based on his results Auel et al. (2017b) adapted the SAM and proposed the Saltation Abrasion Model adapted by Auel (SAMA):

\[
A_s = \frac{Y_m}{k_v f_{st}^2} \left( \frac{s-1}{230} \right) g \left( 1 - \frac{q_s}{q_s^*} \right) \quad \text{[m/s]} \quad (3.59)
\]

The last term in Equation (3.53) denotes the cover effect and can be neglected if sediment transport rate is orders of magnitudes smaller than sediment transport capacity, i.e. \( q_s / q_s^* \ll 1 \). In contrast to the SAM, this model does not account for the mode shift from saltation to suspension, since saltation is the governing process causing hydroabrasion in SBTs.

Auel et al. (2017b) determined \( k_v \) for a range of materials eroded in the laboratory as well as in-situ. The data base encompasses Auel’s (2014) laboratory results, as well as field data from the Asahi SBT in Japan and data sets provided by Sklar and Dietrich (2004), Johnson and Whipple (2010) and Inoue et al. (2014). They found that \( k_v \) increases with invert material tensile strength and tends to stabilize at \( k_v \approx 10^5 \) for hard materials such as rock and concrete with \( f_{st} > 1 \) MPa. These values are an order of magnitude lower compared to the values proposed by Sklar and Dietrich (2004) due to both newly determined formulas for the particle motion trajectories and additional data from soft bedrock and mortars. However, further experiments including both laboratory and in-situ tests on various invert materials are needed to clarify the relation between \( k_v \) and the invert material properties allowing upscaling from laboratory to field scale (Auel et al. 2015a, Auel et al. 2017b).

3.5.3 Ishibashi model

The abrasion model of Ishibashi (1983) is most likely the first published mechanistic abrasion prediction model for concrete and steel on hydraulic structures exposed to hydroabrasive impact. It was developed based on laboratory flume experiments with supercritical flow conditions. However, due to the fact that it was written in Japanese the model was not known in the Western part of the world until Auel et al. (2016b) provide a short description. The model accounts for both deformation and cutting wear and is widely applied in Japan (Auel et al. 2016a). The volumetric material loss is defined as:
where \( E_k \) = total deformation work by saltating particles:

\[
E_k = 1.5 \cdot V_{ts} \sum E_i \cdot N_i \cdot n_i \quad \text{[kg\cdotm]} \quad (3.61)
\]

and \( W_f = \) total friction work by grinding particles:

\[
W_f = 1.654 \cdot V_{ts} \sum \gamma_i E_i \cdot N_i \cdot n_i \quad \text{[kg\cdotm]} \quad (3.62)
\]

c1 and c2 are material property constants, taken as c1 = 1.189 \( \cdot \) 10^{-7} [m^2/kg] and c2 = 1.135 \( \cdot \) 10^{-8} [m^2/kg], and c1 = (1.18 - 3.73) \( \cdot \) 10^{-11} [m^2/kg] and c2 = (1.33 - 6.59) \( \cdot \) 10^{-11} [m^2/kg] for concrete and steel, respectively. Furthermore, \( V_{ts} = \) volume of transported sediment, \( E_i = \) kinetic energy of a single particle, \( \gamma_i = \) particle impact angle, \( N_i = L/L_p = \) number of impacts along the total invert length \( L \) with the particle hop length:

\[
L_p = 100d(\theta - \theta_i)^{2/3} \quad \text{[m]} \quad (3.63)
\]

and \( n_i = \) particle number per unit volume:

\[
n_i = \frac{3.6}{\pi d^3} \quad \text{[1/m^3]} \quad (3.64)
\]

The kinetic energy of a particle is determined from Ishibashi and Isobe (1968):

\[
E_i = \frac{F_i^{5/3}}{2.5 \cdot n_i^{2/3} \cdot (d / 2)^{1/3}} \quad \text{[kg\cdotm]} \quad (3.65)
\]

where the auxiliary parameter \( n_i \) accounts for the ratio of the Young’s moduli \( Y_M \) to the Poisson’s ratio \( \nu \) of both the invert (index 1) and the sediment (index 2) material:

\[
n_i = \frac{4}{3 \left[ \left( \frac{1 - \mu_1^2}{Y_{M1}} \right) + \left( \frac{1 - \mu_2^2}{Y_{M2}} \right) \right]} \quad \text{[kg/m^2]} \quad (3.66)
\]

Provided no information is available, the Poisson’s ratio can be taken to \( \mu = 0.3, \mu = 0.2 \) and \( \mu = 0.265 \) for steel, concrete and rock, respectively (Christensen 1996, SIA 2003b, a). Alternatively \( n_i = 2.41 \cdot 10^9 \) kg/m^2 proposed by Ishibashi (1983) for sediment gravel transported over concrete can be taken. The impact force \( F_i \) and the impact angle are defined as:
By applying these formulae to the abrasion data of the concrete lined Asahi SBT gained over 17 years, Auel et al. (2016a) recognized a considerable prediction error. However, cutting wear has a minor effect on brittle materials (Head and Harr 1970, Sklar and Dietrich 2004) and the kinetic energy of the rolling or sliding particles is an order of magnitude smaller than in saltation mode (Auel 2014). Therefore, the abrasion rate obtained from the cutting term exceeds the one of the deformation term and the abrasion rate is considerably overestimated as a result. Hence, Auel et al. (2016a) proposed to skip the cutting wear term for abrasion estimation of brittle materials such as rock, cast basalt and concrete.

This model is also flawed by the fact that the lining material properties and the abrasiveness of the sediment are not or insufficiently accounted for, and therefore include a certain model uncertainty (Chapter 3.3.3).

### 3.5.4 Numerical models

The progress in computing performance in the recent years has enabled the analysis of not only the flow field and particle motion but also the spatial distribution of hydroabrasion by implementing mechanistic and semi-empirical abrasion equations in numerical models. This approach has been applied for abrasion estimations and design optimization of turbines as well as for the investigation of bed variations in open channel flows (Felix 2017, Fukuda and Fukuoka 2017).

One of the most recent analytical approaches was introduced by Fukuda and Fukuoka (2017), who developed a new sub-particle scale discrete element model for abrasion investigations. The flow field was simulated in an Eulerian computation model for solid-liquid multiphase flow. For the particle motion a Lagrangian method was used. To represent the effects of particle size and shape, particles were made by superimposing several small spheres to build variously sized and shaped gravels (Figure 3.11a). The response of the invert material to the abrasive impact was calculated by a modified discrete element method (MDEM) capable to simulate concrete behavior by introducing tensile forces. The model involves the abrasion equation of Ishibashi (1983), while the cutting wear term was neglected. The MDEM estimates the fracture rates and hence the abrasion rates based on the mechanical energy loss, of the moving particles during
collision with the concrete surface causing a strength degradation of the invert material. The abrasion processes, typically occurring in the sub-particle scale, were simulated by modelling concrete with small spheres and using natural tensile strength of the concrete (Figure 3.11b).

Fukuda and Fukuoka (2017) applied this comprehensive numerical abrasion model to invert abrasion experiments conducted in a 48 m long and 1 m wide concrete lined flume and found that this approach appropriately simulates the real-scale particle motion, flow field and hence abrasion rates. However, only the first 33 hours of the experiments could be simulated due to large computational load. A validation was accordingly only done for the early stage of abrasion. Although this result is promising, the large computational power restricts the application to small temporal and spatial applications. Further investigations and prototype calibration tests are required to enlarge the temporal and spatial application range and to enhance the prediction model’s accuracy.
4 Setsups and methods

The general methodology and test setups are presented in this chapter. First, the acquisition of fundamental data is treated. Second, detailed information on the test setups and measuring devices used in the prototype SBTs Solis, Pfaffensprung and Runcahez are given. Third, the laboratory and field test setups for calibration of the bedload transport measuring system (so-called Swiss Plate Geophone System) are reported, followed by the description of laboratory test setup for abrasion measurement of a range of invert materials. Finally, the procedure to calibrate the abrasion model is explained.

4.1 Acquisition of fundamental data

Hydroabrasion depends on the hydraulic and sediment transport conditions, as well as on the invert material and sediment properties. The acquisition of those data is treated in this section. In general, the procedure explained hereafter is applied. If different procedures were used, they were introduced in the subsection of the corresponding test site.

4.1.1 River topography

Regarding hydroabrasion, the bedload transport volume, which depends on river topography, grain size distribution and hydrology, is a decisive input parameter. Therefore, the river topography was determined for the investigated river reaches measuring 1 to 3 km in length, upstream of the SBT inlets. The river slopes were determined based on digital maps using a profile-tool (www.map.geo.admin.ch) following the procedure recommended by the Federal Office for the Environment FOEN (Hunziker et al. 2013). The same tool was applied to determine the representative flume cross section, based on the cross sections generated every 100 to 300 m.

4.1.2 Hydraulics

Generally, the discharge is directly measured on site. The flow depths in the rivers were calculated based on river topography and hydrographs, assuming uniform flow, while the flow depths in the SBTs were determined by applying backwater calculations to the hydrographs.
4.1.3 Bedload transport in the river

Bedload transport in the rivers was calculated based on river topography and hydrographs using the following equations applicable to gravel bed rivers: Meyer-Peter and Müller’s formula (Equation (3.36)), Wong and Parker’s formula (enhanced version of Equation (3.36)), Smart and Jaeggi’s formula (Equation (3.39)), Rickenmann’s formula (Equation (3.41)), Parker’s formula (Equation (3.42)), Cheng’s formula (Equation (3.43)).

Different correction factors were also included. In order to avoid overestimation of bedload transport rates the critical Shields parameter was calculated according to Equation (3.40). The existence of an armor layer was accounted for by adapting the critical Shields parameter according to Equation (3.19). To take into account the energy dissipation in steep channels, either the slope-corrected Shields parameter (Equation (3.44)) or the adapted slope $S'$ (Equation (3.45)) were used. These factors were not applied to (MPM) and (WP) since they had led to unrealistically high discharge for initiation of motion. The bedload transport effective channel width is generally not equal to the channel width. Therefore, Equation (3.46) was used to determine the bedload transport effective channel width. The grain size coefficient $(d_{90}/d_{30})$, included in (SJ)'s equation, was set to 1.05 according to Smart and Jaeggi (1983). The applied volumetric bedload transport capacities are as follows:

$$Q_v^* = 8\sqrt{g} \left(\frac{k_{SR,S}}{k_{SR,r}}\right)^{1.5} R_s S - \theta_c (s-1) d \cdot b' \quad [\text{m}^3/\text{s}] \quad \text{(MPM)} \quad (4.1)$$

$$Q_v^* = 3.97\sqrt{g} \left(\frac{k_{SR,S}}{k_{SR,r}}\right)^{1.5} R_s S - \theta_c (s-1) d \cdot b' \quad [\text{m}^3/\text{s}] \quad \text{(WP)} \quad (4.2)$$

$$Q_v^* = 4 \frac{RU}{(s-1)} 1.05 \cdot (S')^{1.6} \left(1 - \frac{\theta}{\theta_c}\right) b' \quad [\text{m}^3/\text{s}] \quad \text{(SJ)} \quad (4.3)$$

$$Q_v^* = 2.5 \cdot Fr^{1.1} \sqrt{\theta} (\theta - \theta_c) \sqrt{(s-1)gd^2} \cdot b' \quad [\text{m}^3/\text{s}] \quad \text{with } \theta, Fr = f(S') \quad \text{(R)} \quad (4.4)$$

$$Q_v^* = 13\theta_s^{0.5} e^{\theta_s^{0.5}} \sqrt{(s-1)gd^2} \cdot b' \quad [\text{m}^3/\text{s}] \quad \text{(C)} \quad (4.5)$$

$$Q_v^* = 11.2 \left(\frac{\theta_s - 0.03}{\theta_s^3}\right)^{1.5} \sqrt{(s-1)gd^2} \cdot b' \quad [\text{m}^3/\text{s}] \quad \text{(P)} \quad (4.6)$$

Note that the effective sediment transport rate depends also on the sediment supply and can be considerably smaller than the theoretical transport capacity. At the Asahi SBT detailed sediment
studies were conducted and estimates were compared with both reservoir sedimentation data and bed elevation survey. The results showed that the one-dimensional analysis of sediment transport capacity overestimated the effective sediment transport by a factor of 2 (Auel et al. 2016b). Therefore, whenever possible, the results of the bedload estimates were checked by data sets of reservoir sedimentation and river bed surveys in the present study.

A further issue is the effect of an armor layer. The presence of an armor layer was investigated by applying various criteria listed in Chapter 3.2.2. If an armor layer exists, significant bedload transport is assumed to occur when the Shields parameter exceeds the critical value for armor layer scour. However, for certain ratios of sediment size to armor block size, suction can remove base material even though the criterion for sediment transport is not fulfilled. The possibility of suction removal was checked by comparing the particle size ratio of armor layer and base material with the ratio of the critical Shields parameter for suction removal (Equation (3.20)) and initiation of bedload transport.

### 4.1.4 Sensitivity analysis of bedload estimation

An analysis was conducted on the bedload estimations of the Reuss River at the Pfaffensprung SBT to assess the sensitivity of the obtained results. The input parameters were accordingly changed by ±25%. Among all input parameters the mean particle size \(d_m\) and the slope \(S\) have the highest impact on the bedload estimation with ±50% and ±90%, respectively. Altering the channel width \(b\) has a significant impact on the estimated bedload at ±20%, while altering \(d_{90}\) has only a marginally small effect on the result (less than ±3%).

The river topography in the investigated river reach is relatively stable and was determined with an error of less than 5%, while the error in the GSD estimation varied by ±20% (Appendix F.2). The error of discharge measurements is typically ±10%. The overall error for the bedload estimation was calculated following the procedure explained in Appendix C and resulted in approx. ±50% deviation. However, due to the intermittent characteristic of the bedload transport processes, even higher deviations may occur.

### 4.1.5 Bedload transport capacity in the SBT

Different equations are available to calculate the bedload transport capacity in fixed bed channels with low relative roughness heights. The following equations are most suitable for applications in SBTs with supercritical open channel flows: Pedroli (1963) (P); Novak and Nalluri (1975) (NN), Smart and Jäggi (1983) (SJ) and Auel (2014) (A). (P) holds for slopes up
to 2% but many SBTs exhibit higher slopes. (A) was developed based on only a few data points at slopes between $S = 0.01 - 0.04$. However, (A) was particularly established for the flow conditions in SBTs, and was therefore also considered.

It is noted that a major issue of SBTs is to ensure operating safety and avoid clogging of the SBT. Therefore, the sediment transport capacity of the SBT must always exceed the sediment supply of the river.

### 4.1.6 Suspended sediment transport

Although suspended sediment transport has insignificant impact on hydroabrasion at SBTs, it is of particular interest regarding reservoir sedimentation and bypass efficiency analysis. The suspended sediment loads were either obtained from indirect SSC measurements by means of turbidimeters or estimated respecting the catchment characteristics. For the latter, two approaches were used: (I) application of bedload to suspended sediment load ratios (cf. Chapter 3.2.7) using the estimated bedload and (II) application of specific suspended sediment supply rates to the catchment area.

The specific turbidity, i.e. turbidity/SSC of monodisperse suspensions is approximately proportional to $1/d$. Moreover, particles larger than 0.25 - 1.0 mm were found to have little impact on the turbidity (Campbell and Spinrad 1987, Black and Rosenberg 1994). Therefore, turbidity measurements were assumed to depend only on particles smaller than 0.5 mm. The turbidity probes were calibrated by means of bottle sampling. The resulting calibrations were applied to the turbidity point measurements and expanded to SSC-profiles according to Rouse (1937) described in Chapter 3.2.7. By integrating the SSC and discharge time series finally, the suspended sediment mass load (SSL) was computed.

### 4.1.7 Material properties

Invert material properties are of prime importance for hydroabrasion investigations describing the resistance of an invert material against erosions. A common value to characterize a material is its strength. Often the compressive strength is the only provided parameter, whereas further parameters are required for abrasion models. The parameters can be determined according to the following equations. However, it should be noted that material properties obtained from material tests are preferable over derived values, since the latter might deviate from the actual value due to model errors and uncertainties.
The standard SIA 262:2003 provides the following formula for the determination of the Young’s modulus, whereby kE is a coefficient accounting for the aggregate type (equal to 10,000 for sound river gravel) and fc,cyl is the cylindrical compressive strength:

\[ Y_{st} = k_E \sqrt{f_{c,cyl}} \quad [\text{Pa}] \quad (4.7) \]

with

\[ f_{c,cyl} \approx 0.8 \cdot f_c \quad [\text{Pa}] \quad (4.8) \]

Noguchi et al. (2009) analyzed existing data sets of laboratory experiments in order to find a universal equation considering not only compressive strength but other influencing parameters such as aggregate type, mineral admixtures and concrete density \( \rho_c \). The proposed formula reads:

\[ Y_{st} = k_1 \cdot k_2 \cdot 33500 \cdot \left( \frac{\rho_c}{\rho_s} \right)^2 \left( \frac{f_{c,cyl}}{f_c} \right)^{1/3} \quad [\text{Pa}] \quad \text{for } 40 < f_{c,cyl} < 160 \text{ MPa} \quad (4.9) \]

With \( k_1 = 1.005 \) for river gravel, \( k_2 = 0.95 \) for silica fume addition, the reference density and compressive strength \( \rho_s = 2400 \text{ kg/m}^3 \) and \( f_c^* = 60 \text{ MPa} \), respectively. If no data are available, the density of concrete is recommended to \( \rho_c = 2500 \text{ kg/m}^3 \), whereas a detailed list of rock density is provided by Tenzer et al. (2011).

The uniaxial tensile strength \( f_t \) of normal concrete is roughly 10% of the compressive strength (SIA 262:2003) and is related to the bending tensile strength \( f_{bt} \) and splitting tensile strength \( f_{st} \) as follows (Bamforth et al. 2008):

\[ f_t = f_{bt} / 1.5 = 0.90 \cdot f_{st} \quad [\text{Pa}] \quad (4.10) \]

A common correlation between splitting tensile strength and cylindrical compressive strength was reported by Arioglu et al. (2006):

\[ f_{st} = 0.387 \cdot f_{c,cyl}^{0.63} \quad [\text{Pa}] \quad \text{for } 4 < f_{c,cyl} < 120 \text{ MPa} \quad (4.11) \]

In the present study, splitting tensile strength was either derived from bending tensile strength, using Equation (4.10), or from the compressive strength (cf. Equations (4.11) and (4.8)) if it was not experimentally determined.

Note that concrete can be classified based on its compressive strength into (1) normal concrete with a cylindrical compressive strength of \( f_{c,cyl} < 50 \text{ MPa} \) and a water-cement ratio of \( w/c = 0.45 - 0.7 \), (2) high-strength concrete with \( f_{c,cyl} = 50 - 100 \text{ MPa} \) and \( w/c \approx 0.35 \) and (3) ultra-high strength concrete with \( f_{c,cyl} > 100 \text{ MPa} \) and \( w/c \approx 0.2 \). High performance concrete
(HPC) and ultra-high performance concrete (UHPC) are defined as concretes meeting special combinations of performance (e.g. impermeability, durability and resistance against chemical and mechanical stresses) and uniformity requirements not always achieved routinely using conventional constituents and normal mixing, placing, and curing practices (ACI 2013).

4.2 Geophone calibration

A Swiss Plate Geophone System (SPGS) was installed for continuous bedload transport measurement in the Solis SBT. This device requires a site-specific calibration, which was performed both in the laboratory and the field. In the following, the SPGS is explained followed by a description of both the laboratory and the field calibration setup.

4.2.1 Swiss Plate Geophone System

The Swiss Plate Geophone System (SPGS) developed by the Swiss Federal Institute for Forest, Snow and Landscape Research WSL is a robust device allowing for continuous bedload transport monitoring in rivers and torrents with high flow velocities (Rickenmann and Mc Ardell 2007, Turowski and Rickenmann 2009, Rickenmann and Fritschi 2010, Rickenmann et al. 2012, Rickenmann et al. 2013, Wyss et al. 2015, Rickenmann et al. 2016, Koshiba et al. 2017). The SPGS is submersible and consists of an elastically bedded steel plate mounted flush to the channel bed. The plate is equipped with a geophone sensor (GS-20DX, manufactured by “Geospace Technologies”, Houston, Texas), encased by a waterproof aluminum housing (Figure 4.1). The length, width and thickness of the plate corresponding to streamwise, transversal and vertical directions are 36 cm, 50 cm and 1.5 cm, respectively. The bearing between the steel plate and the mounting steel box is made of rubber (elastomer type CR/SRB-standard 65±5, manufactured by “Angst + Pfister”, Zurich, Switzerland). Besides signal damping issues, this bearing serves for isolating vibrational noise generated in the surroundings.

The sensor does not directly measure bedload transport, but registers the vibration signal of the geophone plate, i.e. the vertical plate oscillations induced by impingement of passing particles (Turowski et al. 2013). The signal output is a voltage corresponding to the plate velocity and is recorded at a sampling frequency of 10 kHz. To filter out background noise and vibrations generated by clear water flow, a threshold signal value of 0.1 V is defined in accordance with other applications (Rickenmann et al. 2013, Wyss 2015, Chiari et al. 2016). The detection particle size amounts to \( d = 20 - 30 \) mm (Morach 2011, Rickenmann et al. 2012, Wyss 2016b, Koshiba et al. 2017). Since the signal amplitude decreases with decreasing particle size, the
detection particle size can be reduced by decreasing the threshold signal value (Wyss 2015). However, reducing the threshold signal value by 50% did not significantly affect the computed total sediment mass and therefore was not applied herein (cf. Chapter E.3).

Figure 4.1  a) Geophone sensor (courtesy of Geospace Technologies), and b) open measuring unit showing the steel plate and the waterproof aluminum casing of the sensor

The number of impulses ‘Imp’ above the threshold value (Figure 4.2) correlates linearly with bedload mass $m$ (Rickenmann 1997, Rickenmann et al. 2012). The linear relation coefficient between impulses and bedload mass $K_b$ is used to estimate the sediment transport rate and is defined as:

$$K_b = \frac{\text{Imp}}{m} \quad [1/\text{kg}]$$ (4.12)

This calibration coefficient is affected by flow conditions, grain size and shape. As adequate reproduction of these conditions in the laboratory is difficult, a site-specific calibration is required (Rickenmann and McArdell 2007, Rickenmann et al. 2012, Rickenmann et al. 2013, Wyss et al. 2014, Wyss et al. 2015, Wyss et al. 2016a, Wyss et al. 2016b). To this end, either a certain amount of sediment is mechanically supplied to the SPGS or the actual sediment transport rates are monitored by using either fixed bedload traps, retention basins or moving baskets (Rickenmann and McArdell 2008, Rickenmann et al. 2012, Rickenmann et al. 2014, Chiari et al. 2016). To calibrate the Solis SPGS, sediment trapping is not feasible and mechanical sediment supply is elaborate and expensive. Therefore, in a first step systematic laboratory calibration tests were performed. Afterwards, the laboratory calibration was supplemented and validated by means of in-situ tests using mechanically supplied sediments (Albayrak et al. 2016, Albayrak et al. 2017, Mueller-Hagmann et al. 2017b).

The calibration coefficient can be further affected by signal interference induced by impact overlaps (Wyss 2016a, Dhont et al. 2017, Koshiba et al. 2017). The probability of this interference can be determined by $z_p$, defined as the ratio of total signal envelope time exceeding
the impulse counting threshold (Figure 4.2) to the total bedload sampling duration $T$ (Wyss et al. 2015):

$$z_p = \frac{\sum \Delta t_i}{T} \quad [-]$$  \hfill (4.13)

At $z_p \leq 0.01$, the signals of impinging particles rarely overlap and do not significantly affect the measurements, so that the bedload analysis is expected to deliver accurate results. However, with increasing $z_p$, the effect of signal overlaps increases and causes a certain signal saturation, biasing bedload estimations particularly for $z_p > 0.1$ (Wyss, 2016).

![Schematic geophone signal](image)

Figure 4.2  Schematic geophone signal

Recent investigations reveal that not only sediment transport but also grain size information can be extracted from the amplitude of the SPGS signals (Wyss et al. 2014, Wyss et al. 2016a, Wyss et al. 2016b). The amplitude class method (AC-method) introduced by Wyss (2016) bases on the fact that larger particles exhibit higher impact energy, resulting in higher signal amplitudes compared to small particles (Etter 1996, Rickenmann et al. 2014). By introducing different threshold amplitudes for different particle size classes, the relative number of impulses and the corresponding bedload mass of each particle size class are computed.

### 4.2.2 Laboratory geophone calibration

The SPGS installed in the Solis SBT was calibrated in a 7.55 m long, 0.50 m wide and 0.8 m deep laboratory flume with a maximum discharge of 180 l/s. The slope of the flume was $S = 0.01$ and the bed was concrete-lined. The right and left flume walls were made of steel and of plexiglas, respectively (Figure 4.3a and b).

The discharge was controlled with a magnetic flow meter and transferred from pressurized flow to supercritical free-surface flow using a jetbox developed at VAW (Schwalt and Hager 1992).
The flow was slightly decelerated along the flume. The flow depths for each test run was measured at the upstream end of the geophone by using a point gauge (accuracy of ±0.25 mm). The measurement accuracy decreased with increasing flow velocities due to air entrainment and irregular water surface. All test runs were conducted at the same steady-state flow conditions with water depth $h = 0.10$ m and cross-sectional average flow velocity $U = 3$ m/s.

The geophone was inclined 10° against the flume bed (Figure 4.3b). The sampling rate was 10 kHz, and an impulse threshold value of 0.1 V was selected.

![Figure 4.3](image)

**Figure 4.3**  a) Overview of the experimental flume in operation (flow from bottom to top), and b) side view of the geophone 10° inclined against the flow (flow from right to left)

### 4.2.2.1 Sediment samples

To reproduce the site-specific sediment transport characteristics of the Solis SBT, the sediments used in the tests were similar to the sediments in the Solis Reservoir in size and shape. The tested GSD was limited by the detection threshold of the device and the maximum grain size expected to pass through the SBT. The SPGS in Solis detects sediment particles larger than 2 to 3 cm (Morach 2011). The largest particle diameter corresponded to the $d_{90} = 150$ mm determined at the inflow of the Solis Reservoir.

Natural river sediments were used in the test. The sediments were sieved and classified based on their mass with ±20% deviation from the mean class weight. The mean particle diameter ($b$-axis) ranged from 22 to 158 mm and varied ±10% within the mass classes due to the variation
of the naturally shaped grains. Figure 4.4 shows the particle mass as a function of the particle diameter. The relation between particle size and mass is derived from the volume formula for spheres \((\pi/6 \cdot d^3)\) and enhanced by a correction factor of 0.762 considering the site-specific particle shapes by applying the least square fitting. The solid density was assumed to be \(\rho_s = 2650 \text{ kg/m}^3\). The fit results in \(m = 0.762 \cdot \pi/6 \cdot \rho_s \cdot d^3\) with \(R^2 = 0.998\).

<table>
<thead>
<tr>
<th>(d \text{ [mm]})</th>
<th>(m \text{ [g]})</th>
</tr>
</thead>
<tbody>
<tr>
<td>22</td>
<td>16</td>
</tr>
<tr>
<td>28</td>
<td>32</td>
</tr>
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<td>36</td>
<td>64</td>
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<td>46</td>
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<td>96</td>
<td>1024</td>
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<tr>
<td>123</td>
<td>2048</td>
</tr>
<tr>
<td>158</td>
<td>4096</td>
</tr>
</tbody>
</table>

Figure 4.4   Particle diameter and mass of the tested material

4.2.2.2  Test program

All tests were conducted at the same hydraulic conditions with \(S = 0.01, h = 0.10 \text{ m}\) and \(U = 3 \text{ m/s}\), while the sediment size and supply rate were varied. Since the sediment particles were too large for the available sediment dosing machine, sediment was manually supplied at a distance of 1.5 m downstream from the jetbox at the flume inlet (Figure 4.3a). The particles were added with three different supply rates “single”, “normal” and “pack”. “Single” means single stone experiments without particle interaction or impact overlap. “Normal” is defined as a supply rate smaller than the sediment transport capacity of the flow, but high enough to enable particle interactions and signal interference. The supply duration of normal supply was 2 s. A bulk of particles was supplied simultaneously for the “pack” test runs, which resulted in a supply duration of less than 0.5 s.

In a first test series, single stone tests with uniform sediment were conducted. In a second test series, the supply rate was increased to normal supply. The sediment samples consisted of 20 stochastically picked stones from one grain size class. The number of stones was reduced for the larger classes due to their heavy weight. The two largest size classes were supplied stone per stone due to their large mass. Hence, only single stone experiments were conducted with these size classes. To investigate the influence of supply rate on impact overlap in a third test
series, uniform sediment material was supplied simultaneously. A fourth test series using two different sediment mixtures namely coarse and fine mixtures, was conducted to study the effect of the sediment uniformity on the measurements. The sediment mixture samples and their GSD are shown Figure 4.5 and Figure 4.6, respectively. Their compositions are given in Table 4.1. The mass of the largest particles of the coarse and fine mixture were 1024 g, and 512 g, respectively. The number of particles of the fine mixture was 42, i.e. twice as high as of the coarse mixture. Assuming a linear distribution of grain size within ±10% around the mean, the grain size of each size class and \( \sigma = (d_{84}/d_{16})^{0.5} \) of both mixtures were calculated.

![Figure 4.5](image)

**Figure 4.5**  a) Coarse and b) fine sediment mixtures

**Table 4.1**  Composition of sediment mixtures

<table>
<thead>
<tr>
<th>Mixture</th>
<th>( d ) [mm]</th>
<th>( m ) [gr]</th>
<th>Number of stones per run [-]</th>
<th>( \sigma = (d_{84}/d_{16})^{0.5} ) [-]</th>
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<tr>
<td>“coarse”</td>
<td>36</td>
<td>64</td>
<td>16</td>
<td>1.86</td>
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<td>( d_m = 78 ) mm</td>
<td>75</td>
<td>256</td>
<td>4</td>
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<tr>
<td></td>
<td>123</td>
<td>1024</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>“fine”</td>
<td>28</td>
<td>32</td>
<td>32</td>
<td>1.86</td>
</tr>
<tr>
<td>( d_m = 61 ) mm</td>
<td>59</td>
<td>128</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td></td>
<td>96</td>
<td>512</td>
<td>2</td>
<td></td>
</tr>
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</table>
Figure 4.6  Grain size distribution of the sediment mixtures assuming a deviation of ±10% from the mean diameter of each grain size class

Table 4.2 summarizes the test conditions of the 4 test series. Sediment supply duration denotes the target supply duration. However, the effective supply duration may deviate from the target supply duration about ± 20% due to the manual sediment supply. The sediment transport rate was calculated based on the bedload sampling duration of the SPGS and the weighted sediment mass. Note that the sediment supply duration did not correspond to the bedload sampling duration.
Table 4.2 Parameters of the geophone calibration test runs in laboratory

<table>
<thead>
<tr>
<th>Test Series</th>
<th>GSD</th>
<th>Supply rate</th>
<th>Supply duration [s]</th>
<th>Stones per run [-]</th>
<th>d [mm]</th>
<th>Mass per run [g]</th>
<th>Repetitions [-]</th>
<th>$Q_s$ [kg/s]</th>
<th>$z_p$ [-]</th>
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</table>
4.2.3 Field geophone calibration

4.2.3.1 Experimental test setup

The SPGS installed at the outlet of the Solis SBT was calibrated in-situ by using three different sediment size classes. Three calibration runs, one for each grain size class, were conducted. The test procedure was:

- Deposition of 10 m$^3$ of sediment downstream of the SBT intake gate
- SBT operation at high reservoir water level and SPGS signal recording
- Raw data analysis of the SPGS

All test runs were conducted at identical conditions listed in Table 4.3. The reservoir level was high, i.e. 0.83 - 0.85 m below the full supply level of 823.75 masl. Thus, the bed shear stresses were too low to entrain settled sediments from the reservoir and only clear water entered the SBT. The sediment deposits were located at the SBT inlet and the discharge was $Q = 50$ m$^3$/s in order to achieve characteristic bedload transport of typical SBT operation, while limiting water loss. The corresponding average flow velocity was $U = 9.8$ m/s and the flow depth at the SPGS was $h = 1.16$ m. These values deviate from the typical SBT operating conditions (Table 4.3). However, the sediment transport processes of the calibration tests and of typical SBT operation can be assumed to be identical since for both cases the flow was supercritical, i.e. $F > 1$ and the flow depth was significantly larger than particle size, i.e. $h/d > 3$.

### Table 4.3 Conditions in the SBT during normal operation and calibration tests

<table>
<thead>
<tr>
<th></th>
<th>Calibration tests</th>
<th>Typical SBT operation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reservoir water level [masl]</td>
<td>823.91 ± 0.01</td>
<td>816 - 814.5</td>
</tr>
<tr>
<td>$Q$ [m$^3$/s]</td>
<td>~ 50</td>
<td>90 - 170</td>
</tr>
<tr>
<td>$h$ [m] (at outlet)</td>
<td>~ 1.16</td>
<td>2 - 3.6</td>
</tr>
<tr>
<td>$U$ [m/s]</td>
<td>~ 9.8</td>
<td>10.5 - 10.8</td>
</tr>
<tr>
<td>$F$ [-]</td>
<td>~ 2.9</td>
<td>2.4 - 1.7</td>
</tr>
<tr>
<td>$h/d$ [-]</td>
<td>~ 3 - 40</td>
<td>10 - 100</td>
</tr>
</tbody>
</table>

4.2.3.2 Sediment samples

The sediments used for the geophone calibration were taken from the gravel plant “Kieswerk Albula AG, Tiefencastel”, located on the Albula River at the head of the Solis Reservoir. Hence, the sediment properties were identical to the sediments transported through the SBT during normal operation.
Three sediment samples, namely fine material, coarse material and mixture were used. Their GSD was provided by the supplier and checked by line-sampling according to Fehr (1987). The gravimetric portion of a grain size class $\Delta p_i$ was determined as:

$$\Delta p_j = \frac{n_i}{\sum n_i} d_{m,i}^{1.8}$$

with $n_i$ = number of particles in the grain size class $i$ and the corresponding mean grain size $d_{m,i}$ (Fehr 1987). The exponent for the particle weighting takes into account flow driven particle size sorting processes and is typically 0.8 for river engineering purposes. Due to the absence of hydraulic exposure the exponent applied here is 1.8 according to Fehr’s (1987). The mixture sample additionally required a correction of the fine-grained limb. The fraction of the particles smaller than 16 mm were assumed to amount to 15% of the total sediment mass and were accordingly included. The resulting GSDs are shown in Figure 4.7 and listed in Table 4.4. The sediment samples used for the calibration tests cover the whole range of the sediment transported through the SBT (=Solis GSD). Depending on particle size, different transport modes arise (Chapter 4.3.5). The fine material used in Test 1 is expected to be mainly in suspension, while the coarse and mixture materials of Test 2 and 3 are mainly transported as bedload resulting in a higher interaction with the SPGS (Figure 4.7).

![Figure 4.7](image)

Figure 4.7 GSD of the three test runs of the geophone field calibration, the Solis sediment, and the transport mode depending on grain size for design operation conditions in the Solis SBT

The sediments were weighed at the gravel plant and transported by a dumper to the test site located 20 m downstream of the SBT inlet. After dumping, the sediment was mechanically spread across the whole tunnel width with a layer thickness of 20 cm and a wedged forehead (Figure 4.8a and b).
Table 4.4 Sediment of the calibration test runs and the natural Solis sediment

<table>
<thead>
<tr>
<th>Name</th>
<th>$d$ [mm]</th>
<th>$d_m$ [mm]</th>
<th>$\sigma = (d_{84}/d_{16})^{0.5}$</th>
<th>Volume [m$^3$]</th>
<th>Mass [to]</th>
</tr>
</thead>
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<tr>
<td>Fine material</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Test 1</td>
<td>16 - 32</td>
<td>25 ± 2</td>
<td>1.3</td>
<td>10.01</td>
<td>15.3</td>
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<td>Coarse material</td>
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<td>Test 2</td>
<td>32 - 63</td>
<td>45 ± 4</td>
<td>1.3</td>
<td>9.94</td>
<td>15.4</td>
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<td>Mixture</td>
<td>Test 3</td>
<td>0 - 400</td>
<td>210 ± 20</td>
<td>11.53</td>
<td>18.4</td>
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<td>Solis natural</td>
<td>Solis GSD</td>
<td>0 - 300</td>
<td>60</td>
<td>6.95</td>
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</table>

Figure 4.8 a) Dumping of sediment, and b) sediment of the size class 32 to 63 mm (Test 2) spread across the tunnel

4.3 Test case Solis SBT

The test setup of the Solis SBT is presented herein. First, general information on the river including the sediment properties, historical information on the reservoir and detailed information on the SBT are given. Second, the test fields equipped with different invert materials are introduced, followed by information on the instrumentation and the description of the procedure applied to determine the abrasion depths. Third, the site-specific distinction between bedload and suspended load is provided. Finally, the method to estimate the sediment transport rates and masses into and out of the Solis Reservoir are described in detail.

4.3.1 Albula River

The Albula drains a catchment area of 900 km$^2$, located between 822 and 3406 masl, of which 1.5% is covered by glaciers. The mean annual discharge is 29.5 m$^3$/s, and the flood peaks of the one-, five- and 100-year events are $H_{Q_1} = 90$ m$^3$/s, $H_{Q_5} = 170$ m$^3$/s and $H_{Q_{100}} = 280$ m$^3$/s, respectively. The two rivers Julia and Albula coalesce about 1 km upstream of the Solis Reservoir shown in Figure 4.10. The Julia River is strongly affected by the upstream reservoirs
Burvagn and Marmorera feeding the Tiefencastel power plant. Its contribution to the annual sediment inflow to the Solis Reservoir is accordingly small compared to the Albula (Zarn 2010). Therefore, sediment transport is only investigated in the Albula, while the contribution of the Julia River was neglected.

The topography of the Albula was investigated based on 12 cross-sections determined with aerial photos and elevation maps provided by www.map.geo.admin.ch (Appendix E.1). The mean slope and channel width of the Albula are $S = 0.0088$ and $b = 18.75$ m, respectively. These values are in agreement with former sediment transport investigations (Zarn 2009, 2010).

The bed material of the Albula mainly consists of limestone with less than 20% quartz (GEWISS 2014). The mean annual bedload supply of the Albula was expected to range between 40’000 and 55’000 m$^3$ (Zarn 2009, 2010). Its GSD was investigated by VAW (2008), Schläppi (2009), Awazu (2015) and WSL (2015). The characteristic particle sizes and GSDs are listed and presented in Table 4.5 and Figure 4.9, respectively. The results are in agreement, except from Schläppi (2009), who reported smaller particle sizes as a consequence of the sampling location in the reservoir instead of in the river. The GSD of the sediment transported through the SBT is assumed to follow the one from VAW (2008) with a slight tendency toward the average of the existing GSD information (Table 4.5 and Figure 4.9).

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$d_{30}$</td>
<td>0.15</td>
<td>0.02</td>
<td>0.15</td>
<td>0.028</td>
<td>0.015</td>
</tr>
<tr>
<td>$d_{50}$</td>
<td>0.04</td>
<td>0.009</td>
<td>0.028</td>
<td>0.053</td>
<td>0.035</td>
</tr>
<tr>
<td>$d_{m}$</td>
<td>0.06</td>
<td>0.019</td>
<td>0.041</td>
<td>0.069</td>
<td>0.06</td>
</tr>
<tr>
<td>$d_{84}$</td>
<td>0.107</td>
<td>0.040</td>
<td>0.075</td>
<td>0.150</td>
<td>0.097</td>
</tr>
<tr>
<td>$d_{90}$</td>
<td>0.15</td>
<td>0.050</td>
<td>0.103</td>
<td>0.175</td>
<td>0.15</td>
</tr>
</tbody>
</table>
4.3.2 Solis Reservoir and SBT

The Solis Reservoir was commissioned in 1986. It is fed by the Albula and by the tailrace channel of the HPP Tiefencastel (Figure 4.10). Initially, the total storage volume was $4.07 \times 10^6$ m$^3$, with an active volume of $1.46 \times 10^6$ m$^3$. The total storage capacity is relatively small compared to the mean annual runoff of $\text{MAR} = 853 \times 10^6$ m$^3$, which results in a low capacity-inflow ratio of $\text{CIR} = 0.0048$. The stored water is directed to turbines at the HPP Sils and Rothenbrunnen (design discharge $Q_d = 22$ and $25$ m$^3$/s, respectively) before being released to the Albula or Hinterrhein River, respectively.

![Figure 4.10](map.png)  
**Figure 4.10** Overview of the hydraulic scheme of the Solis Reservoir with SBT and HPPs Tiefencastel, Sils and Rothenbrunnen (map: courtesy of map.geo.admin.ch)
From 1986 to 2008, 25% of the reservoir storage capacity was lost due to reservoir sedimentation (Figure 4.11). Bathymetric surveys showed that the Solis Reservoir suffered from considerable sedimentation of 79'800 m$^3$ per year on average (Meisser-Vermessungen 2015), despite sand and gravel mining at the reservoir head. The mean annual excavation volume since the commissioning of the dam was 31’400 m$^3$ (TBA-GR 2015) so that, the mean annual sedimentation volume amounts to 79’800 m$^3$ + 31’400 m$^3$ = 111’200 m$^3$. The settled material was partially relocated from the active to the dead storage volume by reservoir drawdowns in 2006 and 2008. However, assuming a constant aggradation rate, the hydropower generation would have been increasingly affected, and the aggradation body was expected to reach the dam by 2012, endangering its operational safety (Auel et al. 2011, Oertli and Auel 2015). To reduce the sedimentation and restore the interrupted sediment transport in the river reach, a 1 km long SBT was constructed and commissioned in 2012 (Figure 4.12).

The slope of the SBT is $S = 0.019$ and the area of the arched tunnel cross section is 18.5 m$^2$. The design discharge of $Q_d = 170$ m$^3$/s corresponds to a five-year flood event. Since the SBT intake is located below the drawdown level, the inflow is pressurized and no acceleration section is required. After the intake, the flow is decelerated but remains supercritical with Froude number $F \approx 1.7$ at design discharge. Hence, a sufficient sediment transport capacity is ensured along the tunnel.

![Figure 4.11](image.png)  
**Figure 4.11**  Evolution of the aggradation body since the commissioning of the reservoir, intake of the SBT put into operation in 2012, active storage and gravel plant

Prior to a flood event with discharges above 90 m$^3$/s, the reservoir water level is drawn down to levels close to 816 masl to ensure sufficient sediment transport capacity between the reservoir head and the SBT inlet. At approach flow discharges above 90 m$^3$/s the SBT is put in operation and diverts bedload and suspended load with a flow velocity around 11 m/s past the dam to the
downstream river reach. The entire approach flow is bypassed up to an approach flow discharge of 170 m$^3$/s. Hence, the downstream part of the reservoir between the SBT guiding structure and the dam is protected from sedimentation. At higher discharges, water is additionally released by the dam spillway and bottom outlets. Therefore, the surplus flow passes the guiding structure. While bedload is still diverted by the guiding structure to the SBT, considerable amounts of suspended sediment enter the downstream part of the reservoir, where they partially settle.

### 4.3.3 Test fields

Six 10 m long and 4.4 m wide test fields and an elaborate measurement system to continuously monitor the SBT operating conditions have been implemented in the SBT. Since the tunnel bends cause non-uniform sediment transport across the tunnel width, the test fields were implemented at the end of the 527 m long straight section. Thus, unequal impact of bedload particles across the tunnel width on the invert is minimized (Figure 4.12).

![Figure 4.12 Overview of Solis SBT with six test fields, and installed measurement devices](image)

The test fields were implemented in winter 2011/12 and separated by steel beams in order to prevent damage propagating from one test field to the other. A large spectrum of invert materials such as various concretes, steel, wood and rubber was pre-evaluated by considering cost and the risk of sudden collapse of the supporting structure. Finally, four different concrete mixtures, a steel armoring (embedded on self-compacting concrete) and cast basalt plates were chosen and installed (Figure 4.13). Table 4.6 summarizes the materials and their composites. Besides the normal SBT lining consisting of a C75/85 concrete with 45 kg of steel fibers (Normal Concrete, NC), a similar but slightly weaker concrete C55/67 with higher steel fiber...
content (High-Strength Concrete, HSC) was selected. Another concrete type contains plastic fibers instead of steel fibers and a shrinkage reducer (Low Shrinkage Concrete, LSC). Furthermore, a high alumina cement concrete (High Alumina Cement concrete, HAC) known for its high abrasion resistance and an Ultra-High Performance reinforced Concrete (UHPC) were installed.

![Figure 4.13 Sketch of the test fields at the Solis SBT](image)

### Table 4.6 Concrete mixture of the test fields at the Solis SBT

<table>
<thead>
<tr>
<th>Composite [kg/m³]</th>
<th>NC</th>
<th>HSC</th>
<th>LSC</th>
<th>HAC</th>
<th>UHPC</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEM II/A-D 52.5 R</td>
<td>536</td>
<td>450</td>
<td>390</td>
<td></td>
<td></td>
</tr>
<tr>
<td>High alumina cement</td>
<td></td>
<td></td>
<td></td>
<td>515</td>
<td></td>
</tr>
<tr>
<td>CEM II/B-M</td>
<td></td>
<td></td>
<td></td>
<td>1100</td>
<td></td>
</tr>
<tr>
<td>Water</td>
<td>177</td>
<td>185</td>
<td>172</td>
<td>206</td>
<td>187</td>
</tr>
<tr>
<td>Steel fibers:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dramix RC-80/30-BP</td>
<td>45</td>
<td>60</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel fibers:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>236</td>
</tr>
<tr>
<td>Dramix OL-13/0.16</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Plastic fibers:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3</td>
</tr>
<tr>
<td>Superfiber 40/8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand 0/4 mm*</td>
<td>760</td>
<td>713</td>
<td>760</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gravel 4/8 mm*</td>
<td>456</td>
<td>382</td>
<td>456</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gravel 8/16 mm*</td>
<td>684</td>
<td>626</td>
<td>684</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gravel 8/11 mm*</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Alag 0/4</td>
<td></td>
<td></td>
<td></td>
<td>1030</td>
<td></td>
</tr>
<tr>
<td>Alag 4/10</td>
<td></td>
<td></td>
<td></td>
<td>1030</td>
<td></td>
</tr>
<tr>
<td>Quartz sand</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>870</td>
</tr>
<tr>
<td>Superplasticizer</td>
<td>13.4</td>
<td>11.3</td>
<td>0</td>
<td>35</td>
<td></td>
</tr>
<tr>
<td>Shrinkage reducer</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>11.7</td>
</tr>
</tbody>
</table>

* Material from the local gravel plant "Nisellas" in Tiefencastel
Most concretes were produced by “Beton und Kies AG Unterrealta” in Cazis, Switzerland, and directly delivered by a truck mixer to the test site, while the UHPC and the HAC were mixed on site by using small concrete mixers (capacity 0.50 m$^3$). After implementation, the concretes were compacted and flattened by poker vibrators and beams resulting in flat, well-cured surfaces (Figure 4.14a). Since the HAC produces high hydration heat, it was installed in checkered concreting stages measuring 1.5 m × 2.0 m. The working joints were slightly visible after the installation (Figure 4.14b). The cast basalt plates (20 cm × 20 cm × 5 cm) provided by “Gerbas GmbH” Ibach, Switzerland, were paved by using a two-component cementitious adhesive. The 3 mm wide joints were filled with a two-component epoxy grout under pressure (Figure 4.14c). The 20 mm thick steel armoring (steel type S235 (CEN 2004)) was installed following a special procedure: (I) positioning of the steel plates (4.4 m × 1.0 m) equipped with spacers, adjusting screws and anchors, (II) welding the steel plates together and (III) filling the void between armoring and underground with self-compacting concrete (SCC) consisting of C35/45 concrete. The obtained surface was flat, but the weld seam was visible (Figure 4.14d).

The invert material properties are listed in Table 4.7. For each concrete test field the material properties, i.e. compressive and bending tensile strengths, were determined in the laboratory of “Griso Prüflabor AG” in Untervaz, Switzerland, by using three 28-day samples produced during the installation. The sample geometries are listed in the Appendix, Table E.2. Table 4.7 includes additionally the properties of the cast basalt and steel provided by the manufacturer.

<table>
<thead>
<tr>
<th>Material</th>
<th>Density $\rho_c$ [kg/m$^3$]</th>
<th>Strengths</th>
<th>Young's Moduli $Y_M$ [GPa]</th>
<th>Thickness [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>NC</td>
<td>2500 ± 12</td>
<td>$f_c$ = 105 ± 1.8, $f_{st}$ = 11.5 ± 0.5, $f_{bt}$ = 8.5 ± 0.3*</td>
<td>38.3**</td>
<td>0.3</td>
</tr>
<tr>
<td>HSC</td>
<td>2474 ± 11</td>
<td>$f_c$ = 78.9 ± 2.6, $f_{st}$ = 12.4 ± 0.5, $f_{bt}$ = 9.2 ± 0.3*</td>
<td>35.3**</td>
<td>0.3</td>
</tr>
<tr>
<td>LSC</td>
<td>2444 ± 20</td>
<td>$f_c$ = 84.7 ± 2.2, $f_{st}$ = 10.8 ± 0.6, $f_{bt}$ = 8.0 ± 0.4*</td>
<td>36.1**</td>
<td>0.3</td>
</tr>
<tr>
<td>HAC</td>
<td>2699 ± 13</td>
<td>$f_c$ = 86.3 ± 3.4, $f_{st}$ = 11.5 ± 0.8, $f_{bt}$ = 8.5 ± 0.5*</td>
<td>36.4**</td>
<td>0.15</td>
</tr>
<tr>
<td>UHPC</td>
<td>2400 ± 12</td>
<td>$f_c$ = 187 ± 11, $f_{st}$ = 20.9 ± 1.1, $f_{bt}$ = 15.5 ± 0.7*</td>
<td>47.1**</td>
<td>0.08</td>
</tr>
<tr>
<td>Cast basalt*</td>
<td>2950</td>
<td>$f_c$ = 300 - 450, $f_{st}$ = 45, $f_{bt}$ = 33</td>
<td>111</td>
<td>0.05</td>
</tr>
<tr>
<td>Steel S235**</td>
<td>7850</td>
<td>$f_y = 235$ &amp; $f_u = 360$</td>
<td>210</td>
<td>0.02</td>
</tr>
</tbody>
</table>

*calculated using Equation (4.10), ** calculated using Equation (4.9), ***data provided by the supplier; $f_y$ = yield stress, $f_u$ = rupture stress
Figure 4.14 The a) HSC, b) HAC, c) cast basalt tile and d) steel armoring test fields after implementation at the Solis SBT in spring 2012 (view in flow direction)
4.3.4 Instrumentation

The monitoring at the Solis SBT includes not only measurements in the SBT but also outside of the SBT, which were provided by third parties, i.e. FOEN, ewz and WSL. The location of the measuring stations outside of the SBT are shown in Figure 4.15. The discharge in the Albula and the Julia Rivers were continuously measured by the FOEN at the gauging stations number 2141 and 2418 in Tiefencastel, respectively. The reservoir water level was monitored by ewz using a pressure sensor RIPRESS (manufactured by Rittmeyer, Baar, Switzerland) mounted near the bottom outlet of the dam at 787 masl. The operator ewz moreover provided the hydrographs of their HPPs Tiefencastel, Sils and Rothenbrunnen and of the outlets of the Solis Reservoir. Suspended sediment transport was monitored by means of turbidimeters installed in the Albula at Tiefencastel (FOEN measuring station 2141) and in the tailrace channel of the HPP Sils. The bedload transport in the Albula was monitored by means of a SPGS installed by WSL about 500 m upstream of the FOEN station in April 2015 (Rickenmann et al. 2016). The SPGS consists of 30 steel plates and its width of 15 m corresponds to the channel width at the measuring cross-section. The measurement data from 2015, i.e. the time series of number of impulses, were provided by Rickenmann (2016) for this study. The devices were calibrated based on bedload transport estimations using (SJ) and (R) applied to the hydrograph of 2015.

![Figure 4.15](map.geo.admin.ch)  
Figure 4.15 Overview of the gauging stations of the EFON, WSL and ewz (map: courtesy of map.geo.admin.ch)

The test setup in the Solis SBT is shown in Figure 4.12 and serves for continuous, simultaneous and real time monitoring of hydraulic and sediment transport conditions in the SBT. The bedload transport was monitored by means of a Swiss Plate Geophone System (SPGS) installed at the outlet of the SBT, 100 m downstream of a right hand bend ($R = 145$ m, $\alpha = 46.5^\circ$, Figure
4.12). In order to capture not only temporal but also spatial variations of bedload transport, the SPGS consists of 8 geophone units covering the entire tunnel width (Figure 4.16). In contrast to the typical geophone applications, the flow regime in the Solis SBT is highly supercritical. A pre-study was conducted in the laboratory to address this issue (Chapter 4.2.1). The test results revealed that particle detection rates increased with the inclination of the geophone plate against the flow direction compared to the common flush mounting with the invert. Thus, the geophone system in Solis was accordingly designed with an inclination of 10° against flow direction. The steel casing of the geophones was installed on removable wedge in order to enable adaptations of the geophone inclination (Figure 4.17). More information on the SPGS is provided in Chapter 4.2.1.

![Figure 4.16 Geophone system consisting of eight single measuring units at the outlet of the Solis SBT one year after installation (view in flow direction)](image1)

![Figure 4.17 Cross section of the geophone system with removable wedges enabling adjustments of the geophone inclination (flow from right to left)](image2)

The suspended sediment transport was monitored by turbidimeters (Turbimax W CUS41 manufactured by Endress + Hauser, Reinach, Switzerland) mounted 20 cm above the tunnel.
bottom in niches in the left and right SBT tunnel walls 18 m upstream of the SPGS. The niches are covered by a perforated steel sheet, thus water exchange is ensured, while the sensors are protected. The signal transmitter (Liquisys M CUM22) from the same company transmits the signal to a data logger. Similar turbidimeters and measurement transmitters are installed at the inflow (FOEN station 2141 at the Albula in Tiefencastel) and in the tailrace channel of the Sils HPP fed by the reservoir. Both stations are illustrated in Figure 4.10. The transmitters were delivered with standard settings. One standard setting, namely the air bubble correction, needed to be changed from 3% to 100% to extend the measurement range beyond turbidities of 350 FNU. Without air bubble adaption, a non-linearity led to unrealistically high turbidities as well as high fluctuations indicating incorrect measurements above this limit (Brunschwig 2012). Since this problem was not known from the beginning, this effect distorted measurements at the FOEN gauging station 2141 from the installation until November 2012 and from December 2014 to July 2015 due to probe exchange. In the tailwater of the HPP Sils the measurements from the installation until March 2013 were affected due to the wrong setting. The turbidity measurement in the Solis SBT was not biased by this phenomena since the air bubble threshold of the turbidimeters was correct from the beginning.

Hydraulic conditions in the SBT were monitored by (I) two pressure sensors mounted in niches near the turbidity sensors and (II) a radar installed at the tunnel ceiling 90 m upstream of the outlet. The pressure sensors PR 26W (manufactured by Keller AG, Winterthur, Switzerland) are piezo-resistive pressure transmitters equipped with an integral vent tube. The maximum error is ±0.5%. The radar RQ-30 (manufactured by Sommer Mess-Technik, Koblach, Austria, distributed and installed by tytec AG, Glarus, Switzerland) measures flow velocity and water depth with an accuracy of ±1% and ±0.2%, respectively. However, the effective accuracy is lower due to air entrainment and rough water surface. For small discharges up to 30 m³/s the error is below ±10% and decreases with increasing discharge to ±5% for the normal operating range (Seeberger 2013). The SBT discharge was also calculated based on the inlet gate position and the reservoir level monitored by a pressure sensor RIPRESS (manufactured by Rittmeyer, Baar, Switzerland) mounted near the bottom outlet of the dam at 787 masl shown in Figure 4.15. The measurement error of the displacement transducer of the gate and of the energy head at the SBT intake is ±1.0% and less than ±1.5%, respectively. The head losses at the inlet were determined based on laboratory experiments with an accuracy of ±3.5%. The resulting error from error propagation for discharge calculation resulted in ±4%.

The data acquisition was automatically triggered by the gate opening. The sampling frequency was 1/60 Hz for all instruments except for the SPGS, which as sampled at 10 kHz.
The surface of the tunnel invert was mapped by a laser scanner (FARO Focus 3D, manufactured by FARO, Lake Mary, United States). The procedure is explained in Chapter 4.4.3. The first laser scanning was done in 2012 after the installation. Further scans were conducted in the winter seasons 2014/15 and 2016/17.

### 4.3.5 Sediment transport, properties and classification

The total sediment load \( TL \) includes suspended sediment load \( SL \) and bedload \( BL \). The threshold between suspended sediment and bed load transport depends on the particle size and hydraulic conditions (Chapter 3.2.3). For river engineering purposes the particle threshold size is often assumed to be 1 mm (Maniak 2010). However, the sediment transport conditions in the SBT significantly deviate from rivers. The threshold grain size in the SBT is determined based on the design operation conditions using three different approaches (Chapter 3.2.3). The threshold grain size varies between 33 mm (Kresser 1963), 22 mm (Abbott and Francis 1977, assuming 80% of the particles in suspension) and 12 mm (assuming \( V_s/U_\ast = 1 \)). Hence, there is no strict limit, rather a smooth transition from bedload to suspension as shown in Figure 4.18.

On average, particles larger than 22 mm are transported as bedload and can be detected by the SPGS in the SBT (cf. Chapter 5.1.1). Therefore, this size class is denoted as \( BL_{22} \) (Figure 4.18).

![Figure 4.18 Solis GSD with the sediment classification and the detection limits of the devices](image)

The relation between \( BL \) (on the basis of a 1 mm grain size threshold) and \( BL_{22} \) depends on the GSD and is roughly \( BL_{22} \approx 2/3 \cdot BL \) for the Solis GSD in the Albula (cf. Chapter 4.3.1). Suspended sediment accordingly comprise particles with \( d < 22 \) mm. Particles with a diameter of less than 0.5 mm can be monitored using turbidimeters (cf. Chapter 4.1.6) and are denoted as “fine suspended sediments” \( (SSL_{\text{fine}}) \). The size class between 0.5 mm and 22 mm is denoted as “coarse suspended sediment” \( (SSL_{\text{coarse}}) \). It cannot be detected by either the SPGS or the
turbidimeters. Therefore $SSL_{coarse}$ was derived from the GSD based on the measured $SSL_{fine}$ and $BL_{22}$.

To obtain the accumulated volumes (denoted by the indices $v$) from the estimated sediment masses, a sediment bulk density of 1800 kg/m$^3$ for $BL_{22}$ and of 1350 kg/m$^3$ for $SSL_{fine}$ and $SSL_{coarse}$ was assumed. Note that on average 31'400 m$^3$ out of the annual sediment transport volume in the Albula was excavated at the reservoir head. The excavation volume was assumed to consist of 50% $BL_{22}$ ($d \geq 22$ mm) and 50% of $SSL_{coarse}$. ($d = 0.5 - 22$ mm).

The methodologies to determine the sediment inflow and outflow of the Solis Reservoir for the three particle size classes (Figure 4.18) are presented in the following subsections.

### 4.3.6 Bedload transport into the Solis Reservoir

Regarding sediment transport into the Solis Reservoir, the sediment transport in the Julia River is of minor importance due to the desilting effect of the upstream reservoirs. Therefore, bedload transport into the Solis Reservoir was assumed to originate solely from the Albula, while the contribution of the Julia River was neglected.

The bedload transport capacity in the Albula was determined by applying the bedload equations introduced in Chapter 4.1.3 to the hourly hydrographs of the Albula (FOEN Station 2141). Zarn (2009, 2010) investigated bedload transport in the Albula and reported that the sediment transport capacity upstream of the Solis Reservoir is significantly higher than that at the reservoir head and the sediment supply in the catchment is large, so that the sediment transport rate equals the sediment transport capacity. The bedload transport rate was accordingly assumed to be equal to the capacity.

Between 1987 and 2016, the estimated mean annual bedload volumes ($BL_v$) varied between $46 \cdot 10^3$ m$^3$ and $174 \cdot 10^3$ m$^3$, the corresponding $BL_{22,v}$ between $31 \cdot 10^3$ m$^3$ and $116 \cdot 10^3$ m$^3$ (Table 4.8). The (MPM) led to the highest values, followed by (C), (P) and (WP). The mean annual sedimentation volume amounts to $111.2 \cdot 10^3$ m$^3$ (cf. Chapter 4.3.2) and consists of both bedload and an unknown share of suspended sediment. The result obtained from (MPM) is not realistic, since the inflowing bedload volume would be even higher than the total sedimentation volume. The results obtained from (WP), (P) and (C) imply an unrealistically low share of suspended sediment in the settled sediments of 2 - 22% and hence are not considered for further investigations as well. The equations of (R) and (SJ) led to similar estimates, which are in good agreement with Zarn’s (2009, 2010) result of $40 - 55 \cdot 10^3$ m$^3$. Moreover, they resulted in a realistic share of suspended sediment in the sedimentation volume of 44%. Therefore, the
average of (R) and (SJ) was used in this study to determine the bedload transport rates in the Albula and to calibrate the SPGS installed in the Albula.

Table 4.8 Mean annual bedload transport volume in the Albula between 1987 and 2016

<table>
<thead>
<tr>
<th>Reference</th>
<th>(MPM)</th>
<th>(WP)</th>
<th>(R)</th>
<th>(SJ)</th>
<th>(P)</th>
<th>(C)</th>
<th>Average of (SJ) and (R)</th>
<th>Zarn (2009, 2010)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( BL_v ) [10^3,m^3]</td>
<td>174</td>
<td>86</td>
<td>51</td>
<td>46</td>
<td>105</td>
<td>109</td>
<td>49</td>
<td>40 - 55</td>
</tr>
<tr>
<td>( BL_{22,v} ) [10^3,m^3]</td>
<td>116</td>
<td>57</td>
<td>34</td>
<td>31</td>
<td>70</td>
<td>73</td>
<td>32</td>
<td>27 - 37</td>
</tr>
</tbody>
</table>

Note that bedload transport is subjected to large fluctuations due to its intermittent character. Its estimation includes model uncertainties and measurements errors so that the bedload estimations error can still amount to ±50% or even more.

4.3.7 **Bedload transport out of the Solis Reservoir**

The bedload can be released from the reservoir by the SBT and / or the bottom outlet. The \( BL_{22} \) released by the bottom outlet was neglected because only a small amount of bedload particles in the close vicinity of the bottom outlet can be re-entrained and sluiced through the outlet at high reservoir water levels. Moreover, the share of coarse particles in the aggradation body decreases from the reservoir head towards the bottom outlet at the dam (VAW 2008).

The bedload transport rates in the Solis SBT were determined from the SPGS measurement by applying the following data analysis: (I) determination of the instantaneous GSD of the bypassed bedload using the AC-method (cf. Chapter 4.2.1), (II) computation of the instantaneous calibration coefficients \( K_b \) based on the instantaneous GSD by using Equation (5.1) to (5.3) with the parameter listed in Table 5.3, (III) adopting \( K_b \) using Equation (5.4) to account for the signal saturation and (IV) calculation of the instantaneous bedload transport rates by applying the instantaneous \( K_b \) to the SPGS signal, i.e. the number of impulses.

The estimations still include uncertainties despite accounting for variations in GSD and signal saturation. These originate from the laboratory and field calibration conditions possibly affecting the AC-method. According to the possible range of calibration coefficient and GSD, the effective bedload volume can be assumed to be at minimum half or at maximum twice the estimation.

4.3.8 **Fine suspended sediment transport into the Solis Reservoir**

The determination of the fine suspended sediment \( (d \leq 0.5\,\text{mm}) \) transported into the Solis Reservoir is reported in this subchapter. The mass of fine suspended sediment \( (SSL_{\text{fine}}) \) results
from summing up the product of the instantaneous fine suspended sediment concentration (SSC) and discharge $Q$ over $n$ time steps:

$$SSL_{fine} = \sum_{i=1}^{n} SSC_i \cdot Q_i \cdot \Delta t \quad [kg/s] \quad (4.15)$$

Between 1987 and 2016 the time step $\Delta t$ was usually 60 minutes. During SBT operations the time step was reduced to 15 minutes at the measuring locations outside of the SBT and to 1 minute inside the SBT.

The $SSL_{fine}$ is supplied by the HPP Tiefencastel, the Albula and the Julia River. The contribution of the latter is small due to upstream reservoirs releasing small discharges with low sediment concentrations. The $SSC$ in the Albula was determined by using turbidity data for the years 2011 to 2016. For earlier years, i.e. 1987 to 2010, the instantaneous $SSC$ was determined based on a $SSC$-discharge rating curve applied to the hydrograph because no turbidity data were available (except for some measurements in 1993). The $SSL_{fine}$ of the HPP Tiefencastel is not monitored but estimated as explained afterwards.

The calibration of the turbidimeter, two different $SSC$-discharge rating curves, and the estimation of the $SSL_{fine}$ at the HPP Tiefencastel are presented and discussed below.

**Calibration of the turbidimeter in the Albula**

The transport of fine suspended sediment in the Albula was monitored by a turbidimeter. To convert turbidity ($Tr$) to $SSC$ data, the turbidimeter was calibrated using 151 representative bottle samples ($SSCB$) from 2010 and 2011, of which 5 data points were skipped due to measurement error beyond 350 FNU (cf. Chapter 4.3.4). Application of the least squares fitting resulted in $SSCB = 1.9 \cdot Tr$ with $R^2 = 0.63$ (Figure 4.19). This is in agreement with theory suggesting a linear correlation between $SSC$ and turbidity, provided constant particle properties such as i.e. particle composition, surface roughness, size and shape (Walling 1977, Campbell and Spinrad 1987, Black and Rosenberg 1994, Gippel 1995, Spreafico et al. 2005, Haimann et al. 2014, Felix 2017). However, the sediment properties, in particular the particle sizes, are not constant and affect the turbidimeter calibration. The particle size typically increases with increasing discharge (Black and Rosenberg 1994). Based on field investigations, Teixeira et al. (2016) proposed to account for the particle size effect by applying a power fit instead of a linear fit. Application of a power fit to the Albula data leads to $SSCB = 0.0704 \cdot Tr^{1.23}$ with a considerably lower correlation coefficient of $R^2 = 0.37$ and hence was not applied in this study (Figure 4.19)
The specific turbidity i.e. \( Tr/SSC \), of the Albula is 0.53 FNU/(mg/l), which is higher than that of Felix (2017). He tested the same device using various suspensions made of water and different mineral materials in the laboratory and obtained specific turbidities up to 0.35 FNU/(mg/l). The differences occur due to (I) different particle properties, i.e. shape, size, color, surface structure, (II) deviating hydraulic conditions and (II) different device-specific factory settings of the turbidimeters.

Figure 4.19 shows that the data largely scatter indicating some degree of uncertainty in determining SSC and hence SSL\(_{fine}\). The standard deviation is \( \sigma = 84 \) mg/l. Among the data points, 92% and 96% lie within \( \pm \sigma \) and \( \pm 2\sigma \), respectively. However, the absolute standard deviation does not represent the data scatter for low SSC, in contrast to the relative deviation \( \sigma_r \). Given that 68% of the data points lay within the relative deviation, corresponding to the share of \( \pm \sigma \) of the Gaussian distribution, the data scatter is less than 80% from the mean and can be described by \( \pm \sigma_r = \pm 0.80 \cdot SSC_B \) as shown in Figure 4.19. This relation describes the deviation over the whole \( SSC_B \) range and therefore is assumed to represent the error of SSL\(_{fine}\) estimation.

**Figure 4.19** Linear \( SSC_B \)-turbidity correlation used for the calibration of the turbidimeter in the Albula in Tiefencastel obtained from bottle sampling and power law relation proposed by Teixeira *et al.* (2016)

**\( SSC \)-discharge rating curve in the Albula**

Two different \( SSC \)-discharge (\( SSC-Q \)) rating curves were established: (I) \( SSC \)-discharge correlation based on bottle samples (\( SSC_B \)-\( Q \)) and (II) \( SSC \)-discharge correlation based on the calibrated turbidity measurements (\( SSC_{Tr} \)-\( Q \)).

Figure 4.20 shows the \( SSC_B \) as a function of discharge of 274 bottle samples collected in 1993, 2010 and 2011 in the Albula, among which 169 data points were skipped because of error in
discharge measurements. By applying least square fitting to the remaining 105 data points using a power fit (Chapter 3.2.7), $SSC_B = 0.15 \cdot Q^{1.702}$ with $R^2 = 0.49$ was obtained. The standard deviation is $\sigma = 34 \text{ mg/l}$, and 88% and 94% of the data points lie within $\pm \sigma$ and $\pm 2\sigma$, respectively. The relative deviation involving more than 68% of the data points, corresponding to the share of $\pm \sigma$ of the Gaussian distribution, equals to $\sigma_r = \pm 0.75 \cdot SSC_B$ (Figure 4.20). This relationship was assumed to represent the error of $SSL_{fine}$ estimation.

Figure 4.20  $SSC_B$-$Q$ correlation in the Albula in Tiefencastel obtained from bottle sampling

The $SSC_T$-$Q$ correlation was determined based on hourly turbidity and discharge measurements in the Albula between 2012 and 2016. Figure 4.21 shows the $SSC_T$ determined based on the calibrated turbidity measurements as a function of the discharge in the Albula. The least square fit led to $SSC_T = 4.30 \cdot Q^{1.12}$ with $R^2 = 0.23$. The standard deviation is $\sigma = 411 \text{ mg/l}$ and over 93% lie within $\pm \sigma$. The relative deviation involving more than 68% of the data points, corresponding to the share of $\pm \sigma$ of the Gaussian distribution, equals to $\sigma_r = \pm 0.86 \cdot SSC_B$ (Figure 4.21). Accordingly, the estimation error of $SSL_{fine}$ was assumed to amount to less than $\pm 86\%$.

The data scatter is higher for the $SSC_T$-$Q$ correlation compared to the $SSC_B$-$Q$ correlation (Figure 4.20) due to the large data set with more than 42’000 data points covering a considerable larger $SSC$ and $Q$ data range. Moreover, the $SSC_T$-$Q$ correlation is based on continuous multiyear measurements in contrast to the $SSC_B$-$Q$ correlation based on discrete point measurements. Therefore, application of $SSC_T$-$Q$ is expected to result in more realistic $SSC_{fine}$ estimations. Despite this, both rating curves are applied to the hydrograph data and their results are discussed in Chapter 6.3.1.
No monitoring of the SSC in the water released by the HPP Tiefencastel exists. Therefore, the SSC\textsubscript{fine} time series of the HPP Tiefencastel was assumed to be equal to that of the HPP Sils (cf. Chapter 4.3.9). The reasons for that are: (I) both HPPs are in the same catchment and are subsequent HPPs of the same HPP cascade and (II) both HPPs are fed by reservoirs assumed to exhibit similar desilting characteristics.

### 4.3.9 Fine suspended sediment transport out of the Solis Reservoir

The fine suspended sediment is released by different outlet structures from the reservoir, i.e. the SBT, the headraces of HPP Sils and Rothenbrunnen, the bottom outlet, the spillway and the environmental flow. The sediment load of the latter is insignificant and therefore was neglected. The SSC and hence SSL\textsubscript{fine} estimations of the other outlet structures are explained in the following.

#### SOLIS SBT

The concentration of fine suspended sediment was continuously monitored by two turbidimeters installed in the Solis SBT. The turbidity time series was converted to a time series of suspended sediment concentration using the calibration obtained from the Albula measurements (cf. Figure 4.19 in Chapter 4.3.8) since the turbidimeters at both locations are of equal type. The suspended sediment concentration profile was determined based on the point measurement assuming the Rouse’s (1937) profile (Equation (3.52)). The mean SSC was obtained by integrating the local suspended sediment concentration over flow depth respecting
the flow velocity profile (Equation (3.6)). Application of Equation (3.51) resulted in the transport rate of suspended sediment $SSR$, the integration of which over time yields to the suspended sediment mass load $SSL$.

**HPP SILS**

The turbidity was continuously measured in the tailrace channel of the HPP Sils by a turbidimeter. To convert turbidity data to $SSC$ data, the turbidimeter was calibrated using 80 bottle samples collected between 2010 and 2016. Figure 4.22a shows the $SSC$ as a function of turbidity. The least square fit resulted in $SSC = 0.70 \cdot Tr$ with $R^2 = 0.77$. The standard deviation is $\sigma = 37$ mg/l. However, the absolute standard deviation does not represent the data scatter for low $SSC$. The relative deviation involving more than 68\% of the data points, corresponding to the share of $\pm \sigma$ of the Gaussian distribution, equals to $\sigma_r = \pm 0.27 \cdot SSC_B$ (Figure 4.22a). This relation describes the deviation over the whole $SSC_B$ range and therefore is used in the data analysis. The estimation error of $SSL_{\text{fine}}$ was accordingly assumed to amount to less than $\pm 27\%$.

This $SSC_B-Tr$ calibration deviates from those determined in the Albula (cf. Figure 4.19 in Chapter 4.3.8) with a 2.7 times smaller gradient and a significantly smaller deviation. This is because of the desilting effect of the Solis Reservoir resulting in smaller particle sizes and a narrower GSD.

The annual average $SSC_{Tr}$ at the HPP Sils slightly varies with $\pm 13\%$ around 25.9 mg/l as shown in Figure 4.22b. This average value was used to estimate the $SSL_{\text{fine}}$ at the HPP Sils for prior years when no turbidity measurements existed.

![Figure 4.22](image)

**HPP ROTHENBRUNNEN**

No $SSC$ monitoring exists at the HPP Rothenbrunnen. Since the intakes of both HPPs Sils and Rothenbrunnen are at the same location, their $SSC$ time series were assumed to be identical.
SPILLWAY AND BOTTOM OUTLETS

No SSC and turbidity data from the spillway and the bottom outlet were available. The SSC time series of the bottom outlet and the spillway were assumed to be similar to the SSC time series in the Albula (cf. Chapter 4.3.8) with a reduced amplitude due to the desilting effect of the Solis Reservoir. A linear dilution profile over the water depth was assumed at the dam (Figure 4.23). The desilting factor $DF$ at the HPP intakes ($z = 35$ m above bottom outlet) defined as the ratio between fine suspended sediment concentration in the Albula and the HPPs ($DF = \frac{SSC_{Tr, Albula}}{SSC_{Tr, HPP Sils}}$) was 9.3 based on turbidity measurements. A desilting factor of 6 was assumed at the bottom outlet ($z = 1.2$ m) based on the observations at the Mapragg Reservoir, located just 30 km north of Solis and exhibiting a similar shape, size, depth and capacity of $5.3 \times 10^6$ m$^3$ (Müller and De Cesare 2009). Application of a linear fit to these two desilting factors resulted in $DF = 5.9 + 0.1 \cdot z$. A desilting of $DF = 10.0$ was obtained for the spillway ($z = 42$ m) using this relationship. The SSC time series in the bottom outlet and the spillway were calculated by applying these desilting factors to the SSC time series in the Albula (cf. Chapter 4.3.8).

![Assumed desilting profile over depth at the Solis Dam](image)

**Figure 4.23** Assumed desilting profile over depth at the Solis Dam

### 4.3.10 Coarse suspended sediment into and out of the Solis Reservoir

The coarse suspended sediment inflow into the Solis Reservoir originates from the Albula and the Julia Rivers. Since the Julia River is strongly affected by upstream reservoirs, its minor contribution was neglected and only the $SSL_{coarse}$ of the Albula was considered in this study. The outflow of coarse suspended sediment is released by the SBT and the bottom outlet. However, significant volumes were only expected in the SBT, because the share of coarse particles in the accumulation body decreases from the reservoir head towards the dam (Zarn
Only a small fraction of the coarse particles settling close to the bottom outlet intake can be sluiced.

The transport of coarse suspended sediment in the SBT is neither detectable by the SPGSs nor by the turbidimeters. Therefore, the estimation of the $SSL_{coarse}$ in the Albula and the SBT is explained as follows.

In the Albula the ratio of bedload to total load defined as $k_{BL22} = BL_{22}/TL$ was assumed to be constant. The $SSL_{coarse}$ was computed using the following equation with the determined $BL_{22}$ and $SSL_{fine}$ (cf. Chapters 4.3.6 and 4.3.8):

$$SSL_{coarse} = TL - SSL_{fine} - BL_{22} = (1/k_{BL22} - 1)BL_{22} - SSL_{fine} \quad [m^3] \quad (4.16)$$

In the SBT, the $SSL_{coarse}$ was determined based on the size class ratios $SSL_{coarse}/SSL_{fine}$ and $SSL_{coarse}/BL_{22}$. The ratios were determined from the data obtained during SBT operation on May 15, 2015 (Chapter 6.4.1.). This operation served as a reference (denoted with an asterisk) because, it was the only SBT operation during which bedload transport was measured in both the Albula and the SBT. Using the determined $BL_{22}$ and $SSL_{fine}$ values (Chapters 4.3.7 and 4.3.9), the $SSL_{coarse}$ can be calculated by using the following two equations:

$$SSL_{coarse} = BL_{22} \left( \frac{SSL_{coarse}}{BL_{22}} \right) \quad [m^3] \quad (4.17)$$

$$SSL_{coarse} = SSL_{fine} \left( \frac{SSL_{coarse}}{SSL_{fine}} \right) \quad [m^3] \quad (4.18)$$

The GSD and hence the size class ratios of the sediment in the SBT vary from operation to operation. The average of Equation (4.17) and (4.18) is used in this study to account for this variation.

### 4.4 Test case Pfaffensprung SBT

The test setup of the Pfaffensprung SBT is presented herein. After general information on the river including the sediment properties, the reservoir and the SBT are described in detail. Then the properties of the tested invert materials are given followed by the description of the instruments used and procedure applied to determine the abrasion depths.
4.4.1 Reuss River

The Reuss drains a catchment area of 390 km² elevated between 801 and 3623 masl, of which 11% is covered by glaciers (GEWISS 2014). The mean annual runoff at the Pfaffensprung SBT is 20.5 m³/s, and the one, ten and 50 year flood discharges are 220 m³/s, 360 m³/s and 460 m³/s, respectively (VAW 1992). The topography of the Reuss in the vicinity of the SBT was investigated based on aerial photos and elevation maps provided by map.geo.admin.ch (Appendix F.1). Based on these data, the determined river bed slope and channel width were $S = 0.0374$ and $b = 18$ m, respectively, which are in agreement with former investigations (VAW 1988, 1992).

The bed material of the Reuss at Pfaffensprung consists of gneiss, slate and granite. The latter is dominant and contains 20 - 40% of quartz. Information about the GSD of the bed material is provided by VAW (1992), Vischer et al. (1997), Awazu (2015) and Wolgemuth (2015). The characteristic particle sizes are listed in Table 4.9. Field observations revealed that the $d_{90}$ value provided by both VAW (1992) and Vischer et al. (1997) are not representative. The most recent analysis conducted by Awazu (2015) and Wolgemuth (2015) are expected to provide the most reliable information with regard to the present conditions. The latter determined the GSD based on areal pictures analyzed by BASEGRAIN (more information are provided in Appendix F.2). His results are more or less in agreement with VAW (1992). Awazu (2015) investigated only the particles smaller than 0.75 m resulting in a bias towards smaller grain sizes. Therefore his values were not included in this study. Finally the GSD used in this study was determined based on the gray marked values of Table 4.9.

Table 4.9 Grain size information about the gravel bed material of the Reuss and values used for bedload and abrasion analysis

<table>
<thead>
<tr>
<th>Available data</th>
<th>Used values</th>
</tr>
</thead>
<tbody>
<tr>
<td>VAW (1992)</td>
<td>VAW (1992)</td>
</tr>
<tr>
<td>Vischer et al. (1997)</td>
<td>Vischer et al. (1997)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$d_{30}$ [m]</td>
<td>0.10</td>
<td>No data</td>
<td>0.076</td>
<td>0.041</td>
<td>0.076</td>
</tr>
<tr>
<td>$d_{m}$ [m]</td>
<td>0.25 - 0.3</td>
<td>0.25</td>
<td>0.25</td>
<td>0.15</td>
<td>0.25</td>
</tr>
<tr>
<td>$d_{90}$ [m]</td>
<td>0.56</td>
<td>2.7</td>
<td>0.61</td>
<td>0.35</td>
<td>0.68</td>
</tr>
</tbody>
</table>
4.4.2 Pfaffensprung Reservoir and SBT

The Pfaffensprung Reservoir with the 282 m long SBT shown in Figure 4.24 was built in 1922 and hence was the first SBT in Switzerland. The reservoir impounds the Reuss and has a capacity of $0.17 \cdot 10^6$ m$^3$, which is relatively small compared to the mean annual runoff of $645 \cdot 10^6$ m$^3$. The reservoir belongs to a cascade of three hydropower plants operated by the Swiss Federal Railway (SBB) and is located in Wassen, Canton Uri, in the Swiss Alps. The SBT cross section is horse-shoe shaped and the longitudinal slope is 3%, except for the 25 m long acceleration section at the inlet, where the slope is 35%. In this transition zone the discharge is accelerated, and the tunnel width decreases from 11.5 m at the intake to 4.7 m, corresponding to the normal profile width.

![Figure 4.24 a) Plan view, b) cross section and c) longitudinal profile of the Pfaffensprung SBT](image)

After the acceleration section and the following right side bend (radius $R \approx 140$ m, angle $\alpha \approx 40^\circ$) the tunnel axis is straight until the outlet. The design discharge is 220 m$^3$/s and corresponds to a one year flood event, whereas the maximum discharge can reach up to 240 m$^3$/s (Müller and Walker 2015). The flow velocity depends on the discharge and varies from 6 up to 15 m/s.

Approach flow discharges smaller than $Q_0 = 40$ m$^3$/s are conveyed into the reservoir for hydropower generation purposes. At higher discharges the surplus inflow is bypassed around the reservoir to the downstream river for about 100 - 200 days per year on average. When the discharge exceeds the SBT capacity, the surplus discharge enters the reservoir and is released via the spillway and / or the bottom outlets. Suspended sediment is always carried with the flow and thus partially enters the reservoir, whereas bedload particles only enter the reservoir during
flood events with discharges exceeding the SBT capacity. As a result, reservoir sedimentation is almost completely avoided. Over the last decades, this operation regime has been proven to be appropriate to minimize the sediment input into the reservoir. The annually accumulated sediment volume of less than 6’000 m$^3$ evacuated once a year (SBB 2009, 2015) is very low compared to the estimated annual sediment supply of roughly 200’000 to 260’000 m$^3$ (Chapter 7.2.3). As a result, about 98% of the inflowing sediment were bypassed, and only 2% settled in the reservoir on average.

Initially, the invert was paved with 60 cm thick granite blocks (ca. 0.3 m × 0.5 m) and subjected to strong hydroabrasive wear every year (Müller and Walker 2015). The hydroabrasive impact of coarse sediment particles with high quartz content led to extensive abrasions of around 30 cm and 60 cm after 75 years (Vischer et al. 1997) along the tunnel and the inlet section, respectively (Müller and Walker 2015). Despite significant material loss in the range of half of the initial lining thickness, the blocks themselves stayed stable in place. Due to secondary currents induced by the right bend, the bedload transport was concentrated along the inner side of the curve, leading to a corresponding abrasion path (SBZ 1943). In the following, different linings such as larger granite blocks, different concretes, cast basalt, steel and wood were tested, but none of the materials met the target abrasion resistance (Vischer et al. 1997). Therefore, regular refurbishments were executed during low flows in winter seasons using various materials. Examples of the invert conditions of differently lined sections after repeated abrasion and repair are shown in Figure 4.25.

Based on the preliminary results gained during the present study, the operator of the SBT decided to replace the invert lining with Urner Granite. During refurbishment works performed in the winters 2013/14, 2014/15, 2015/16 and 2016/17 the whole invert, except the test fields described hereafter, was successively replaced.
4.4.3 Test setup and devices

Four test fields were implemented at the Pfaffensprung SBT during the refurbishment works in the low flow winter seasons 2011/12 and 2012/13. Two 4.4 m wide and 10 m long test fields were installed at the outlet of the tunnel (location 1 in Figure 4.26) and two other 4.4 m wide and 20 m long test fields were installed in the curvature section of the tunnel (location 2 in Figure 4.26). To avoid abrasion propagation from one field to the other, all test fields were separated by steel beams. The test fields at location 1 still exist, whereas the test fields at location 2 were removed after two years of operation and replaced by Urner Granite.

Both test locations contained a granite and a concrete test field. The granite test fields consist of Urner Granite locally mined at Gurtnellen and is known for its high quality and high compressive strength of $f_c = 260$ N/mm$^2$. The different layers of the granite pavement are shown in Figure 4.27a. The granite pavement consists of granite blocks placed on a bedding mortar, and a drainage layer made of concrete with $d = 8 - 16$ mm. The 30 cm thick and 1 m$^2$ large granite blocks were placed in a staggered order without joints (accuracy $\pm 2$ mm) and physically attached to the tunnel bottom (Figure 4.27b and Figure 4.28). The blocks were uniquely shaped to follow the tunnel geometry without gaps between the blocks. The gaps between the walls and the granite pavement were filled with cast basalt plates and an abrasion-resistant mortar (basalt sand and gravel of $d = 0 - 5$ mm with 500 kg cement/m$^3$, cement type “FONDU”) or abrasion-resistant mortar only. Removal of the blocks by suction or uplift is avoided by both the tight and splined block placement and the high weight of 690 to 860 kg per block.

![Figure 4.26](image_url) Overview of the Pfaffensprung facility with reservoir, dam, guiding structure and SBT with test areas 1 and 2 and discharge measurement devices
The other test fields were equipped with a 30 cm thick lining of high-strength concrete. While the concrete at location 1, i.e. concrete C1, contains steel fibers, the concrete at location 2, i.e. concrete C2, is identical to C1 except for steel fibers (Table 4.10). The concrete was produced at a local concrete plant “Mattli Beton AG” in Wassen, Switzerland, and transported by a truck mixer to the intake of the SBT. After dumping and leveling, the concrete was compacted and flattened by a poker vibrator and a vibrating beam. For quality control purposes, concrete samples were produced and tested at an age of 28 days in the laboratory of “TFB AG” in Wildegg Switzerland. The measured compressive strength $f_c$ (determined at cubes $15 \times 15 \times 15$ cm) and bending tensile strength $f_{bt}$ (determined at bars $12 \times 12 \times 36$ cm) are listed in Table 4.11 together with the calculated splitting tensile strength $f_{st}$ (Equation (4.10) in Chapter 4.1.6) and the Young’s moduli $Y_M$ (Equation (4.9) in Chapter 4.1.6).
Table 4.10 Concrete mixtures of the test fields in the Pfaffensprung SBT

<table>
<thead>
<tr>
<th>Composite [kg/m³]</th>
<th>Concrete 1 (C1)</th>
<th>Concrete 2 (C2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEM II/A-D 52.5 R</td>
<td>550</td>
<td>550</td>
</tr>
<tr>
<td>Water-cement ratio</td>
<td>0.30</td>
<td>0.28</td>
</tr>
<tr>
<td>Steel fibers RC-80/30-BP</td>
<td>40</td>
<td>-</td>
</tr>
<tr>
<td>Sand 0/4 mm</td>
<td>600</td>
<td>600</td>
</tr>
<tr>
<td>Gravel 4/8 mm</td>
<td>429</td>
<td>429</td>
</tr>
<tr>
<td>Gravel 8/11 mm</td>
<td>257</td>
<td>257</td>
</tr>
<tr>
<td>Gravel 11/16 mm</td>
<td>429</td>
<td>429</td>
</tr>
<tr>
<td>Superplasticizer</td>
<td>14</td>
<td>14</td>
</tr>
</tbody>
</table>

Table 4.11 Properties of the implemented materials in the Pfaffensprung SBT

<table>
<thead>
<tr>
<th></th>
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<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete 1 (2011/12)</td>
<td>C1</td>
<td>30</td>
<td>2460 ± 30</td>
<td>108 ± 2</td>
<td>15.2 ± 0.4</td>
<td>11.3 ± 0.3</td>
<td>38.6+</td>
</tr>
<tr>
<td>Concrete 2 (2012/13)</td>
<td>C2</td>
<td>30</td>
<td>2455 ± 45</td>
<td>78 ± 21</td>
<td>15.1 ± 1.6</td>
<td>11.2 ± 1.1</td>
<td>34.6+</td>
</tr>
<tr>
<td>Granite 1 (2011/12)</td>
<td>G1</td>
<td>30</td>
<td>2650++</td>
<td>260 ± 20</td>
<td>10 ± 2++</td>
<td></td>
<td>59.0+</td>
</tr>
<tr>
<td>Granite 2 (2012/13)</td>
<td>G2</td>
<td>30</td>
<td>2650++</td>
<td>260 ± 20</td>
<td>10 ± 2++</td>
<td></td>
<td>59.0+</td>
</tr>
</tbody>
</table>

*calculated using Equations (4.10), **large standard deviation because of bad water quality used for concrete production, †calculated using Equation (4.8) and (4.9), ‡provided by the quarry.

The compressive strength of the test field C1 was additionally measured in-situ by a Schmidt Hammer (*DIGI-SCHMIDT 2000*, produced by *proceq*, Schwerzenbach, Switzerland) on the abraded surface in March 2016 (Figure 4.29). The Schmidt Hammer is a non-destructive, easily in-situ applicable and fast instrument to immediately determine the compressive strength (SIA 2009). The compressive strength is determined via the rebound energy. A drawback of this technique is that the result is only an indication of about the first 30 mm of the concrete depth and is affected by moisture content, surface texture and roughness, type of aggregate and hammer inclination (Williams and Robinson 1983, Qasrawi 2000). Moreover, the measurement accuracy is strongly affected by the surface roughness due to the reduced contact area between hammer head and material (Williams and Robinson 1983). Measurements at sawn and weathered rock surfaces resulted in an underestimation of compression strength by 30 - 50% and 80%, respectively since weakening of the surface leads to partial energy dissipation.

The measurement raster was 1 m × 1 m. The hammer was vertically orientated and the measurements were taken from a humid but not wet surface. At least 5 measurements were conducted at each measurement point. The standard deviation of the measurements was
± 2.8 N/mm². The averaged compressive strengths largely varied from one point to the other point. The measured values varied from $f_c = 34$ N/mm² to $f_c = 112$ N/mm², and 75% of the measurements were between 70 and 100 N/mm². The mean compressive strength was $f_{c,m} = 76$ N/mm², which is 42% lower than the compressive strengths of samples taken during the construction and tested in the laboratory (cf. Table 4.11). This deviation is attributed to the suboptimal field conditions during the Schmidt Hammer measurements, namely the rough surface and the structural weakening due to hydroabrasion.

Figure 4.29 Compressive strength [MPa] of the C1 test field determined by means of the Schmidt Hammer three years after implementation

The discharges in the SBT and of the Reuss were continuously monitored by radars (VEGA Messtechnik AG, Pfäffikon, Switzerland) installed within the SBT near the outlet (VEGAPULS 54K) and at the reservoir head (VEGAPULS 51K) as shown in Figure 4.26.

The material loss of the test fields was quantified using a 3D-Laser scanner (Zoller + Fröhlich Imager 5006, manufactured by Zoller + Fröhlich, Wangen in Allgäu, Germany, Figure 4.30). The laser point has a diameter of 3 mm in a distance of 1 m and exhibits a beam divergence of 0.22 mrad. At a distance of 10 m from the laser, the root mean square deviation of the measurements is 0.7 mm, which decreases with decreasing distances. The distance between the laser scanner positions varied between 5 and 7 m, so that the resulting root mean square deviation is below 0.7 mm. The target-based scan registration caused an additional error of $2.2 ± 0.4$ mm (Jenny 2016). The resulting error of each single point of the abrasion map is ± 3 mm in space and ± 2 mm in vertical direction. The error of the spatially averaged abrasion depths is even smaller due to averaging.

Prior to the scans, the invert surface was prepared, i.e. cleaned by a broom and kept free from ground water by installing a temporary drainage. For the latter, the water was ponded at the top end of the test fields and routed within plastic pipes along the tunnel wall. Despite this, certain
areas were not available for the analysis, since they were either covered by the drainage or by seeped water. The scans were performed with a resolution of 0.036° in vertical and horizontal direction, i.e. a resolution of 1.5 mm near the laser linearly decreasing with increasing distance. Each scan generated a cloud of 100'000'000 measurement points, being the limit for data analysis. To improve the accuracy of the scan registration, the targets were additionally scanned with the highest available resolution of 0.009° in horizontal and vertical direction.

Figure 4.30 Measurement installation at test section 1 during the laser scan in March 2014 with (1) the scanner, (2) the temporary mounted targets and (3) the ground water bypass (view in flow direction)

The surfaces of the test fields were initially scanned after the installation. The subsequent scans were taken every winter during the low flow season and allowed investigations of the development of the abrasion pattern and abrasion rates at the test fields. The total observation period for test site 1 and 2 was 4 and 2 years, respectively (Table 4.12). At every time step, the invert was recorded by several overlapping scans, which were registered based on 6 to 8 targets using the software “Z + F LaserControl”. The single scans were combined based on least square image matching and generated a dense set of three-dimensional point measurements of the surface, i.e. a 3D point cloud. This was done by using the software “Geomagic Studio”. Finally, the vertical differences between the measurement points and the reference surface, generated based on the reference point cloud, was computed with “Geomagic Qualify”, resulting in a high-resolution abrasion map.

For abrasion analysis the mean abrasion depths were computed by spatially averaging the obtained abrasion depths. However, regarding the design life of the invert materials and maintenance, the spots of maximum abrasion depths are of particular interest. Assuming that
the 95%-percentile abrasion value is decisive for refurbishment, this value was determined and is denoted herein as maximum abrasion depth.

Table 4.12 Dates of laser scan measurements and corresponding observation durations

<table>
<thead>
<tr>
<th>Date</th>
<th>Concrete 1</th>
<th>Granite 1</th>
<th>Concrete 2</th>
<th>Granite 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Winter 2011/12</td>
<td>28.3.2012</td>
<td>Initial</td>
<td>Initial</td>
<td>-</td>
</tr>
<tr>
<td>Winter 2012/13</td>
<td>18.12.2012</td>
<td>1 year</td>
<td>1 year</td>
<td>-</td>
</tr>
<tr>
<td>Winter 2012/13</td>
<td>26.3.2013</td>
<td>-</td>
<td>Initial</td>
<td>Initial</td>
</tr>
<tr>
<td>Winter 2013/14</td>
<td>26.2.2014</td>
<td>2 years</td>
<td>2 years</td>
<td>1 year</td>
</tr>
<tr>
<td>Winter 2014/15</td>
<td>17.3.2015</td>
<td>3 years</td>
<td>3 years</td>
<td>2 years</td>
</tr>
<tr>
<td>Winter 2015/16</td>
<td>3.5.2016*</td>
<td>4 years</td>
<td>4 years</td>
<td>-</td>
</tr>
</tbody>
</table>

* The SBT was not in operation for the first 4 months of 2016, hence these measurements represent the abrasions of 2015

4.5 Test case Runcahez SBT

The test setup of the Runcahez SBT is presented herein. After general information on the river including hydrograph data and sediment properties, the SBT is described in detail. Then, the properties of the tested invert materials are provided, followed by the description of the instruments used and procedure applied to determine the abrasion depths.

4.5.1 Rein da Sumvitg River

The total catchment area of the Runcahez Reservoir spans 270 km², while the direct drainage area is 55.6 km² and is located between 1276 and 3164 masl. 10.7% of the area is covered by glaciers and the mean annual runoff is 2.3 m³/s, while the discharges for the 1 and 100 year flood events in the Rein da Sumvitg were estimated to be 40 and 160 m³/s, respectively (Axpo 2011). The river slope in the vicinity of the SBT is $S = 0.0365$ and the mean river width $b = 15$ m (Appendix G.1). These values were determined based on aerial photos and elevation maps (method described in Chapter 4.1.1) and are in good agreement with the values from a former investigation (Jacobs et al. 2001).

No discharge measurements are available from the Runcahez Reservoir, SBT or the Rein da Sumvitg at Runcahez. Therefore, the hydrographs for the river at Runcahez were determined by scaling the measurements of the neighboring FOEN gauging station at Encardens (Station Number 2430) situated 3.5 km upstream of the reservoir as follows:
\[ Q_0 = \text{const} \cdot Q^* \quad \text{[m}^3/\text{s}] \]  

where \( Q_0 \) = approach flow discharge in the Rein da Sumvitg at Runcahez and \( Q^* \) = reference discharge (herein of the Rein da Sumvitg at the gauging station at Encardens). The scaling constant follows from either:

(I) the ratio of the catchment areas of the river at Runcahez \( A \) and at Encardens \( A^* \):

\[ \text{const} = \frac{A}{A^*} = \frac{56 \text{ km}^2}{21.8 \text{ km}^2} = 2.6 \quad [-] \]  

(II) or from the relation between the reference discharges at Encardens \( Q_i^* \) and at Runcahez \( Q_i \):

\[ \text{const} = \frac{Q_i^*}{Q_i} \quad [-] \]  

The sediment in the Rein da Sumvitg originates from the Gotthard massif, and contains 15 - 40% quartz (Weber 1970, Weber et al. 1970, Zulauf 1993). The GSD of the sediment is shown in Figure 4.31. The mean grain size used for bedload estimation is \( d_m = 0.23 \text{ m} \) and the particle uniformity coefficient is \( \sigma = (d_{84}/d_{16})^{0.5} = 4.5 \), respectively.

<table>
<thead>
<tr>
<th>( d )</th>
<th>[m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>( d_{\text{max}} )</td>
<td>1.20</td>
</tr>
<tr>
<td>( d_{90} )</td>
<td>0.53</td>
</tr>
<tr>
<td>( d_{84} )</td>
<td>0.40</td>
</tr>
<tr>
<td>( d_m )</td>
<td>0.23</td>
</tr>
<tr>
<td>( d_{50} )</td>
<td>0.17</td>
</tr>
<tr>
<td>( d_{30} )</td>
<td>0.06</td>
</tr>
<tr>
<td>( d_{16} )</td>
<td>0.02</td>
</tr>
</tbody>
</table>

Figure 4.31  GSD of the Rein da Sumvitg (adopted from Jacobs et al. (2001))

### 4.5.2 Runcahez Reservoir and SBT

The Runcahez dam was built in 1962 and its reservoir serves as a compensation basin between the hydropower plants Sedrun and Tavanasa of the Vorderrhein power scheme located in the Eastern Alps of Switzerland (Vischer et al. 1997). The original reservoir storage capacity is \( 0.44 \cdot 10^6 \text{ m}^3 \), which is 165 times smaller than the mean annual runoff of its direct catchment.
area with $72.5 \times 10^6$ m$^3$/a (GEWISS 2014). A 572 m long SBT was constructed and put in operation in 1962 in order to prevent reservoir sedimentation. Figure 4.32 shows its plan view and cross section. The archway-shaped cross section of the SBT is 3.8 m wide and 4.5 m high. Originally the invert shape was concave, partially replaced by a plane invert during several refurbishment works. An 80 m long transition section follows the intake, which is equipped with a radial gate. This section includes a 65 m long acceleration section with a maximum inclination of 25% and a cross section narrowing from 7 m at the inlet to 3.8 m corresponding to the regular tunnel width. The regular bottom slope and the design discharge are $S = 0.014$ and $Q_d = 110$ m$^3$/s, respectively. At approach flows exceeding the design discharge capacity of the SBT, the SBT can still be operated. However, the projected free-surface inflow conditions change to pressurized inflow conditions resulting in a higher discharge in the SBT of 190 m$^3$/s (Axpo 2011). In the SBT, supercritical free-surface flow conditions occur and velocity varies between 5 and 16 m/s, depending on discharge.

The SBT is used for bypassing sediment-laden discharges during flood events. If the discharge in the river exceeds 35 m$^3$/s, the SBT is put in operation and diverts the whole inflow around the reservoir (Jacobs et al. 2001). However, due to strong discharge fluctuations, which are typical for such a small and steep Alpine catchment, this operation threshold is not always achieved.

The tunnel lining in the inlet section was initially concrete ($f_c \approx 65$ MPa), while in the downstream part the excavated rock builds the invert, persisting well during the first decades. However, in 1987, 1993, 1995 and 2007 the SBT suffered from massive abrasion damages and required refurbishing. For the refurbishments cast basalt plates and standard concrete (standard concrete used for civil engineering purposes with estimated compressive strength around 35 N/mm$^2$) were used, which did not achieve the target performance (Wehrli 2014). As shown in Figure 4.33a, the cast basalt plates suffered from brittle fractures, were partially removed and represented spots of initial damages, from which a chain reaction of invert removal was activated (Jacobs and Hagmann 2015). Furthermore, the emergency refillings of scour holes
and incision channels with standard concrete led to rapidly re-developed incision channels along the center line of the straight section as shown in Figure 4.33b.

![Figure 4.33 Damages at the Runcahez SBT: a) removed cast basalt plates at the curve near the intake and b) incision channel at the concrete lined straight section (view in flow direction, both pictures were taken during the geodetic survey in fall 2014)](image)

### 4.5.3 Test setup

The test setup in the Runcahez SBT consists of 5 test fields, implemented in 1995 along the tunnel section after the acceleration section and bend (Figure 4.32a, Jacobs et al. 2001). Figure 4.34 gives an overview of the five 10 m long, 3.8 m wide and 30 cm thick test fields. The test fields were equipped with different concrete mixtures: a silica fume concrete (SC), a roller compacted concrete (RCC), a high performance concrete (HPC), a steel fiber concrete (SF) and a polymer concrete (PC). In order to prevent propagation of damages in flow direction to the subsequent test fields, the fields were separated by steel beams. The composition and the properties of the materials are listed in Table 4.13 and Table 4.14. Note that the roller compacted concrete field (RCC) suffered massive abrasion along the tunnel walls and required a replacement after Jacob’s (2001) investigation. Since no data about the replacement material are available, only data of the previously implemented RCC is provided.

![Figure 4.34 Sketch of the test fields implemented in Runcahez SBT](image)
The implemented test fields were geodetically surveyed using an analog leveling device before commissioning in 1996 (Jacobs et al. 2001). Afterwards, annual measurements were conducted for four years. The resolution and the accuracy of the measurements were 0.5 m × 0.2 m and ± 2 mm, respectively. The next survey was done by the author in fall 2014 by using a digital leveling device (Leica Sprinter 250M, manufactured by Leica Geosystems, Switzerland). The resolution of the present study measurement was 0.5 m × 0.4 m and the measurement accuracy was ±2 mm. The upstream first steel beam served as a reference. By comparing measurements conducted at different time stages, the abrasion depths were determined. Hence, the data set was enlarged to 19 years, which is, to the author’s knowledge, the first long-term in-situ monitoring of hydroabrasion under alpine conditions.

The abrasion measurement results need careful interpretation. During the first 4 years, the abrasion depths were in the range or close to the measurement accuracy of ±2 mm, resulting in implausible trends (decreasing abrasion with time) and large fluctuations. Furthermore, small abrasion depths only allow statements about the performance of the topmost and curing affected layers.
material. The near-surface material exhibits a different structure and composition compared to the core material and hence its performance is not representative. The time scale of hydroabrasion processes ranges from years to decades. Therefore, the results gained after 19 years within this time scale, were considered as most significant and were used for hydroabrasion analysis. For the RCC test field, which was replaced after 4 years, only the 4 years data from the properly compacted zone were used although its actual long-term behavior would likely differ.

### 4.6 Underwater bed load test

The invert materials of the Solis and Pfaffensprung SBTs were sampled, and tested by means of an underwater bedload drum to investigate the scalability of abrasion rates from laboratory to prototype scale and to evaluate the applicability of existing abrasion prediction models to the underwater bedload test. This subchapter presents the test setup and test procedure including the sample production and preparation of the underwater bedload test.

#### 4.6.1 Test setup

The hydroabrasion laboratory tests were conducted by Claudia Bellmann at the Institute of Construction Materials of the Technical University of Dresden (TUD), Germany, by means of the Dresdner Drum (cf. Chapter 3.4.1 and Mechtcherine et al. 2012). The drum was filled with a mixture of water and abrasive material consisting of 10 kg of water and 10 kg of hardened steel spheres with a mean grain size of 11.9 mm. Table 4.15 lists the diameters and masses of the spheres.

<table>
<thead>
<tr>
<th>d [mm]</th>
<th>3</th>
<th>4</th>
<th>6.9</th>
<th>7.9</th>
<th>10</th>
<th>14</th>
<th>16</th>
<th>24</th>
<th>32</th>
<th>Σ=10'020</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mass [g]</td>
<td>615</td>
<td>796</td>
<td>732</td>
<td>1'152</td>
<td>2'737</td>
<td>1'906</td>
<td>976</td>
<td>568</td>
<td>539</td>
<td>10'020</td>
</tr>
</tbody>
</table>

The drum was rotated 16.8 times per minute resulting in a mean flow velocity of 0.80 m/s. Each test had 60'000 rotations over 59 hours. Due to partial filling of the drum (Figure 3.8a) each sample was exposed to hydroabrasion during 9.2 hours. The mean water depth was 0.054 m. The sample weight and surface was regularly measured by means of a precision balance and a laser scanner, respectively, to detect the material loss and to analyze the hydroabrasion processes in space and time (Figure 4.35c).
4.6.2 Samples

Various invert materials of the Pfaffensprung and Solis SBTs were sampled and tested. For the former, plate samples (300 mm × 300 mm × 50 mm) were produced during the installation of the C2 concrete test field (Section 4.4.3). For the latter, drill core samples (d = 150 mm) were taken from each concrete test field one year after installation but still before the first SBT operation. Table 4.16 lists the tested invert materials and the sample geometries.

Table 4.16 Sample properties from the SBTs Pfaffensprung and Solis

<table>
<thead>
<tr>
<th>SBT</th>
<th>Material</th>
<th>Number of samples</th>
<th>Sample type</th>
<th>Sample geometry [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pfaffensprung</td>
<td>C2</td>
<td>4</td>
<td>Plate</td>
<td>300 × 300 × 50</td>
</tr>
<tr>
<td>Solis</td>
<td>NC</td>
<td>4</td>
<td>Drill core</td>
<td>150 × 300</td>
</tr>
<tr>
<td>Solis</td>
<td>HPC</td>
<td>4</td>
<td>Drill core</td>
<td>150 × 300</td>
</tr>
<tr>
<td>Solis</td>
<td>LSC</td>
<td>4</td>
<td>Drill core</td>
<td>150 × 300</td>
</tr>
<tr>
<td>Solis</td>
<td>HAC</td>
<td>4</td>
<td>Drill core</td>
<td>150 × 300</td>
</tr>
<tr>
<td>Solis</td>
<td>UHPC</td>
<td>4</td>
<td>Drill core</td>
<td>150 × 300</td>
</tr>
</tbody>
</table>

The drill core samples were specially prepared for the drum test due to their cylindrical shape. The procedure is shown in Figure 4.35 and followed as: (a) cutting a 50 mm thick slice at the uppermost part of the drill core sample, (b) embedding the slice in a dummy plate measuring 300 mm × 300 mm × 50 mm and (c) exposing the drill core samples to surface abrasion measurements using a template. The concrete mixture of the embedding material was identical to the sample material in order to avoid biased results.

![Figure 4.35](image-url)  
Drill core sample production and procedure for the Dresden Drum test: a) cutting a 50 mm thick slice from a drill core, b) embedding in a dummy plate, c) surface abrasion measurement with a template using a laser (courtesy of C. Bellmann)
4.7 Abrasion model calibration

The saltation abrasion models described in Chapter 3.5 include abrasion coefficients, i.e. either an abrasion coefficient or material parameters. These coefficients were revealed to depend on various invert material and sediment properties and hence to be site-dependent. Literature provides some values, however their application may yield to unrealistic estimates, due to deviating invert material or sediment properties (Auel et al. 2017d). Therefore, the abrasion models were calibrated by means of prototype experiments (Mueller-Hagmann et al. 2017a).

In the following, the approaches and assumptions made to calibrate the abrasion models, i.e. the saltation abrasion models of Sklar and Dietrich (2004) and Auel et al. (2017b) as well as the abrasion model of Ishibashi (1983), are reported.

The model input parameters, i.e. the hydraulic operating conditions, the sediment load and the spatially averaged abrasion depths for the Pfaffensprung, Runcahez and other SBTs, are listed in the Appendix H, Table H.1, Table H.4 and Table H.7, respectively.

4.7.1 Saltation abrasion models

The abrasion coefficient \( k_v \) of the SAM and SAMA was calibrated by means of prototype experiments (cf. Chapter 4.2, 4.4, 4.3) and literature data in this study. Therefore, the SAM (Equation (3.54)) and the SAMA (Equation (3.59)) were rewritten as follows:

\[
\text{SAM: } k_v = \frac{1}{A_s} 0.08 g (s-1) \frac{Y_M}{f_{st}^2} \cdot q_s \left(1-\frac{q_s}{q_s'}\right) \left(\frac{U_s}{V_s}\right)^{-1} \left[1-\left(\frac{U_s}{V_s}\right)\right]^{1.5} \quad [m/s]
\]

\[
\text{SAMA: } k_v = \frac{1}{A_s} \frac{Y_M}{f_{st}^2} \left(s-1\right) \frac{g}{230} q_s \left(1-\frac{q_s}{q_s'}\right) \quad [m/s]
\]

The abrasion rates were determined based on the spatially averaged abrasion depths divided by the operating duration, since instantaneous abrasion rate data were not available. For the Pfaffensprung SBT annual abrasion depths measurements were performed, so that annually averaged abrasion rates were used herein. The abrasion depths after the first three years in the Runcahez SBT were below 3 mm and significantly affected by measurements errors. Therefore, only the data obtained after 4 and 19 years were considered. The abrasion data from the Mud Mountain SBT consist of thickness measurements of the initial 25.4 mm thick steel lining (VAW 2016a). These measurements do not represent the actual abrasion depths at the SBT inlet due to a several decimeter deep incision channel. Therefore, this section was separated from
the rest to avoid a biased result. Moreover, the mean abrasion depths were spatially averaged for different SBT sections according to the tunnel plan view geometry: inlet bend, straight section and outlet bend (Table H.7 in Appendix H.3).

The bedload transport capacity and settling velocity in the SBT were determined using Equation (3.49) and (3.30), respectively. The critical Shields parameter for initiation of particle motion was $\theta_c = 0.005$. The Young’s modulus was either assumed to be constant, i.e. $Y_M = 50$ GPa following Sklar and Dietrich (2004), or was obtained from laboratory tests (either directly determined based on stress-strain tests or derived from measured material strength according to Equation (4.9) and (4.11)).

### 4.7.2 Ishibashi abrasion model

The abrasion model of Ishibashi (1983) consists of a deformation wear term accounting for the impacts of saltating particles, and a cutting wear term accounting for the grinding stresses (Chapter 3.5.3). The latter was found to lead to overestimated abrasion depths for brittle materials and was proposed to omit in order to increase the prediction accuracy (Auel et al. 2016b). Therefore, this term was skipped for rock, concrete and mortars, and Equation (3.60) was rewritten as follows:

\[
c_1 = \frac{A_{\eta}}{E_k} \quad [m^2/kg]
\]  
\[
(4.24)
\]

Abrasion of steel is dominated by cutting wear, whereas deformation wear is less pronounced due to the ductile material behavior (Chapter 3.3.3). Moreover, the friction work is more than an order of magnitude higher than the deformation work, so that the deformation wear plays a minor role and can be neglected. Re-writing Equation (3.60) hence results in:

\[
c_2 = \frac{A_{\eta}}{W_f} \quad [m^2/kg]
\]  
\[
(4.25)
\]

The model includes an auxiliary parameter accounting for the ratio of the Young’s moduli $Y_M$ to the Poisson’s ratio $\mu$ of both the invert material and the sediment. Due to a lack of data, $Y_M = 50$ GPa and $\mu = 0.265$ was assumed for the sediment (cf. Chapter 3.5.3). The Young’s moduli of the invert material are given in Chapter 4.4.3 and 4.5.3. The Poisson’s ratios were $\mu = 0.2$ for concrete / mortar / foam / soft rock (i.e. $f_s < 1$ MPa), $\mu = 0.265$ for rock and $\mu = 0.3$ for steel, respectively.
5 Results of the geophone calibration

The results of the laboratory and field geophone calibrations are presented in this chapter. The effects of particle size, sediment transport rate and flow velocity, and geophone plate inclination were investigated. The results of the calibrations were compared with literature and are discussed herein. Finally, the SPGS installed at the Solis SBT was calibrated based on these findings.

5.1 Results of the laboratory geophone calibration

This subchapter contains the results of the laboratory geophone calibration. The effects of particle size, sediment transport rate, flow velocity and geophone plate inclination were investigated and discussed. Application of the uniform sediment test results to the sediment mixtures revealed good correlations between estimated and effective sediment transport masses thereby confirming the calibration coefficients. The calibration coefficient of the SPGS installed at the Solis SBT was determined according to the Solis GSD and is in a good agreement with the calibration coefficient determined based on Rickenmann et al.’s (2013) SPGS data.

5.1.1 Uniform sediment test

Figure 5.1a shows the number of impulses created by single impinging particles (Test series I) versus the corresponding particle diameter. As expected, the number of impulses increases with particle size due to the increasing kinetic energy of the moving particle and follows Imp $\sim d^{3/2}$ with $R^2 = 0.972$. The deviation is less than $\pm 15\%$. The corresponding calibration coefficients defined as $K_b = \text{Imp}/m$ are listed in Table 5.1 and shown in Figure 5.1b as a function of $d$. In contrast to the number of impulses, $K_b$ does not follow a power law trend but is affected by several phenomena resulting in a complex correlation between $K_b$ and $d$. The detection grain size threshold varies between 22 and 36 mm, which is in line with literature reporting $d = 20 - 30$ mm (Morach 2011, Rickenmann et al. 2012, Wyss 2016b, Koshiba et al. 2017). The particle hop length of these particles was approx. $2/3$ of the geophone length, so that these particles impact the geophone plate 1.5 times on average. However, the impact energy of such small particles remains below the threshold for impulse counting and thus is not recorded (Figure 5.1b). The energy of the particle increases and amplifies the signal with increasing size.
Therefore, the calibration coefficient increases. However, with a further increase of particle size, the particle rolling probability (Equation (3.21)) increases, resulting in lower vertical kinetic energy and $K_b$ (Figure 5.1b). Note that the relative particle submergence, i.e. the ratio of particle diameter to water depth $d/h$, may also affect the particle motion and hence $K_b$. This effect was not studied herein and requires further investigations.

Figure 5.1b shows $K_b$ as a function of $d$ for different sediment supply rates, i.e. “single particle” (= Test Series I), “normal supply” (= Test Series II) and “pack supply” (= Test Series III). The $K_b$ values for the experiments with higher sediment supply rates (“normal supply” and “pack supply”) are significantly lower than those for the “single particle” experiments at corresponding grain sizes. The difference between normal and pack supply is insignificant. Although the bedload transport supply rate was 4 times higher for the “pack supply” than for “normal supply”, the sediment cluster blurs, and leads to similar sediment transport rates at the geophone (Table 4.2) and hence similar $K_b$-values.

As a result, $K_b$ not only depends on particle size but also on the bedload transport rate. This is attributed to particle impact overlapping and signal damping at $z_p > 0$ (Wyss 2016b, Koshiba et al. 2017). Figure 5.2 shows the $z_p$-values as a function of the specific bedload transport rate $q_s$. Although the $z_p$-values increase with increasing $q_s$, there is almost no difference between the $z_p$-values of two different sediment supply types “normal” and “pack” due to the similar $q_s$ at the geophone plate.
Table 5.1 Calibration coefficient depending on grain size and sediment supply rate

<table>
<thead>
<tr>
<th>$d$ [mm]</th>
<th>22</th>
<th>28</th>
<th>36</th>
<th>46</th>
<th>59</th>
<th>75</th>
<th>96</th>
<th>123</th>
<th>158</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_{b,\text{single}}$ [1/kg]</td>
<td>0.0</td>
<td>3.3</td>
<td>15.2</td>
<td>22.0</td>
<td>20.8</td>
<td>18.5</td>
<td>11.6</td>
<td>7.3</td>
<td>6.9</td>
</tr>
<tr>
<td>$K_{b,\text{normal}}$ [1/kg]</td>
<td>0.48</td>
<td>4.6</td>
<td>11.4</td>
<td>16.0</td>
<td>13.8</td>
<td>14.5</td>
<td>8.7</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$K_{b,\text{pack}}$ [1/kg]</td>
<td>-</td>
<td>-</td>
<td>11.8</td>
<td>-</td>
<td>14.9</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Figure 5.2 $z_p$ vs. specific gravimetric bedload rate for “normal” and “pack supply” depending on particle size

5.1.2 Sediment mixture tests

The applicability of the uniform sediment test results (Test Series I-III) to sediment mixtures (Test Series IV) is investigated and treated in this subsection. Laboratory experiments with two different sediment mixtures were conducted (see Chapter 4.2.2.1). The $K_b$ of each mixture was obtained from the weighted average values shown in Figure 5.1b. The resulting $K_b$ for the coarse and the fine mixture are $K_b = 11.3$ 1/kg and $K_b = 11.7$ 1/kg, respectively. These $K_b$-values were used to estimate the sediment transport mass for each of the 92 test runs. Figure 5.3a illustrates the estimated transported masses, i.e. the minimum, the 25% quartile, the median, the 75% quartile and the maximum value, and the weighted sample masses for three different sediment supplies. The fine mixture exhibits a lower variance than the coarse mixture. Figure 5.3b shows that the mean estimated and measured mass for the fine mixture collapse well, whereas they deviate by 20% for the coarse mixture. Both the higher variance and the larger estimation error of the coarse mixture (where one large particle of 1024 gr represents 30% of the sample mass) may be attributed to the relatively low number of particles. This may not be representative but biased in terms of particle size shape, transport mode and hence $K_b$. Increasing the number of
Results of the geophone calibration

Sediment particles may improve the estimation accuracy. This assumption should be confirmed with further investigations.

Figure 5.3  a) Estimated and measured sediment mass for the coarse and fine mixture, respectively, and b) mean estimated vs. measured sediment mass for the coarse and fine mixture

5.1.3 Solis SPGS calibration

The calibration coefficient assuming the Solis GSD (Figure 4.9) obtained from the laboratory experiments at mean flow velocity of $U = 3$ m/s (Figure 5.1b) resulted in $K_b = 10.1$ l/kg. This value is in agreement with the data of Rickenmann et al. (2013) presented in Figure 5.4. However, as shown in Figure 5.4, $K_b$ significantly decreases with increasing flow velocity. This is due to increasing particle hop length causing fewer particle impacts on the geophone plate. A best fit is given by $K_b = 39.6U^{-1.41}$ with $R^2 = 0.58$. $K_b = 1.34$ l/kg was obtained by applying this equation to the mean flow velocity in the Solis SBT with $U \sim 11$ m/s (Figure 5.4). However, this value is questionable due to the unknown GSD of the bedload transport in the SBT and differences between this study and Rickenmann’s study in terms of geophone inclination, particle properties and channel morphology (i.e. bed slope and roughness).

Furthermore, the relative particle submergence $d/h$ significantly affects the particle transport trajectories. The calibration coefficient $K_b$ cannot be investigated in the laboratory due to limited discharge capacity. For these reasons, a field instead of a laboratory calibration was additionally conducted (cf. Sections 4.2.3 and 5.2).
Results of the geophone calibration

Figure 5.4 Calibration coefficient $K_b$ as a function of mean flow velocity, with $K_b$ determined in laboratory and $K_b$ extrapolated according to the hydraulic conditions in the Solis SBT assuming the Solis GSD

5.2 Results of the field geophone calibration

The results of the field geophone calibration involving three test runs in the Solis SBT are presented in this section. First, the bedload transport measurements during the test runs are presented. Second, the calibration coefficients $K_b$ of each test run is explained, and third, the calibration of the SPGS in the Solis SBT is treated. Different equations to compute $K_b$ considering the instantaneous GSD and sediment transport rate using the $z_p$-value were established.

5.2.1 Bedload measurements

An uneven lateral bedload transport distribution across the tunnel width was observed during all three test runs. Figure 5.5 shows that sediment transport was concentrated at the right tunnel side indicated by a darker flow area, and almost no particles were transported on the left side. The SPGS measurements confirm these observations and revealed a strong effect of the upstream bend on the lateral sediment transport distribution. Figure 5.6a shows the absolute number of impulses of each geophone plate, and Figure 5.6b presents the relative number of registered impulses, i.e. the number of impulses per plate divided by the total number of impulses across the SBT width. The latter data collapse well irrespective of the test run and thus GSD. Similar lateral bedload transport distributions were measured with the SPGS during previous SBT operations (Chapter 6.2), confirming that the SPGS calibration runs effectively reproduced the sediment transport under real Solis SBT operating conditions.
Results of the geophone calibration

Figure 5.5  SBT outlet with clear water and black sediment jet on the right side during test run 3 with $d = 0 \text{ - } 400 \text{ mm}$ (view in flow direction from underneath the SBT outlet)

Figure 5.6  a) Triggered impulses, and b) relative number of impulses registered by each geophone plate during the three field calibration test runs

Figure 5.7 shows the time series of the impulses and the $z_p$-values for each geophone during the calibration test runs. The uniform material tests (Test 1 and 2) show similar transport characteristics. The geophone system started to register bedload transport 135 s after the gate opening (Figure 5.7a and c). The time lag between the gate opening and the detection of the center of the bedload bulk at the outlet was about 165 seconds and includes a certain duration for entrainment of the deposited particles. Sediment particles were first detected by geophone 1 on the right tunnel side and were progressively detected by the other geophones. The time lag between geophone 1 and 8 was around 20 s. The number of impulses increased up to a quasi-constant value, stayed constant for a certain period before decreasing and displaying a quasi-symmetric trapezoidal impulse curve. The number of impulses significantly decreased with increasing distance from the right tunnel wall.
The sediment mixture test revealed different sediment transport behavior compared to the uniform material tests. All geophones simultaneously started to register particles impacts around 10 s earlier than in the uniform material test (Figure 5.7e). The impulse curve follows a triangular instead of a trapezoidal shape. This result is attributed to the GSD differences between the tests, i.e. mixture versus uniform GSD. Furthermore, the impulse curves recorded by geophone 1 to 4 were quasi-symmetric, whereas they exhibited a steep rising and then a flat receding limb recorded by the others. Similar to test 1 and 2, the number of impulses decreased from right to left, confirming the tunnel bend effect on the lateral sediment transport distribution (Figure 5.7a, c and e).

The mean particle transport velocity, defined as the flow stretch divided by the time lag between SBT opening and bedload bulk center registered at the outlet resulted in 5.8, 5.8 and 6.4 m/s, for Test 1, 2 and 3, respectively. By assuming fully developed particle motion from the beginning, mean particle velocities of 10.2, 10.7 m/s and 8.8 m/s result from Equation (3.25) or 7.72, 7.63 and 7.22 m/s from Equation (3.31) assuming $U_p = V_p \cdot \cos(\alpha)$ with $\alpha = \arctan(H_p/[L_p/2])$ from Equation (3.32) and (3.33). The deviation between the measured mean particle velocity and the calculated values is attributed to the time required for the sediment entrainment, which lasted between 40 and 70 s.

The signal overlap probability of particle impacts increases with increasing specific bedload transport rate. To assess this probability, the particle impact overlap indicator $z_p$ introduced by Wyss et al. 2015 (Chapter 4.2.1) was determined for each test and geophone (Figure 5.7b, d and f). The lateral $z_p$ distribution follows the shape of the corresponding impulse curves and hence the lateral bedload distribution, which decreases from the right to the left side. The difference between geophone 1 and 8 is larger for Test 1 and 2 than for Test 3. This is attributed to the narrower grain size spectrum. During the phase with almost constant number of impulses (= main bedload transport phase) the $z_p$-values of Test 2 and 3 were between 8 and 75%. This was significantly higher compared to Test 1 with values between 3 and 45% (Figure 5.7b, d). This phenomenon is attributed to the larger particle size causing higher amplitudes, lower signal frequency and resulted in higher $z_p$-values (Wyss et al. 2016a). However, according to Wyss (2016) $z_p$-values are extremely high for all tests, indicating significant signal interference that affected measurements.
The data from the present laboratory and field calibration tests were compared with Wyss’ (2016) data in Figure 5.8. The data agree and follow the same trend, in particular for particles with $d = 46 - 96$ mm. Deviations of the smaller and larger particles are due to the grain
size effect. With decreasing particle size, $z_p$ and thus signal saturation declines. The smallest particles with $d = 22 - 28$ mm are close to the detection threshold of the SPGS and led to low detection percentages and $z_p$-values.

Wyss (2016) proposed a linear relationship of $z_p = 0.0064 \cdot q_s$ with $R^2 = 0.94$ to describe the relation between specific gravimetric bedload transport rates and corresponding $z_p$-values at the bedload measurement station in the Erlenbach River (thin line in Figure 5.8). However, an exponential function, asymptotically converging to 1, is more suitable to represent the physical process of signal saturation (dotted line in Figure 5.8). The best fit for Wyss’ data with a flat geophone is given by $z_p = 1 - \exp(-0.0072 \cdot q_s)$ with almost the same correlation coefficient of $R^2 = 0.93$. For bedload transport rates below 65 kg/(s·m) corresponding to $z_p \approx 0.4$, the difference between both fits is less than 13%, but increases rapidly beyond this limit. However, such high bedload transport rates rarely occur even in Alpine Rivers with high specific bedload transport rates.

The best fit for the own data set with a geophone inclination of $10^\circ$ (including laboratory and field calibration) follows $z_p = 1 - \exp(-0.00456 \cdot q_s)$ with $R^2 = 0.75$. Wyss (2016) stated that the effect of signal saturation at $z_p < 0.01$ is insignificant and can be neglected. However, in SBTs high bedload transport rates with correspondingly high $z_p$-values are possible and hence a certain degree of signal saturation can occur. The latter can be accounted for by applying one of the two exponential fits according to the geophone installation (0° or 10° inclined).

![Figure 5.8](image-url) $z_p$ depending on sediment transport rate at the Erlenbach River (Wyss 2016) with the corresponding fit, as well as for the laboratory and field calibration experiments
5.2.2 Bedload calibration coefficient $K_b$

The bedload calibration coefficient $K_b$ was determined for each test run (Table 5.2). Figure 5.9 shows $K_b$ versus particle size for the Solis field and laboratory calibration tests as well as Morach’s (2011) laboratory tests. As expected, $K_b$ varies with particle size (see Chapter 4.7, Morach 2011, Wyss 2016b). Despite different hydraulic and bedload transport conditions, the $K_b$-values are in a similar range, particularly for particles with $d > 50$ mm. This indicates that the effect of flow velocity decreases for particle sizes larger than $d = 50$ mm. In contrast, $K_b$ for particles with $d \approx 25$ mm is significantly higher for the field calibration compared to the laboratory calibration due to the effect of the flow velocity. The velocity in the Solis SBT is higher than that in the laboratory. As a result, the kinetic energy of the impinging particles is higher, which amplifies the signal and causes higher $K_b$-values. The difference between Morach’s (2011) and the results of this study may come from electric interference. This caused a significantly higher background noise, amplified the signal and biased the results, in particular for small particles with low signal amplitudes in the range of the detection threshold.

Table 5.2 Grain size and $K_b$ based on the field calibration

<table>
<thead>
<tr>
<th></th>
<th>Test 1</th>
<th>Test 2</th>
<th>Test 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>$d$ [mm]</td>
<td>16 - 32</td>
<td>32 - 63</td>
<td>0 - 400</td>
</tr>
<tr>
<td>$d_m$ [mm]</td>
<td>25 ± 2</td>
<td>45 ± 4</td>
<td>210 ± 20</td>
</tr>
<tr>
<td>$K_b$ [1/kg]</td>
<td>6.80</td>
<td>13.55</td>
<td>5.05</td>
</tr>
</tbody>
</table>

Figure 5.9 $K_b$ as a function of particle size $d$ for the laboratory and field calibration as well as Morach’s (2011) results with the bimodal correlation and the Frechet fit for the field data.

The data sets from the field and laboratory calibrations were fitted with (I) a bimodal function containing a linear rising (Equation (5.1)) and a power law falling limb (Equation (5.2)) and
(II) a Frechet distribution (Equation (5.3)), which was already successfully applied by Wyss (2016) and Wyss et al. (2016a, 2016b). The coefficients $c_1$ to $c_8$ were determined for each data set and listed in Table 5.3 for both approach (I) and (II).

\[ K_b = c_1 \cdot d + c_2 \quad (5.1) \]

\[ K_b = c_3 \cdot d^{c_4} \quad (5.2) \]

\[ K_b = c_3 \cdot c_6 \cdot c_7 \cdot c_8 \left[ 1 - \exp \left( - \left( \frac{c_6}{d} \right)^{c_7} \right) \right]^{c_8-1} \cdot d^{-(c_7+1)} \cdot \exp \left( - \left( \frac{c_8}{d} \right)^{c_7} \right) \quad (5.3) \]

### 5.2.3 Solis SPGS calibration

The SPGS in the Solis SBT was calibrated based on the field calibration test, the laboratory calibration and the results of Morach (2011). The $K_b$-values for typical Solis SBT operating conditions were determined by applying a weighted averaging method based on the Solis GSD using Equation (5.1), (5.2) and (5.3) with the coefficients $c_1$ to $c_8$ listed in Table 5.3. The resulting $K_b$-values were summarized in Table 5.4. For sensitivity purposes, the $K_b$-values were additionally determined for a coarse (according to Awazu 2015) and a fine GSD (according to Schläppi 2009) covering the whole possible grain size bandwidth shown in Figure 4.9. Based on the field calibration, $K_b$ is expected to vary from 9.6 to 10.8 l/kg for the Solis GSD. The lower and upper limits are between 8.7 and 9.7 l/kg for the coarse and 11.4 and 12.6 l/kg for the fine GSD, respectively. The laboratory calibration test results lead to comparable $K_b$ values, while the values determined from Morach’s (2011) data are 10 - 20% higher than those of the other two calibrations.

<table>
<thead>
<tr>
<th>Linear fit Equation (5.1)</th>
<th>Power fit Equation (5.2)</th>
<th>Frechet fit Equation (5.3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c_1$</td>
<td>$c_2$</td>
<td>$R^2$</td>
</tr>
<tr>
<td>Field</td>
<td>0.50</td>
<td>-6.7</td>
</tr>
<tr>
<td>Laboratory</td>
<td>0.61</td>
<td>-11.4</td>
</tr>
<tr>
<td>Morach (2011)</td>
<td>0.90</td>
<td>-13.8</td>
</tr>
</tbody>
</table>
Table 5.4 Test conditions and impulse counting threshold of the calibration tests with the weighted averaged $K_b$–values for the Solis GSD applying a bimodal and a Frechet fit to the Solis GSD as well as for a coarse and a fine GSD

<table>
<thead>
<tr>
<th>Data set</th>
<th>$U$ [m/s]</th>
<th>$h$ [m]</th>
<th>$q_s$ [kg/s]</th>
<th>Impulse threshold [V]</th>
<th>Bimodal fit ($K_{b,coarse}$ $K_b$ ($K_{b,fine}$)</th>
<th>Frechet fit ($K_{b,coarse}$ $K_b$ ($K_{b,fine}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Field</td>
<td>9.8</td>
<td>1.2</td>
<td>~100</td>
<td>0.1</td>
<td>(8.7) 9.6 (11.4)</td>
<td>(9.7) 10.8 (12.6)</td>
</tr>
<tr>
<td>Laboratory</td>
<td>3</td>
<td>0.10</td>
<td>~10</td>
<td>0.1</td>
<td>(9.1) 10.1 (11.8)</td>
<td>(9.5) 10.7 (12.6)</td>
</tr>
<tr>
<td>Morach (2011)</td>
<td>7.4</td>
<td>0.052</td>
<td>~10</td>
<td>0.2</td>
<td>(10.6) 12.2 (15.6)</td>
<td>(9.8) 11.6 (15.7)</td>
</tr>
</tbody>
</table>

The transport capacity in the controlling river section and downstream is not sufficient to carry sediment represented by the coarse GSD. Hence, the coarse GSD is not representative and the corresponding $K_b$-value were neglected. Since the hydraulic conditions and sediment transport rates in the laboratory do not represent the field conditions, a geophone calibration based on laboratory results is questionable. Moreover, Morach (2011) applied a different impulse threshold of 0.2 V instead of 0.1 V due to electric interferences causing large background noise. Therefore, both laboratory results are only used for a plausibility check of the field results but not for calibration purposes. Finally, the average $K_b$ for the Solis GSD obtained from the field calibration amounts to $K_b = (9.6 \, \text{l/kg} + 10.8 \, \text{l/kg})/2 = 10.2 \, \text{l/kg}$. This value is in agreement with the values reported by Rickenmann et al. (2013).

Geophone calibration is further affected by the sediment transport rate and accordingly the impact overlap coefficient $z_p$. Figure 5.10 shows the $K_b$-values of the 287 laboratory ($d = 22 - 123 \, \text{mm}$) and the three field calibration test runs ($d = 0 - 400 \, \text{mm}$) as a function of $z_p$. Only multiple particle experiments (Test Series II-IV) with more than one impact are shown, since $z_p = 0$ for single particle experiments (Test Series I, Chapter 4.2.2.2). Although the data in Figure 5.10 largely scatter due to the grain size effect, the results show that $K_b$ decreases with increasing $z_p$. By applying the least squares fitting to log-transformed laboratory data $K'_b = 8.5 \cdot z_p^{-0.11}$ and a standard deviation of $\sigma = \pm 4.19 \, \text{l/kg}$ was obtained. The first constant includes a fitting parameter and the particle size-dependent calibration coefficient. The universal shape of the fitting equation accounting for the GSD and the signal saturation represented by $z_p$ follows as:

$$K'_b = 0.60 \cdot K_b \cdot z_p^{-0.11} \, [\text{l/kg}]$$ (5.4)

The results of the three field calibration tests are in a similar range as the laboratory results and follow the trend of the latter despite higher $z_p$-values. The deviations from the mean fit is attributed to the grain size effect shown in Figure 5.1 and Figure 5.9. Hence, the effect of signal saturation on the calibration coefficient revealed in the laboratory is confirmed by the field tests.
To account for this, Equation (5.4) was applied to the SPGS measurements in the Solis SBT to determine the instantaneous bedload transport rates.

![Figure 5.10](image)

Figure 5.10  $K_b'$ as a function of $z_p$ for the laboratory and field calibration

### 5.3 Amplitude class method (AC)

Wyss (2016) introduced the amplitude class method (AC-method) to extract information not only on bedload mass but also on GSD from the SPGS signals (cf. Chapter 4.2.1). The method is based on the fact that larger particles exhibit higher impact energy, resulting in higher signal amplitudes compared to small particles (Etter 1996, Rickenmann et al. 2014). By introducing different threshold amplitudes for different particle size classes, the relative number of impulses and the corresponding bedload mass of each particle size class is computed. More detailed information on data analysis techniques is presented by Turowski et al. (2008), Rickenmann et al. (2012, 2014), Wyss (2016) and Wyss et al. (2016a).

This approach was applied to the field measurement data to investigate its applicability and accuracy in the Solis SBT. Table 5.5 lists the threshold amplitudes for each particle size class of the laboratory and the field tests. Note that the maximum amplitude of the laboratory test was determined based on the single particle tests (Test Series I) without any particle impact interaction, and the maximum amplitude of the field tests equals to the 99%-percentile amplitudes of each test run. Figure 5.11 shows the maximum amplitudes versus particle sizes for the laboratory and field calibration tests as well as Wyss’ (2016) data. The amplitudes from the present study increase with increasing particle size for all test series. However, for $d > 96$ mm the maximum amplitude was quasi-constant for the laboratory tests. This is
attributed to the effects of the particle transport mode varying with particle size and the relative particle submergence on the geophone measurements. For particles with \( d > 96 \) mm the relative submergence was \( d/h < 1 \), which hindered particle saltation and a further increase of the signal amplitude. Visual observation during the laboratory tests confirmed this hypothesis.

Compared to Wyss’s (2016) data, the present data show similar but slightly higher signal amplitudes (Figure 5.11). This difference may originate from the effect of geophone mounting (10° counter-inclined versus flush to the bottom, i.e. 0°) and the flow conditions.

Table 5.5  Particle size and corresponding maximum signal amplitudes of the single particle laboratory tests (Test Series I) and the three field calibration tests runs

<table>
<thead>
<tr>
<th>( d ) [mm]</th>
<th>22</th>
<th>28</th>
<th>36</th>
<th>46</th>
<th>59</th>
<th>75</th>
<th>96</th>
<th>123</th>
<th>158</th>
<th>16-32</th>
<th>32-63</th>
<th>0-400</th>
</tr>
</thead>
<tbody>
<tr>
<td>( d_{\text{max}} ) [mm]</td>
<td>22</td>
<td>28</td>
<td>36</td>
<td>46</td>
<td>59</td>
<td>75</td>
<td>96</td>
<td>123</td>
<td>158</td>
<td>32</td>
<td>63</td>
<td>400</td>
</tr>
<tr>
<td>( A_{\text{max}} ) [V]</td>
<td>-</td>
<td>0.12</td>
<td>0.34</td>
<td>0.64</td>
<td>1.20</td>
<td>2.39</td>
<td>3.98</td>
<td>4.78</td>
<td>5.12</td>
<td>0.47</td>
<td>1.25</td>
<td>3.8</td>
</tr>
</tbody>
</table>

Figure 5.11  Maximum amplitudes as a function of particle size by Wyss (2016) as well as of the single particle laboratory calibration (Test Series I) and the field calibration tests

The upper and lower threshold amplitudes for three sediment classes used in the field calibration tests, i.e. 16 - 32 mm, 32 - 63 mm and 63 - 400 mm are listed in Table 5.6. Since the particle size class 1 and 2 correspond to those of Test 1 and 2, the calibration coefficients \( K_b \) are identical. The \( K_b \)-value of the third particle size class was computed based on the \( K_b \)-value of Test 3 with \( d = 0 - 400 \) mm including particles of the first two particle size classes (Table 5.2 and Table 5.6).
Table 5.6  Grain sizes, amplitude thresholds and calibration coefficients of the three particle size classes obtained from the field calibration

<table>
<thead>
<tr>
<th>Class</th>
<th>Class 1</th>
<th>Class 2</th>
<th>Class 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>$d$ [mm]</td>
<td>16 - 32</td>
<td>32 - 63</td>
<td>63 - 400</td>
</tr>
<tr>
<td>Upper threshold $A_{\text{max}}$ [V]</td>
<td>0.47</td>
<td>1.25</td>
<td>3.8</td>
</tr>
<tr>
<td>Lower threshold $A_{\text{min}}$ [V]</td>
<td>0.1</td>
<td>0.47</td>
<td>1.25</td>
</tr>
<tr>
<td>$K_b$ [1/kg]</td>
<td>6.80</td>
<td>13.55</td>
<td>2.93</td>
</tr>
</tbody>
</table>

By applying these $K_b$-values and these thresholds to the SPGS measurements, the sediment mass and the GSD of each test run, respectively, were estimated. The resulting sediment masses are listed in Table 5.7. The estimations significantly deviate from the measured sediment masses for Test 2 and 3. Figure 5.12 shows the calculated GSD together with the measured GSD of each test run. The share of the smallest and largest particle size classes of Test 2 and 3 were considerably overestimated and underestimated, respectively. This is attributed to the fact that the impact of a large particle causes a sequence of decreasing signal amplitudes, which presumably were partly assigned to smaller particle size classes. Moreover, the high sediment transport rates result in significant particle impact overlapping and hence signal damping. Both causes a bias towards the smaller particle size classes.

Table 5.7  Estimated and weighted sediment masses for each field calibration test run

<table>
<thead>
<tr>
<th>Test</th>
<th>Test 1</th>
<th>Test 2</th>
<th>Test 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>$d$ [mm]</td>
<td>16 - 32</td>
<td>32 - 63</td>
<td>0 - 400</td>
</tr>
<tr>
<td>Estimated mass [to]</td>
<td>15.2</td>
<td>29.5</td>
<td>13.8</td>
</tr>
<tr>
<td>Measured mass [to]</td>
<td>15.2</td>
<td>15.4</td>
<td>18.4</td>
</tr>
<tr>
<td>Mass deviation [%]</td>
<td>0</td>
<td>91.3</td>
<td>25.1</td>
</tr>
</tbody>
</table>

Figure 5.12  Target and AC-method based calculated GSD of the three field calibration test runs
5.4 Discussion

The calibration of a SPGS, i.e. the calibration coefficient $K_b$, is generally not constant but a function of particle size and shape, hydraulic condition and mounting torque (Morach, 2011, Wyss, 2016). Further parameters affecting $K_b$ are the mean flow velocity and the geophone plate inclination. Figure 5.13 shows $K_b$ as a function of mean flow velocity for different geophone inclinations, i.e. 0° (= flush to the bottom) and 10° (=counter inclined against flow direction). The data largely scatter due to different testing conditions such as: (1) the geophone configurations, i.e. plate inclination, (2) relative grain submergence, (3) sediment transport rates and (4) impulse counting threshold values. Due to a lack of data, it was not possible to investigate and reliably quantify the effect and interactions of these parameters. Therefore, further investigations are required.

Despite this large data scatter in Figure 5.13, it is clearly shown that $K_b$ of flush installed geophone plates feature a decreasing trend with increasing flow velocity. In contrast, the data for the 10° inclined geophone plates was quasi-constant. This indicates that the calibration of 10° inclined SPGS does not depend on the flow velocity for the investigated flow velocity range of $U = 3 - 10$ m/s. Hence, extrapolation of the laboratory results at $U = 3$ m/s to calibrate the Solis SPGS for $U \approx 11$ m/s would result in a significant underestimation of $K_b$ and hence overestimation of bedload transport rates in the Solis SBT. Therefore, the calibration coefficient $K_b = 1.34$ l/kg presented in Chapter 5.1.3 was not used to determine the bedload transport rates in the Solis SBT.

![Figure 5.13](image-url) $K_b$ as a function of mean flow velocity and geophone inclination, data from Rickenmann et al. (2013), Morach (2011) and gained from laboratory and field calibration experiments for the Solis SPGS
The laboratory and field calibration furthermore revealed a significant effect of bedload transport rate on the SPGS calibration. The calibration coefficient $K_b$ decreases with increasing bedload transport rate due to impact overlaps causing a signal damping and a certain degree of signal saturation. Hence, the impulse counting method and the application of the AC-method lead to inaccurate results and incorrect GSD information for high sediment transport rates (Wyss 2016). Therefore, signal saturation should be accounted for in the calibration of SPGS, in particular for high bedload transport rates.

The SPGS in the Solis SBT was calibrated based on field and laboratory calibration tests. Since the bedload transport rates were low during normal SBT operation, the approach of the AC method was applied to determine the instantaneous GSD as a first approach. Based on that, the weighted average calibration coefficients $K_b$ were determined using Equation (5.3). To account for the particle impact overlap and signal damping $K_b$ was scaled according to Equation (5.4). It is noted that $z_p$-scaling bases on laboratory results and its applicability to field scale, in particular beyond the calibration range of $z_p = 0.003 - 0.3$ includes an uncertainty, requiring further investigations.

Note that geophone calibration is strongly site-specific. The results presented herein hold only for Solis, but can serve as a rough guideline for other sites.
### 6 Results of the test case Solis SBT

The results of the sediment transport and hydroabrasion investigations of the Solis SBT are reported and discussed in this chapter. First, the hydraulics in the Albula and the Solis SBT are presented, and the site-specific distinction between bedload and suspended load is specified. Second, the sediment transport estimations and the effect of the SBT on the sediment transport in the river are presented and validated. Third, the results of the abrasion monitoring of the different invert materials are presented, and fourth, the effect of the reservoir and SBT operating conditions on the bypass efficiency are discussed.

#### 6.1 Hydraulics

The flow velocities and depths in the river as well as in the SBT were calculated based on discharge hydrographs of the river and of the SBT, respectively. The temporal interval was 15 minutes in the Albula, and 1 min for the SBT discharge. The hydraulic and geometric parameters used are listed in Table 6.1. In the Albula uniform flow was assumed and the Manning-Strickler flow equation (Equation (3.12)) was applied. The flow conditions in the SBT varied, so that backwater calculations (with Equation (3.8) and (3.10)) were performed. Figure 6.1 shows the water depth, the flow velocity and the Froude number in the SBT as a function of distance from the SBT inlet for the SBT design discharge capacity. After the pressurized inlet the highly supercritical flow is slowly decelerated along the tunnel but remains supercritical until the outlet. It is noted that the section with the lowest bed shear stress near the outlet is decisive regarding the bedload transport capacity. The mean approach flow and bypassed discharge during SBT operations are listed in Table 6.3.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Albuла River</th>
<th>Solis SBT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cross section</td>
<td>Trapezoidal</td>
<td>Archway (Chapter 4.3.3)</td>
</tr>
<tr>
<td>Bank slope 1:1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Channel width [m]</td>
<td>18.75</td>
<td>4.4</td>
</tr>
<tr>
<td>Slope [%]</td>
<td>0.88</td>
<td>1.9</td>
</tr>
<tr>
<td>Roughness</td>
<td>$k_{st} = 29.0 \text{ m}^{1/3}/\text{s}$</td>
<td>$k_s = 3 \text{ mm}$</td>
</tr>
</tbody>
</table>
6.2 Bedload transport

In this subchapter first, the initiation of bedload transport in the Albula and then the bedload transport in the Albula are presented. Next, the bedload transport in the SBT is reported and discussed.

6.2.1 Bedload transport initiation in the Albula River

The initiation of bedload transport depends on the existence of an armor layer. According to the different criteria listed in Table 6.2 the Albula exhibits an armor layer. Therefore, a higher erosion resistance of the bed material was expected. The critical Shields parameter ($\theta_c = 0.047$) was accordingly adapted by applying a mean particle size of the substrate $d_{m,substrat} = 60$ mm and the armor layer $d_{m,armor} = 80$ mm to Equation (3.19). The resulting $\theta_c' = 0.060$ was used in this study for the bedload estimation. The corresponding threshold discharge for initiation of bedload transport is 31.5 m$^3$/s. This is in good agreement with the result of the bedload measurement of Rickenmann (2016), which revealed that significant bedload transport, i.e. $Q_s \geq 1$ kg/s occurs at discharges between 19 and 50 m$^3$/s.

Even if hydraulic conditions remain below the threshold for the initiation of particle motion, bedload transport can occur in case of sediment supply from upstream reaches or suction removal of particles from between the armor layer blocks. The former is not considered herein, for the latter Equation (3.20), introduced Sumer et al. (2001), is used to determine the critical Shields parameter for suction removal. For the Albula it is $\theta_{su} = 0.30$. The ratio $\theta_{su}/\theta_c' = 5$ is
Results of the test case Solis SBT considerably larger than \(d_{m,\text{armor}}/d_{m,\text{substrat}} = 1.3\). Therefore, suction removal does not take place and was not considered for bedload estimations in this study.

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Value</th>
<th>Armor layer</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gessler (1965)</td>
<td>(d_{s4}/d_{50} &gt; 2.0)</td>
<td>2.8</td>
</tr>
<tr>
<td>Little and Mayer (1976)</td>
<td>((d_{s4}/d_{16})^{1/2} &gt; 1.3)</td>
<td>7.0</td>
</tr>
<tr>
<td>Chin (1985)</td>
<td>(d_{s4}/d_{50} &gt; 1.5 &amp; d_{\text{max}}/d_{50} &gt; 1.8)</td>
<td>2.8 &amp; 5.7</td>
</tr>
<tr>
<td>Schöberl (1992)</td>
<td>((d_{s4}/d_{16})^{1/2} &gt; 1.35, d_{s4}/d_{50} &gt; 1.55 &amp; d_{m}/d_{50} &gt; 1.05)</td>
<td>7.0, 4.3 &amp; 1.7</td>
</tr>
<tr>
<td>Günter (1971)</td>
<td>((d_{s4}/d_{16})^{1/2} &gt; 1.4 - 1.6)</td>
<td>7.0</td>
</tr>
</tbody>
</table>

### 6.2.2 Bedload transport into the Solis Reservoir

The estimated annual bedload volumes \(BL_{22,v}\) in the Albula between 1987 and 2016 are shown in Figure 6.2. The volumes varied between 4.3 and \(94.6 \cdot 10^3\ m^3\) with an average of \(35.2 \cdot 10^3\ m^3\) between 1987 and 2016.

![Figure 6.2 Annual \(BL_{22,v}\) in the Albula](image)

On average, \(31.4 \cdot 10^3\ m^3\) of sediment are annually excavated by the gravel plant situated at the reservoir head. This volume was assumed to consist of 50% \(BL_{22,v}\) and 50% \(SL_{\text{coarse,v}}\). As a result, a volume of \(BL_{22,v} = 35.2 \cdot 10^3\ m^3 - 0.5 \times 31.4 \cdot 10^3\ m^3 = 16.8 \cdot 10^3\ m^3\) deposited in the reservoir.

### 6.2.3 Bedload transport in the SBT

The SBT has been in operation 9 times since its commissioning in 2012. The annual average, total and single operation data are listed in Table 6.3. These are: approach flow discharge \(Q_0 = Q_{\text{Albula}} + Q_{\text{Julia}} + Q_{\text{HPP Tiefencastel}}\), bypassed discharge \(Q_{\text{SBT}}\), reservoir water level \(RWL\), operating duration and bypassed bedload volumes. In general, the SBT target operating
conditions are: (I) $Q_0 \geq 90 \text{ m}^3/\text{s}$ and (II) reservoir water level of 816 masl. However, the SBT was operated under deviating conditions during the flushing of the Burvagn Reservoir on the 23rd of May 2014 and for testing instruments on the 9th of June 2015. These two events were considered in the data analysis as well, since they may affect the sediment transport processes in the reservoir and the SBT.

The results from the AC-method revealed that 80 - 95% of the bedload particles were smaller than 28 mm. The resulting GSDs were accordingly finer than expected (cf. GSD of each operation and expected Solis GSD in Figure 6.7). In addition to the SPGS measurements, a 1D numerical simulation\(^1\) of sediment transport assuming a simplified reservoir geometry was conducted. The simulation was performed for the design conditions with $Q_0 = Q_d = 170 \text{ m}^3/\text{s}$ and a reservoir water level of 816 masl. Figure 6.3 shows the resulting longitudinal profile of the energy line, the reservoir water level, the reservoir bed and the critical particle size for initiation of bedload transport between the reservoir head ($x \approx 2900 \text{ m}$) and the SBT intake ($x \approx 0 \text{ m}$). The critical particle size is constant with $d_c \approx 64 \text{ mm}$ between the inflow at $x \approx 2900 \text{ m}$ and $x \approx 200 \text{ m}$ upstream of the SBT intake, but decreases afterwards down to $d_c \approx 20 \text{ mm}$ at the pivot point due to backwater effects. This point represents the critical cross section, where the particles were transported by gravity instead of bed shear stress. Hence, the particles are assumed to be transported through the SBT. This value is slightly smaller than the 28 mm measured by the SPGS in the SBT. This deviation originates from the simplified 1D numerical investigation, which is not capable of reproducing the complex 3D-flow structures in the reservoir. Overall, the results from the SPGS and the 1D-numerical investigation are in agreement and show that the GSD is finer than expected. Based on this simplified 1D numerical simulation the reservoir water level should be reduced to roughly 815.2 masl to avoid backwater effects and hence a drop of the critical particle size.

\(^1\) The channel width was assumed to $b = 47 \text{ m}$ according to Zarn (2009) and VAW (2010). The slope $S = 0.0044$ between the reservoir head and the pivot point of the aggradation body was determined based on bathymetrical data. After the pivot point a scour cone connecting the pivot point and the SBT intake was implemented in the model.
Results of the test case Solis SBT

The bypassed bedload transport rates and $BL_{22}$ in the Solis SBT were determined by applying two methods to the SPGS raw data: (I) the GSD-dependent $K_b$ (Equation (5.3)) to account for the GSD and (II) the $z_p$-scaled GSD-dependent $K_b'$ (Equation (5.4)) to additionally account for the effect of signal saturation. The obtained $K_b$ values were about 40% lower than the value for the Solis GSD with $K_b = 10.2$ l/kg (cf. Chapter 5.1.3) because the estimated GSD was significantly finer than expected (cf. Solis GSD in Figure 4.9). The resulting $BL_{22,v}$ are listed in Table 6.3. The values obtained from Method (I) deviates ±15% from those obtained from Method (II) due to the effect of signal saturation. To account for the effects of GSD and signal saturation, the results of Method (II) were considered in the data analysis. The bypassed $BL_{22,v}$ varied between 0 and 888 m$^3$ per operation with a mean of 227 m$^3$. The mean annual bypassed bedload volume amounted to $BL_{22,v} = 409$ m$^3$ and is one to two orders of magnitude below the net annual bedload inflow of $BL_{22,v} = 16.8\cdot10^3$ m$^3$ (Chapter 6.2.2). This indicates a significant deposition of the inflowing bedload. The results of bathymetric surveys conducted between 2012 and 2015 revealed an accumulation of 46.4\cdot10^3$ m$^3$ per year (Meisser Vermessungen 2015, see Figure 6.6), which confirms the result of the SPGS.
Table 6.3 Bypassed bedload volume separated by year and operation based on a GSD-dependent (Method I) and additionally \(z_p\)-scaled (Method II) calibration coefficient

<table>
<thead>
<tr>
<th>SBT operation</th>
<th>(Q_0) [m(^3)/s]</th>
<th>(Q_{SBT}) [m(^3)/s]</th>
<th>(\overline{RWL}) [masl]</th>
<th>Operation duration [h]</th>
<th>Method I (BL_{22,v} = \overline{f(K_b)}) [m(^3)]</th>
<th>Method II (BL_{22,v} = \overline{f(K_b')}) [m(^3)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>2012</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>03.05.2013</td>
<td>76</td>
<td>68</td>
<td>814.91</td>
<td>16.4</td>
<td>22.9</td>
<td>23.8</td>
</tr>
<tr>
<td>2013</td>
<td>76</td>
<td>68</td>
<td>814.91</td>
<td>16.4</td>
<td>22.9</td>
<td>23.8</td>
</tr>
<tr>
<td>23.05.2014*</td>
<td>93</td>
<td>88</td>
<td>820.24</td>
<td>9.7</td>
<td>0.1</td>
<td>0.1</td>
</tr>
<tr>
<td>29.06.2014</td>
<td>91</td>
<td>102</td>
<td>816.22</td>
<td>5.3</td>
<td>51.6</td>
<td>58.5</td>
</tr>
<tr>
<td>13.08.2014</td>
<td>169</td>
<td>153</td>
<td>815.66</td>
<td>14.0</td>
<td>768.2</td>
<td>888.4</td>
</tr>
<tr>
<td>2014</td>
<td>129</td>
<td>122</td>
<td>817.29</td>
<td>29.0</td>
<td>819.9</td>
<td>947</td>
</tr>
<tr>
<td>15.05.2015</td>
<td>84</td>
<td>76</td>
<td>815.73</td>
<td>10.7</td>
<td>16.2</td>
<td>14.1</td>
</tr>
<tr>
<td>09.06.2015**</td>
<td>78</td>
<td>58</td>
<td>815.70</td>
<td>7.8</td>
<td>0.1</td>
<td>0.1</td>
</tr>
<tr>
<td>2015</td>
<td>82</td>
<td>68</td>
<td>815.72</td>
<td>18.5</td>
<td>16.4</td>
<td>14.1</td>
</tr>
<tr>
<td>11.06.2016</td>
<td>67</td>
<td>80</td>
<td>814.82</td>
<td>13.0</td>
<td>173.6</td>
<td>179.6</td>
</tr>
<tr>
<td>16.06.2016</td>
<td>126</td>
<td>129</td>
<td>814.24</td>
<td>24.2</td>
<td>809.3</td>
<td>871.0</td>
</tr>
<tr>
<td>12.07.2016</td>
<td>54</td>
<td>73</td>
<td>815.40</td>
<td>5.5</td>
<td>7.8</td>
<td>6.6</td>
</tr>
<tr>
<td>2016</td>
<td>99</td>
<td>107</td>
<td>814.55</td>
<td>42.7</td>
<td>990.7</td>
<td>1057</td>
</tr>
<tr>
<td>Operational average</td>
<td>101</td>
<td>99</td>
<td>815.55</td>
<td>11.9</td>
<td>206</td>
<td>227</td>
</tr>
<tr>
<td>Mean per year</td>
<td>101</td>
<td>99</td>
<td>815.55</td>
<td>21.3</td>
<td>370</td>
<td>409</td>
</tr>
</tbody>
</table>

* Flushing Burvagn, ** Test event

The SPGS not only allows to determine the bedload transport rate, but also its distribution across the tunnel width at the SBT outlet. Figure 6.4 shows the relative time-averaged lateral bedload distribution, i.e. \(\overline{BL_{22,i}}/\overline{\sum BL_{22,i}}\) (indices denote the geophone plate number) for each SBT operation. A strong bedload transport concentration on the orographic right side, caused by Prandtl’s first type of secondary currents (Chapter 3.1.3, Hagmann et al. (2015a)), is visible. Similar bedload transport concentrations were measured during the field calibration tests (Figure 5.5 and Figure 5.6 in Chapter 5.2.1) and observed in physical model tests conducted by VAW (2010).
6.3 Fine suspended sediment

This subchapter reports the results of the fine suspended sediment transport estimation into and out of the Solis Reservoir. Furthermore, various approaches applied to determine the SSC are presented and discussed.

6.3.1 Fine suspended sediment transport into the reservoir

The fine suspended sediment transport rates and volumes in the Albula were determined from turbidity measurements, and by applying two different SSC-discharge rating curves. These are: (I) \(SSC_B\cdot Q\) and (II) \(SSC_{Tr}\cdot Q\) (cf. Chapter 4.3.8). The resulting \(SSL_{fine,v}\) and \(BL/SL_{fine}\)-ratios are summarized in Table 6.4. Method (I) resulted in significantly lower \(SSL_{fine,v}\) values and higher \(BL/SL_{fine}\)-ratios than Method (II). The results of the latter are in good agreement with those obtained from the turbidity data and with typical \(BL/SL_{fine}\)-ratios for Swiss torrents of 1:1 and 1:2 (Rickenmann 2001, Turowski et al. 2010). This seems to indicate that Method (II) leads to more realistic results compared to Method (I). Therefore, Method (II) is used in this study to determine \(SSL_{fine,v}\) when no turbidity data are available (Table 6.4). The averaged inflow volume of fine suspended sediment into the Solis Reservoir provided by the Albula resulted in \(SSL_{fine,v} = 60.1 \times 10^3 \text{ m}^3\) between 1987 and 2016.
The $SSL_{\text{fine,v}}$ released by the HPP was determined based on the $SSC_{\text{fine}}$ time series of the HPP Sils and the discharge time series of the HPP Tiefencastel. On average, $SSL_{\text{fine,v}} = 4.8 \cdot 10^3$ m$^3$ was released by the HPP Tiefencastel between 1987 and 2016. The total fine suspended sediment inflow volume resulted to $SSL_{\text{fine,v}} = 64.9 \cdot 10^3$ m$^3$ for the years 1987 to 2016.

**Table 6.4** Mean annual $SSL_{\text{fine,v}}$ and $BL/SSL_{\text{fine}}$-ratios in the Albula between 1987 and 2016 obtained from different methods

<table>
<thead>
<tr>
<th>Method</th>
<th>$SSL_{\text{fine,v}} \ [10^3 \text{ m}^3/\text{yr}]$</th>
<th>$BL/SSL_{\text{fine}} \ [-]$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(I) $SSC_{B-Q}$</td>
<td>(II) $SSC_{T-Q}$</td>
</tr>
<tr>
<td>1987-2011</td>
<td>-</td>
<td>14.4</td>
</tr>
<tr>
<td>2012-2016</td>
<td>93.8</td>
<td>15.9</td>
</tr>
<tr>
<td>1987-2016</td>
<td>-</td>
<td>14.6</td>
</tr>
</tbody>
</table>

### 6.3.2 Fine suspended sediment transport out of the reservoir

The fine suspended sediment are released by the different outlet structures from the Solis Reservoir, i.e. the headraces of HPP Sils and Rothenbrunnen, the dam outlets (including bottom outlet and spillway) and the SBT. The mean annual $SSL_{\text{fine,v}}$ released by the HPP Sils, Rothenbrunnen and the dam outlets was $16.3 \cdot 10^3$ m$^3$/yr between 1987 and 2016.

The bypassed $SSL_{\text{fine,v}}$ per operation and year (highlighted in gray) are listed in Table 6.6. On average, the SBT released $SSL_{\text{fine,v}} = 8.3 \cdot 10^3$ m$^3$/yr and increased the outflow of $SSL_{\text{fine,v}}$ by 50% despite the relative short operation duration of a few hours to days per year compared to the HPP running all year. Thus, the bypassed $SSL_{\text{fine,v}}$ can be expected to be significantly increased with the duration of SBT operation in the future.

### 6.4 Coarse suspended sediment

In the following, the sediment transport and the GSD at the Solis Reservoir during the SBT operation on May 15, 2015 is treated first. This is followed by a presentation of the results of coarse suspended sediment estimation into and out of the Solis Reservoir.

#### 6.4.1 Sediment transport during SBT operation on May 15, 2015

The temporal and spatial sediment transport in the Solis Reservoir and the SBT was investigated based on the measurements obtained during the one year flood event on May 15, 2015. The hydraulic operating conditions during this SBT operation are listed in Table 6.3. The bedload
transport and suspended sediment rates were determined using the SPGSs and turbidimeters installed in the Albula and the SBT.

The coarse suspended sediment mass in the Alula was estimated by assuming gravitational $BL_{22}/TL = 0.25$, which is in line with the value for the Albula reported by Rickenmann et al. (2016). The mass of coarse suspended sediment was converted to the volume by assuming a sediment bulk density of 1800 kg/m$^3$. The inflowing $SSL_{coarse,v}$ was assumed to be conveyed through the SBT past the dam, since SPGS measurements in the SBT revealed that particles of this size were transported through the SBT (cf. Chapter 6.2.3).

To establish a sediment balance for this SBT operation, the sediment volumes were calculated by integrating the volumetric transport rates over the SBT operation duration while respecting the particle travel time between the two monitoring stations in the Albula and in the SBT. The travel time depends on the transport mode and is roughly 1.5 h for bedload and 0.3 h for fine suspended sediment, respectively.

The total volumetric sediment inflow during the SBT operation was $TL_v = 3'506$ m$^3$ consisting of $SSL_{fine,v} = 1'015$ m$^3$, $SSL_{coarse,v} = 1'790$ m$^3$ and $BL_{22,v} = 701$ m$^3$ (Table 6.5). The sediment volume released by the SBT was $TL_v = 5'787$ m$^3$ consisting of $SSL_{fine,v} = 3'983$ m$^3$, $SSL_{coarse,v} = 1'790$ m$^3$ and $BL_{22,v} = 14$ m$^3$. The share of the sediments released by the other outlet structures, i.e. spillway, bottom outlet and HPPs Sils and Rothenbrunnen, was about 5% and hence negligible. The $SSL_{fine,v}$ outflow was about 4 times higher than the $SSL_{fine,v}$ inflow, whereas the $BL_{22,v}$ outflow was more than an order of magnitude below the $BL_{22,v}$ inflow. This indicates a deposition of the bedload and re-suspension of the fine sediments, thus causing a shift of the sieve curve toward the fine sediments as shown in Figure 6.5. The re-suspended volume was larger than the deposited volume resulting in a net sediment erosion of 2'610 m$^3$ (Table 6.5).

Table 6.5 Sediment in- and outflow during the SBT operation on May 15, 2015

<table>
<thead>
<tr>
<th></th>
<th>$SSL_{fine,v}$ [m$^3$]</th>
<th>$SSL_{coarse,v}$ [m$^3$]</th>
<th>$BL_{22,v}$ [m$^3$]</th>
<th>$TL_v$ [m$^3$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Albula (= inflow)</td>
<td>1'015</td>
<td>1'790</td>
<td>701</td>
<td>3'506</td>
</tr>
<tr>
<td>SBT (= outflow)</td>
<td>3'983</td>
<td>1'790</td>
<td>14</td>
<td>5'787</td>
</tr>
<tr>
<td>Other outlets (= outflow)</td>
<td>329</td>
<td>0</td>
<td>0</td>
<td>329</td>
</tr>
<tr>
<td>$\Delta$ = Input − Output</td>
<td>−3'297</td>
<td>0</td>
<td>687</td>
<td>−2'610</td>
</tr>
</tbody>
</table>

Figure 6.5 shows the GSD in the Albula and the SBT. Both GSDs were considerably finer than expected, whereas the GSD in the Albula agrees significantly better with Schläppi’s (2009)
Results of the test case Solis SBT

GSD. This result indicates that the GSD reported by VAW (2008) and assumed by Zarn (2009, 2010) are not representative for the sediment transported in the Albula as well as in the SBT, whereas Schläppi’s (2009) GSD agrees better with the GSD in the Albula. However, the GSD in the Albula and the SBT are obtained from data recorded during a one year flood event. At lower discharges the GSD is expected to be even finer due to the lower bed shear stresses. Therefore, application of the AC-method to the SPGS measurements is recommended to determine the GSD and the bedload transport rates.

The ratios of sediment loads, i.e. $SSL_{coarse}/SSL_{fine} = 0.45$ and $SSL_{coarse}/BL_{22} = 82.7$ obtained from Figure 6.5 for the sediment in the SBT serve as a reference for Equation (4.17) and (4.18) to determine the $SSL_{coarse}$ in the SBT for each operation. The results are presented in the following.

![Figure 6.5](image.png)

Figure 6.5 Sieve curves of the sediment transported in the Albula at Tiefencastel and in the SBT during the SBT operation on May 15, 2015, and the expected GSD as well as Schläppi’s (2009) GSD

### 6.4.2 Coarse suspended sediment transport into and out of the Solis Reservoir

By applying the ratios of the sediment loads obtained in Chapter 6.4.1 to the Equations (4.17) and (4.18), the estimated inflow $SSL_{coarse,v}$ was obtained. It is on average $41.5 \times 10^3$ m$^3$ between 1987 and 2016. According to the mean annual sediment volume excavated at the reservoir head, consisting of $BL_{22,v} = SL_{coarse,v} = 15.7 \times 10^3$ m$^3$, the mean annual volume remaining in the reservoir amounts to $SSL_{coarse,v} = 41.5 \times 10^3$ m$^3 - 0.5 \times 31.4 \times 10^3$ m$^3 = 25.8 \times 10^3$ m$^3$.

The $SSL_{coarse,v}$ released from the Solis Reservoir by the SBT for each operation and year (highlighted in gray) are listed in Table 6.6. On average, the SBT released $SSL_{coarse,v} = 8.3 \times 10^3$ m$^3$ per year.
6.5 Total sediment transport

The total sediment inflow and outflow volume of the Solis Reservoir is addressed in this subchapter. First, the estimated accumulation volume was compared with bathymetrically determined volumes to check the plausibility of the obtained results. Second, the sediment transport in the SBT was treated. Third, a sediment balance based on the previously presented results was established. Finally, the bypass and the trap efficiency of the Solis Reservoir with and without SBT were determined and discussed.

6.5.1 Plausibility check of the results based on bathymetric surveys

The methods to determine the sediment volumes in and out of the Solis Reservoir include uncertainties due to measurement errors and model uncertainties. To assess the plausibility of the obtained results, the estimated accumulation volumes were compared with the volumes obtained from bathymetric surveys. Note, both the estimated and measured accumulation volumes do not involve the excavation volumes of the gravel plant at the reservoir head.

Figure 6.6 shows the estimated and measured accumulation volumes for different time periods. The estimates partly match well with the measurements. The largest deviation were obtained for the periods June ’07 - June ’08, June ’08 - June ’09 and March ’12 - August ’12. The bathymetric data of those years include an error due to lacking data in the upper part of the reservoir. The lacking data were filled with data from a former measurement, performed before sediment from the upper part of the reservoir was relocated to the lower part of the reservoir due to drawdowns. Therefore, the bathymetrically determined accumulation volumes for these periods were underestimated. The uncertainty related to the sediment transport volume estimation of up to ±86% (cf. Chapters 4.3.6 to 4.3.9 for estimation errors) might be another reason for this deviation. However, the estimated mean annual accumulation volume between 1987 and 2015 of $87 \times 10^3$ m$^3$ was only 8% higher than the measured volume of $80 \times 10^3$ m$^3$. This result indicates that the assumptions and correlations applied to determine the sediment inflow and outflow volumes are reasonable and lead to realistic estimates.
6.5.2 **Total sediment transport in the Solis SBT**

Based on the bypassed $BL_{22,v}$, $SSL_{fine,v}$ and $SSL_{coarse,v}$ presented in Chapters 6.2, 6.3 and 6.4, the total sediment volumes ($TL_v$) for each operation and year (highlighted in gray) were determined and listed in Table 6.6. The obtained volumes are in line with the estimated net deposition volume in the Albula downstream of the Solis SBT outlet, measured by means of digital elevation models of the river (Facchini 2017), and hence assumed to result in realistic sediment estimates. Therefore, these values together with the previously presented inflow and outflow sediment volumes were used to determine the sediment balance as well as the bypass and trap efficiency of the Solis Reservoir as presented in Chapter 6.5.3 and 6.6. The mean annual bypassed sediment volume was $TL_v = 17.0 \times 10^3$ m$^3$ with a mean $BL_{22}/SSL$-ratio of 2.4%.

The GSD of the bypassed sediments are shown in Figure 6.7 for each SBT operation. The GSDs vary partly significantly, but all of them are considerably finer than the expected Solis GSD with a mean particle size of $d_m = 60$ mm (cf. Chapter 6.2.3), whereas the GSD of Schläppi (2009) is significantly closer to the determined GSDs. Figure 6.8 shows the mean particle size of each operation as a function of bed shear stress determined at the pivot point based on simplified 1D numerical investigations (cf. Chapter 6.2.3). The mean particle size increases with bed shear stress, in particular for high bed shear stresses, i.e. $\tau_b > 10$ Pa. However, even during the large flood events in August 2014 and July 2016, the mean particle sizes were considerably below the expected value of $d_m = 60$ mm due to the back water effect reducing the threshold particle size for the initiation of motion (cf. Chapter 6.2.3). As a result, the weighted averaged mean particle size of $d_m = 6.3$ mm between 2012 and 2015 was one order of magnitude smaller than expected. The rolling and suspension probability of this particle size in...
the SBT is $P_R = 0.01$ (Equation (3.21)), and $P_{su} = 0.41$ (Equation (3.23)) respectively. This means that approx. half of the particles are transported in suspension rarely impacting the tunnel invert and hence cause low abrasion rates on the tunnel invert (Chapter 6.7).

Table 6.6  Bypassed sediment volumes involving fine and coarse suspended sediment and bedload

<table>
<thead>
<tr>
<th>Operation</th>
<th>$SSL_{fine,v}$ [10^3 m^3]</th>
<th>$SSL_{coarse,v}$ [10^3 m^3]</th>
<th>$BL_{22,v}$ [10^3 m^3]</th>
<th>$TL_v$ [10^3 m^3]</th>
</tr>
</thead>
<tbody>
<tr>
<td>$d$ [mm]</td>
<td>$&lt; 0.5$</td>
<td>$0.5 \leq d \leq 22$</td>
<td>$&gt; 22$</td>
<td>$&lt; d_{max}$</td>
</tr>
<tr>
<td>2012</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>03.05.2013</td>
<td>7.0</td>
<td>3.1</td>
<td>0.02</td>
<td>10.1</td>
</tr>
<tr>
<td>2013</td>
<td>7.0</td>
<td>3.1</td>
<td>0.02</td>
<td>10.1</td>
</tr>
<tr>
<td>23.05.2014*</td>
<td>2.1</td>
<td>0.9</td>
<td>0.0</td>
<td>3.0</td>
</tr>
<tr>
<td>29.06.2014</td>
<td>2.5</td>
<td>1.2</td>
<td>0.06</td>
<td>3.7</td>
</tr>
<tr>
<td>13.08.2014</td>
<td>5.6</td>
<td>15.2</td>
<td>0.9</td>
<td>21.7</td>
</tr>
<tr>
<td>2014</td>
<td>10.2</td>
<td>17.4</td>
<td>0.9</td>
<td>28.5</td>
</tr>
<tr>
<td>15.05.2015</td>
<td>4.0</td>
<td>1.8</td>
<td>0.01</td>
<td>5.8</td>
</tr>
<tr>
<td>09.06.2015**</td>
<td>1.0</td>
<td>0.5</td>
<td>0.00</td>
<td>1.5</td>
</tr>
<tr>
<td>2015</td>
<td>5.0</td>
<td>2.2</td>
<td>0.01</td>
<td>7.2</td>
</tr>
<tr>
<td>11.06.2016</td>
<td>8.4</td>
<td>4.2</td>
<td>0.2</td>
<td>12.7</td>
</tr>
<tr>
<td>16.06.2016</td>
<td>8.6</td>
<td>13.4</td>
<td>0.9</td>
<td>22.9</td>
</tr>
<tr>
<td>12.07.2016</td>
<td>2.5</td>
<td>1.1</td>
<td>0.00</td>
<td>3.7</td>
</tr>
<tr>
<td>2016</td>
<td>19.5</td>
<td>18.7</td>
<td>1.1</td>
<td>39.3</td>
</tr>
<tr>
<td>Operational average</td>
<td>4.6</td>
<td>4.6</td>
<td>0.2</td>
<td>9.5</td>
</tr>
<tr>
<td>Annual average</td>
<td>8.3</td>
<td>8.3</td>
<td>0.4</td>
<td>17.0</td>
</tr>
</tbody>
</table>

* Flushing Burvagn Reservoir; ** Test event

Figure 6.7  GSD of the bypassed sediment for all SBT operations and the initially assumed GSD Solis
Figure 6.8  Mean particle size of bypassed sediment as a function of bed shear stress at the pivot point for all SBT operations

6.5.3  Sediment balance of the Solis Reservoir

The estimated annual sediment in- and outflow volumes of the Solis Reservoir and the excavated volumes from 1987 to 2016 are shown in Figure 6.9. The total annual sediment inflow was on average $TL_{v,in} = 139 \cdot 10^3 \text{ m}^3$, of which $TL_{v,ex} = 31.4 \cdot 10^3 \text{ m}^3$ were excavated by the gravel plant at the reservoir head. As a result, a volume of $TL_{v,in-ex} = 138.9 \cdot 10^3 \text{ m}^3 - 31.4 \cdot 10^3 \text{ m}^3 = 107.5 \cdot 10^3 \text{ m}^3$ remain in the reservoir. The annual sediment outflow volume without the SBT was on average $16 \cdot 10^3 \text{ m}^3$ and contained only fine suspended. The SBT significantly increased this volume to $TL_{v,out} = 33 \cdot 10^3 \text{ m}^3$ by releasing $17 \cdot 10^3 \text{ m}^3$, involving fine suspended sediment, coarse suspended sediment and a small share of bedload. However, the sum of the annual sediment outflow and excavation volumes was always smaller than the annual sediment inflow volume, which resulted in accumulations.

Figure 6.9  Estimated annual sediment inflow and outflow of the Solis Reservoir separated by particle size from 1987 to 2016
Results of the test case Solis SBT

Figure 6.10 shows the total annual accumulation volume $TL_{\text{acc},v} = TL_{\text{in},v} - TL_{\text{ex},v} - TL_{\text{out},v}$, the bypassed sediment volumes and the cumulative accumulation volume with and without the SBT. The estimated mean annual accumulation volume without the SBT amounted to $91 \cdot 10^3 \text{ m}^3$ and was reduced by the SBT to $74 \cdot 10^3 \text{ m}^3$. This result indicates that the annual accumulation volume in the reservoir was reduced by 20% due to the SBT operation.

![Graph showing annual estimated accumulated and bypassed sediment volumes and cumulative accumulation volume with and without SBT]

6.6 Bypass and trap efficiency of the Solis Reservoir

The bypass efficiency of a reservoir is defined as the ratio between total sediment outflow and total sediment inflow, i.e. $BE = TL_{\text{out}}/TL_{\text{in}}$. To determine the bypass efficiency of the Solis Reservoir the mean annual sediment inflow, excavation and outflow with and without SBT were computed. The results are listed in Table 6.7. The bypass efficiency of the Solis Reservoir without the SBT was $BE = 0.15$ and increased to $BE = 0.31$ with the SBT. This value is relatively low compared to other reservoirs with a SBT in operation with $BE = 0.60 - 0.90$ (cf. Chapter 2.4.3), due to the different type of SBT and different SBT operation regimes. Most SBTs are type (A) SBTs with an intake at the reservoir head, from where the inflowing sediments are directly conveyed past the dam. In contrast, type (B) SBTs such as the Solis SBT exhibit an intake within the reservoir allowing intermediate deposition of sediments between the reservoir head and the SBT intake. This can negatively affect the bypass efficiency. Moreover, type (B) SBTs are generally operated only during high flood discharge peaks. As a result, the annual operation duration of the Solis SBT was only 21.3 hours, which is more than an order of magnitude below typical operation duration of type (A) SBTs. Overall, the SBT prolonged the reservoir life time from 22 to 27 years by roughly 23%, which is expected to be raised in near future by enhancing the operating regime.
The trap efficiency $TE$ of a reservoir is defined as $TE = 1 - BE$. The trap efficiency of the Solis Reservoir without the SBT amounted to $TE = 0.85$. However, for the fine suspended sediment it was $TE = 0.75$, and amounted to $TE = 1.00$ for the coarse suspended sediment and the bedload. The SBT increased the amount of released sediment in particular for the finer two size classes and decreased the trap efficiency to $TE = 0.69$. According to Brune’s (1953) criteria (cf. Equations (2.1) to (2.3)) the trap efficiency for suspended sediment is expected to be lower with $TE = 0.10 - 0.41$. The difference is attributed to the fact that Brune’s (1953) criteria accounts only for the capacity-inflow-ratio $CIR$ and the particle size, although further parameters such as the operation regime, the reservoir topography, the type of outlet structures and the reservoir age affect the trap efficiency (Chapter 2.1).

Table 6.7 Mean annual sediment inflow and outflow of the Solis Reservoir and corresponding bypass efficiencies between 1987 and 2016 with and without SBT

<table>
<thead>
<tr>
<th></th>
<th>Without SBT</th>
<th></th>
<th>With SBT</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Excavation</td>
<td>Outflow</td>
<td>Accumulation</td>
</tr>
<tr>
<td>$SSL_{fine,v}$</td>
<td>$[10^3 \text{ m}^3/\text{yr}]$</td>
<td>$[10^3 \text{ m}^3/\text{yr}]$</td>
<td>$[10^3 \text{ m}^3/\text{yr}]$</td>
</tr>
<tr>
<td>$SSL_{coarse,v}$</td>
<td>$[10^3 \text{ m}^3/\text{yr}]$</td>
<td>$[10^3 \text{ m}^3/\text{yr}]$</td>
<td>$[10^3 \text{ m}^3/\text{yr}]$</td>
</tr>
<tr>
<td>$BL_{22,v}$</td>
<td>$[10^3 \text{ m}^3/\text{yr}]$</td>
<td>$[10^3 \text{ m}^3/\text{yr}]$</td>
<td>$[10^3 \text{ m}^3/\text{yr}]$</td>
</tr>
<tr>
<td>$TL_v$</td>
<td>$[10^3 \text{ m}^3/\text{yr}]$</td>
<td>$[10^3 \text{ m}^3/\text{yr}]$</td>
<td>$[10^3 \text{ m}^3/\text{yr}]$</td>
</tr>
</tbody>
</table>

* determined based on the sediment masses

The bypass efficiency of the reservoir might not be a suitable parameter to represent the effect of the SBT on the sediment transport in the Solis Reservoir due to its short operation duration of a few hours per year. Therefore, the operational bypass efficiencies $BE_{op}$ considering only the sediment transport during SBT operations is used. Figure 6.11 shows the volumetric sediment in- and outflows and the $BE_{op}$ of the Solis Reservoir during each SBT operation. The sediment inflow volume was always smaller than the sediment outflow volume, and the operational bypass efficiency was always $BE_{op} > 1.00$. The mean operational total bypass efficiency of bedload with $BE_{op} = 0.38$ was considerably lower and more sensitive to the operating conditions. Significant bedload transport in the SBT, i.e. $Q_{BL22} > 1 \text{ kg/s}$, was only registered at reservoir levels below $RWL < 816 \text{ masl}$ (Figure 6.13) due to strong back water effects (cf. Chapter 6.2.3). In contrast, the operational bypass efficiency of the suspended sediment was on average $BE_{op} = 4.70$ indicating a strong re-suspension of suspended sediments from the reservoir. These are important findings regarding the sediment management of the Solis Reservoir and may contribute to an optimized SBT operating regime in the future.
6.7 SBT operation

The full reservoir supply level is at 823.75 masl and the minimum HPP operating level is 816 masl, which equals the SBT operation level recommended by VAW (2010) based on physical model tests. Prior to a flood event with an expected discharge above the SBT operation target value of 90 m$^3$/s, the reservoir water level is lowered to the SBT operation level. This drawdown may cause financial losses if it is not performed by the turbine flow alone. As a result, water level regulation is a key parameter for optimal SBT operation and is therefore treated in detail in this chapter.

Figure 6.12 shows the time series of the discharge $Q_{SBT}$, the bypassed fine suspended sediment transport rate $Q_{SSL_{fine}}$, the bypassed bedload transport rate $Q_{BL_{22}}$ and the reservoir water level on August 13, 2014. The SPGS data revealed three peaks of bedload transport occurring within a brief time lag after negative peaks of reservoir water level. The fine suspended sediment show a similar behavior but the peaks occurred before those of the bedload due to shorter particle transport velocities and lower thresholds for initiation of transport compared to bedload. At the beginning of the SBT operation, the transport rate of fine suspended sediment quickly increased reaching high values and decreased after 1 hour despite quasi-constant reservoir water level and discharge. Similar behaviors were observed during the other SBT operations indicating erosion of sediments deposited close to the SBT inlet.
The relation between the bedload transport rates in the SBT and the reservoir water level RWL is shown in Figure 6.13 for each SBT operation. The bedload transport rates increase with decreasing reservoir water level. Significant bedload transport in the SBT, i.e. $Q_{BL,22} \geq 1 \text{ kg/s}$ occurred at a reservoir water level below 816 masl. Significant bedload transport for $RWL > 816 \text{ masl}$ was only monitored during the flood event on August 13, 2014 when the approach flow discharge $Q_0$ was equal to the design discharge of the SBT, i.e. $Q_0 = Q_d = 170 \text{ m}^3/\text{s}$. Moreover, Figure 6.13 shows clockwise hystereses. Such hystereses are attributed to a re-entrainment and transport of sediments, deposited during the previous low flow and / or falling limb of the hydrograph (Wood 1977, Kuhnle 1992, Oehy 2003, Humphries et al. 2012, Sumi et al. 2012). When the operation of the Solis SBT is stopped, sediment deposition occurs because the bed shear stress decreases with decreasing discharge and increasing reservoir water level (cf. Figure 6.3 in Chapter 6.2.2). During the subsequent SBT operation, the deposited sediments are re-entrained and conveyed through the SBT causing the observed clockwise sediment transport hysteresis.

A different behavior was only observed during the test event on June 9, 2015 with a mean approach flow discharge $Q_0 = 78 \text{ m}^3/\text{s}$ below the threshold for SBT operation, which is 90 m$^3$/s (Figure 6.13c). Despite the decreasing $RWL$, the transport rates slightly decrease due to decreasing discharge and hence bed shear stress. This result indicates that RWL and approach flow discharge control the sediment transport in the SBT.
Results of the test case Solis SBT

Figure 6.13 Bypassed bedload transport rates as a function of the reservoir water level for all 9 SBT operations; 10-minutes averaged values

Figure 6.14a shows the operationally averaged total sediment $\bar{Q}_{TL}$ and bedload sediment transport rates $\bar{Q}_{BL22}$ in the SBT as a function of the operationally averaged reservoir water level $\bar{RWL}$ for each operation from 2012 to 2016. The $\bar{Q}_{TL}$ slowly increases from 820 masl down to 816 masl, which was recommended by VAW (2010). Below this level, $\bar{Q}_{TL}$ significantly increases with decreasing $RWL$. A similar trend is shown for $\bar{Q}_{BL22}$ despite a considerable lower magnitude. The least square fitting resulted in $\bar{Q}_{TL} = c_1/(\bar{RWL} - c_2)$ with the constants $c_1 = 575.2 \, \text{m} \cdot \text{kg/s}$ and $c_2 = 813.4 \, \text{m}$ and in $\bar{Q}_{BL22} = c_3/(\bar{RWL} - c_4)$ with the constants $c_3 = 2.30 \, \text{m} \cdot \text{kg/s}$ and $c_4 = 814.3 \, \text{m}$ for total sediment transport and bedload transport, respectively. The corresponding correlation coefficients are $R^2 = 0.48$ and $R^2 = 0.12$, which are low because the effect of approach flow discharge is neglected and only few data are available.

To account for both the effect of $RWL$ and $Q_0$, the bed shear stress in the reservoir at the pivot
Results of the test case Solis SBT point was determined and averaged for each operation. Figure 6.14b. shows $Q_{TL}$ and $Q_{BL.22}$ as a function of the operationally averaged bed shear stress $\bar{\tau}_b$ determined at the pivot point (Figure 6.3). The least square fitting resulted in $Q_{TL} = c_5 (\bar{\tau}_b - c_6)^{1.5}$ with $c_5 = 2.50\, m^{1.5}\cdot s^2/\text{kg}^{0.5}$ and $c_6 = -14.33\, \text{Pa}$ and in $Q_{BL.22} = c_7 (\bar{\tau}_b - c_8)^{1.5}$ with $c_7 = 0.21\, m^{1.5}\cdot s^2/\text{kg}^{0.5}$ and $c_8 = 2.10\, \text{Pa}$. The correlation coefficients amount to $R^2 = 0.69$ and $R^2 = 0.62$ for total sediment and bedload transport, respectively. These values are significantly higher than those of the former correlation because the effect of both reservoir water level and approach flow discharge was considered.

The total sediment transport rate at the recommended start of SBT operation, i.e. for $RWL = 816\, \text{masl}$ and $Q_0 = 90\, \text{m}^3/\text{s}$, is approx. 200 kg/s and increases with decreasing $RWL$ and increasing $\bar{\tau}_b$. The threshold $RWL$ for significant bedload transport rates, i.e. $Q_{BL.22} = 1\, \text{kg/s}$ obtained from the correlation is 816.4 masl, whereas the monitoring data suggest a slightly lower value of approx. 815.7 masl (Figure 6.14a). These findings suggest that the SBT should be operated at $RWL \leq 816\, \text{masl}$ and $Q_0 \geq 90\, \text{m}^3/\text{s}$, confirming the SBT operating conditions recommended by VAW (2010). However, the bypass efficiency will presumably be further improved by reducing the target SBT operation $RWL$.

![Figure 6.14](image.png) Operationally averaged total sediment and bedload transport rate in the SBT as a function of a) the operationally averaged reservoir water level and b) the operationally averaged bed shear stress.
Overall, the bypass efficiency of the Solis Reservoir was increased between 2012 and 2015 from $BE = 0.15$ to $BE = 0.31$ due to the 9 SBT operations. Further improvements of the bypass efficiency are expected to be achieved by adopting the revised operation conditions, as previously presented and by extending the operation duration. However, reservoir drawdown and SBT operation imply production and financial losses. The trade-off between bypass efficiency, hence increased reservoir desilting, and energy production depends on various parameters, such as the hydrograph, sediment supply, electricity market and annual reservoir sedimentation. Furthermore, it varies from operation to operation and requires real-time monitoring and controlling.

### 6.8 Hydroabrasion

The test fields were visually inspected after every SBT operation and scanned with a 3D-laser to generate abrasion maps every two years, i.e. in 2012, 2014 and 2016. The obtained abrasion depths after two and four years are presented in Figure 6.15a and b, respectively. The abrasion maps show two types of systematic errors: (1) circular undulating pattern with amplitudes of ±2 mm and (2) a mismatch of the scan hemispheres resulting in a vertical off-set of ±2 mm at the conjunction. These errors are attributed to inappropriate field conditions during the measurements, in particularly low air temperature, high humidity, and dust inside the device (Soudarissanane et al. 2011, Herath 2014). The circular pattern is device-dependent and only observed for the used particular Faro Focus scanner. Due to the small bypassed sediment volumes involving mainly small particles transported in suspension (cf. Chapter 6.5.2) the abrasion depths are expected to be smaller than the magnitude of the measurement error. Visual surveys and abrasion estimations also support this expectation (cf. Chapter 10.2.3). As a result, the determined abrasion maps mainly show the measurement error, whereas information about the abrasion depth could not be extracted. The determined abrasion patterns after two and four years are similar with slightly deviating magnitudes and implausible negative abrasion depths, see green colored areas in Figure 6.15a and b. A trend, such as increasing abrasion depth with time cannot be identified. Hence, further measurements, preferably by using a different device conducted at warmer air temperatures, are recommended.
Results of the test case Solis SBT

Figure 6.15  High-resolution abrasion maps of the Solis SBT test fields after a) two and b) four years, respectively

Since the abrasion depths cannot be quantified from the measurements, they were visually quantified. The visual inspections revealed that all test fields exhibit a smooth grinded surface without break-off of invert fragments or aggregates (cf. Figure 6.16 - Figure 6.20), associated with flat particle impact angles. This is in line with Auel (2014), who investigated the particle trajectories in supercritical flows in SBTs and reported that particle impact angles in SBTs range between 2 to 8° irrespective of the particle size. Such low impact angles are associated with low abrasion rates (cf. Figure 3.7 in Chapter 3.3.3). Further reasons for the negligible invert abrasion are low bedload transport rates, small particle sizes and brief SBT operation durations. Although only small abrasion depths occurred, certain material-dependent differences considering the abrasion pattern and abrasion depths were observed and are treated in the following.

Figure 6.16 shows the steel test field before its commissioning in 2012 (a) and one week after the flood event in August 2014 (b). The rusty-colored surface has been polished due to the sand blasting effect occurring during the SBT operation and gradually re-colored afterwards. One week after the SBT operation in August 2014, only the central zone of the test field, covered with a thin water film, remained polished (Figure 6.16b). The rusty color is caused either by precipitations of the ground water containing solute ferrous iron or / and by corrosion. The latter represents a different type of wear provoking extra material loss in addition to hydroabrasion. However, the reason for this rusty color has so far not been clarified. Closer inspections revealed no signs of comprehensive material loss. The steel test field only shows some abraded areas due surface irregularities at the welding zones (Figure 6.17a) and randomly distributed dips caused by strong bedload particle impacts (Figure 6.17b). This confirms the result of the sediment transport investigation revealing that most of the particles were transported in
Results of the test case Solis SBT suspension, whereas the share of large particles impacting the invert and causing the observed dips is small (cf. Chapter 6.5.2).

Figure 6.16 Steel test field at the Solis SBT a) rusty colored after implementation and b) with polished areas one week after the SBT operation in August 2014

Figure 6.17 Close-up view one week after the SBT operation in August 2014 revealing a) abrasion at the welding zone and b) impact marks

Figure 6.18 shows the cast basalt lining before commissioning in 2012 (a) and after three years of operation (b). In contrast to the steel test field with a random abrasion pattern, the cast basalt exhibits concentrated material losses only at the upstream edges of the tiles. The abrasion starts at the joints, exhibiting a slight negative offset, and propagates in flow direction. This indicates that hydroabrasion is triggered by surface irregularities confirming the results of Auel’s (2014) physical scale model investigation.
The Normal Concrete (NC), the High-Strength Concrete (HSC) and the Low Shrinkage Concrete (LSC) test fields show similar behavior. After three years of operation, remnants of the cement skin were still visible (Figure 6.19). The abrasion is dominated by an undulating pattern of zones without abrasion tracers, zones of partially removed cement layer and zones of exposed concrete components (Figure 6.19b). This pattern is presumably related to an uneven surface geometry due to the implementation. Outstanding areas are abraded first and cause a leveling of the surface roughness.

The High Alumina Cement concrete (HAC) and the Ultra-High Performance reinforced Concrete (UHPC) test fields show relatively less abrasion but some remnants of ground water precipitations even after three years of operation (Figure 6.20). The HAC test field mainly suffered abrasion at the working joints. Figure 6.20a shows that abrasion starts at the working joints and propagates in flow direction, similar to the abrasion characteristics observed at the cast basalt test field. The UHPC test field exhibits even less abrasion marks, i.e. only single abrasion scratches (Figure 6.20b). These observations indicate that the UHPC withstands the...
hydroabrasive impact in the Solis SBT best, whereas the HAC and the other three concretes, i.e. NC, HSC and LSC, exhibit a relatively lower abrasion resistance.

Figure 6.20 Close-up pictures after three years of operation of a) the HAC with remnants of the ground water precipitation and abrasion at the downstream edge of the working joint and b) the UHPC with remnants of the ground water precipitation and single abrasion marks

It is noted that the observed trends presented above only hold for the topmost layer of the materials, which might behave differently than the core material. Reliable statements on the abrasion resistance of the invert materials is only possible based on an extended observation duration with significant abrasion depths on the order of centimeters.
7 Results of the test case Pfaffensprung SBT

The results of the sediment transport and hydroabrasion investigations at the Pfaffensprung SBT are reported and discussed in this chapter. First, the hydraulics and hydrology in the Reuss and the Pfaffensprung SBT are specified. Second, the sediment transport estimates in the river and the SBT are presented and discussed. Finally, the results of the abrasion measurements of the different invert materials, i.e. high-strength concrete and granite pavement, are presented.

7.1 Hydraulics and hydrology

The flow velocities and depths in the Reuss and the SBT were determined based on their daily discharge hydrographs using the Manning-Strickler flow equation (Equation (3.12)) and backwater calculation (with Equation (3.8) and (3.10)), respectively. The hydraulic and geometric parameters used are listed in Table 7.1. The Strickler value for bed roughness was calculated according to Equation (3.13). The flow in the Reuss was assumed to be uniform, whereas it was non-uniform in the SBT. Figure 7.1 shows the water depth, the mean flow velocity and the Froude number in the SBT determined for its design discharge of \( Q_d = 220 \, \text{m}^3/\text{s} \). After the acceleration section, supercritical flow conditions are achieved, which slightly decelerate along the SBT but remains supercritical. With regard to the bedload transport capacity, the section with the lowest bed shear stress near the outlet is decisive and therefore considered for sediment transport calculations.

Table 7.1 Hydraulic and geometric parameters of the Reuss River and the Pfaffensprung SBT

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Reuss River</th>
<th>Pfaffensprung SBT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cross section</td>
<td>Trapezoidal Bank slope 1:1</td>
<td>Horse shoe (Chapter 0)</td>
</tr>
<tr>
<td>Channel width [m]</td>
<td>18</td>
<td>4.4</td>
</tr>
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<td>3</td>
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<tr>
<td>Roughness</td>
<td>( k_{st} = 22.5 , \text{m}^{1/3}/\text{s} )</td>
<td>( k_v = 7 , \text{mm} )</td>
</tr>
</tbody>
</table>
Results of the test case Pfaffensprung SBT

Figure 7.1 Mean flow velocity, flow depth and Froude number along the SBT at the design discharge $Q_d = 220 \text{ m}^3/\text{s}$

The daily discharge in the SBT is plotted against the corresponding daily approach flow discharge in the Reuss in Figure 7.2 for the years 2012 to 2015. The SBT is generally put in operation, when the approach flow discharge in the Reuss $Q_0$ exceeds the critical discharge for the initiation of particle motion $Q_c = 38 \text{ m}^3/\text{s}$ (cf. Chapter 7.2.1).

Figure 7.2 Discharge in the SBT as a function of approach flow discharge in the Reuss with threshold discharge for initiation of motion and SBT operation

The annual SBT operation durations and discharges in both the Reuss and in the SBT are listed in Table 7.2. On average, the SBT was in operation for $T = 118$ days per year, and bypassed a mean discharge of $Q_{SBT} = 7.4 \text{ m}^3/\text{s}$. The mean annual discharge in the Reuss was $\bar{Q}_0 = 25.8 \text{ m}^3/\text{s}$. Since hydroabrasion is governed by bedload transport, which occurs at approach flow discharges of $Q_0 > Q_c = 38 \text{ m}^3/\text{s}$ (cf. Chapter 7.2.1), only SBT operations above this threshold discharge were considered for the hydroabrasion analysis (Chapter 10.2). Sklar and Dietrich (2004) found that moderate discharges, which frequently occur, are the most efficient events regarding bedrock abrasion. Therefore, the integrated effect of the full range of
hydraulic and sediment transport conditions is assumed to be represented by the averaged values considering only bedload-effective discharges, i.e. \( Q_0 > 38 \text{ m}^3/\text{s} \) (denoted by the star). The corresponding averaged operation duration, mean approach flow and bypassed discharge were \( T^* = 76 \text{ d} \), \( Q_0^* = 74.9 \text{ m}^3/\text{s} \) and \( Q_{SBT}^* = 32.1 \text{ m}^3/\text{s} \), respectively. The mean flow depth, flow velocity and Shields parameter in the SBT were \( h^* = 0.75 \text{ m} \), \( U^* = 9.79 \text{ m/s} \) and \( \theta^* = 0.04 \), respectively.

<table>
<thead>
<tr>
<th>Table 7.2</th>
<th>Annual SBT operation duration and hydraulic conditions in the Pfaffensprung SBT for the years 2012 to 2015</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2012</td>
</tr>
<tr>
<td>All year</td>
<td>2012</td>
</tr>
<tr>
<td>( T ) [d]</td>
<td>113</td>
</tr>
<tr>
<td>( Q_0 ) [m(^3)/s]</td>
<td>27.9</td>
</tr>
<tr>
<td>( Q_{SBT} ) [m(^3)/s]</td>
<td>9.8</td>
</tr>
<tr>
<td>( T^* ) [d]</td>
<td>91</td>
</tr>
<tr>
<td>( Q_0^* ) [m(^3)/s]</td>
<td>71.2</td>
</tr>
<tr>
<td>( Q_{SBT}^* ) [m(^3)/s]</td>
<td>35.5</td>
</tr>
<tr>
<td>( h^* ) [m]</td>
<td>0.82</td>
</tr>
<tr>
<td>( U^* ) [m/s]</td>
<td>9.8</td>
</tr>
<tr>
<td>( \theta^* ) [-]</td>
<td>0.04</td>
</tr>
</tbody>
</table>

### 7.2 Sediment transport in the Reuss River

The results of the sediment transport estimates in the Reuss are presented in this section. First, the initiation of bedload transport was investigated. Then, the method to determine the bedload transport capacity in the Reuss was validated based on the results of a numerical investigation. Finally, the validated method was applied to calculate the bedload transport capacities and effective bedload transport rates in the river. The sediment masses were converted to volumes assuming a sediment density of \( \rho_s = 2.65 \text{ to/m}^3 \).

#### 7.2.1 Bedload transport initiation in the Reuss River

The initiation of bedload transport depends on the existence of an armor layer. According to the criteria introduced in Chapter 3.2.2, the Reuss exhibits an armor layer (Table 7.3). The critical Shields parameter for initiation of bedload transport was accordingly adapted by applying Equation (3.19) with \( d_{m,substrat} = 250 \text{ mm} \) and \( d_{m,armor} = 300 \text{ mm} \). The resulting critical Shields parameter is \( \theta_c' = 0.049 \). This result agrees with literature and was therefore used for
the bedload transport estimation in the Reuss (Meyer-Peter and Müller 1948, Fernandez Luque and Van Beek 1976, Princevic and Lamb 2015). The corresponding critical discharge is $Q_c = 38 \text{ m}^3/\text{s}$.

Even if hydraulic conditions remain below the threshold for the initiation particle motion, bedload transport can occur in case of sediment supply from upstream reaches or suction removal of particles from between the armor layer blocks. The former is not considered herein, for the latter Equation (3.20), introduced Sumer et al. (2001), is used to determine the critical Shields parameter for suction removal. For the Reuss it is $\theta_{sw} = 0.31$. The ratio $\theta_{sw}/\theta_c' = 6.3$ is considerably larger than $d_{m,\text{armor}}/d_{m,\text{substrat}} = 1.2$. Therefore, suction removal does not take place and was not considered for the bedload estimations.

Table 7.3 Criteria for armor layer building and their application to the Reuss

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Value</th>
<th>Armor layer</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gessler (1965)</td>
<td>$d_{84}/d_{50} &gt; 2.0$</td>
<td>2.8</td>
</tr>
<tr>
<td>Little and Mayer (1976)</td>
<td>$(d_{84}/d_{16})^{1/2} &gt; 1.3$</td>
<td>4.9</td>
</tr>
<tr>
<td>Chin (1985)</td>
<td>$d_{84}/d_{50} &gt; 1.5 &amp; d_{max}/d_{50} &gt; 1.8$</td>
<td>2.8 &amp; 7.4</td>
</tr>
<tr>
<td>Schöberl (1992)</td>
<td>$(d_{84}/d_{16})^{1/2} &gt; 1.35, d_{90}/d_{50} &gt; 1.55 &amp; d_{m}/d_{50} &gt; 1.05$</td>
<td>4.9, 3.5 &amp; 1.4</td>
</tr>
<tr>
<td>Günter (1971)</td>
<td>$(d_{84}/d_{16})^{1/2} &gt; 1.4 &amp; 1.6$</td>
<td>4.9</td>
</tr>
</tbody>
</table>

7.2.2 Bedload transport capacity in the Reuss River

The bedload transport capacity in the Reuss was determined by applying Equations (4.3) to (4.6) to the hydrograph of 2012 to 2015. Bedload transport is assumed to initiate at $\theta > \theta_c' = 0.049$, since an armor layer exists and no suction removal takes place.

The method to estimate the bedload transport capacity was validated based on a former investigation on the bedload transport in the Reuss during the extreme flood event on August 24/25, 1987 (VAW 1992). The sediment transport was numerically investigated and validated by field surveys (Bezzola et al. 1991, Bezzola 1991, VAW 1992). The investigation covered the entire river section between Andermatt and the Lake of Lucern, stretching 9 km up- and 25 km downstream of the Pfaffensprung SBT, respectively. It was reported that the bedload transport rate in the Reuss at the Pfaffensprung SBT inlet was equal to its capacity due to the extra sediment supplied from the tributary called Maienreuss located 400 m upstream of the SBT. The bedload transport volume in the river at Pfaffensprung during this flood event amounted to $75 \cdot 10^3 \text{ m}^3 - 80 \cdot 10^3 \text{ m}^3$ (VAW 1992). This corresponds to $199 \cdot 10^3 \text{ m}^3 - 212 \cdot 10^3$ to of sediment. The average value, i.e. $205 \cdot 10^3$ to was used as a reference herein.
The bedload transport rates during the flood event on August 1987 were re-calculated in the present study as described in Chapter 4.1.3. The input parameters such as river topography and sediment GSD are reported in Chapter 4.4.1. The hydrograph of the flood event, shown in Figure 7.3, was provided by VAW (1992).

![Hydrograph of the approach flow discharge in the Reuss at Pfaffensprung during the flood event in August 1987; adopted from VAW (1992)](image)

The resulting bedload masses of both the previous (= reference) and the present study are summarized in Table 7.4. The results obtained from different equations deviate less than 20% from the reference, indicating that they provide comparable and realistic estimates for high discharges. However, considerably larger differences between the different equations are observed at lower discharges. Figure 7.4 illustrates the gravimetric bedload transport capacities obtained from the different equations as a function of discharge. The values obtained from (R) exceed the estimates of (SJ), (P) and (C) at discharges of $Q_0 = 38 - 170 \text{ m}^3/\text{s}$ carrying most of the annual bedload. Therefore, (R) was not used herein due to the presumed overestimation. Of the remaining formulae, the (SJ) leads to the highest transport capacities for $Q_0 = 38 - 70 \text{ m}^3/\text{s}$, and to the lowest values at higher discharges. Therefore, the average of (SJ), (P) and (C) was used in order to minimize the estimation error. The resulting bedload transport capacity of the flood that occurred in August 1987 amounts to $216 \cdot 10^3 \text{ t}$. This value deviates only 5% from the reference, so that the applied approach to estimate bedload transport capacity is assumed to result in realistic estimations for the full range of discharge and was therefore used in this study.
Table 7.4  Gravimetric bedload transport capacity during the flood occurring in August 1987 estimated based on different equations and result of the previous study, serving as a reference

<table>
<thead>
<tr>
<th>Reference / Equation</th>
<th>Bedload transport capacity [$10^3$ to]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reference study (VAW 1992)</td>
<td>205</td>
</tr>
<tr>
<td>(R)</td>
<td>249</td>
</tr>
<tr>
<td>(SJ)</td>
<td>167</td>
</tr>
<tr>
<td>(P)</td>
<td>219</td>
</tr>
<tr>
<td>(C)</td>
<td>263</td>
</tr>
<tr>
<td>Average of (SJ), (P), (C)</td>
<td>216</td>
</tr>
</tbody>
</table>

Figure 7.4  Gravimetric bedload transport capacity as a function of approach flow discharge in the Reuss for different bedload equations

7.2.3  Effective bedload transport in the Reuss River

The effective bedload transport rate in a natural river is generally smaller than the bedload transport capacity due to limited sediment supply (Auel et al. 2016a). A previous study revealed that the ratio between bedload transport and bedload transport capacity in the Reuss amounts to $Q_s/Q_s^* = 0.80$ if no side erosion or extraordinary sediment supplies from tributaries occur (VAW 1992). This ratio was used in this study since no flood event with extraordinary sediment supplies occurred between 2012 and 2015. The estimated annual bedload transport masses and volumes between 2012 and 2015 are listed in Table 7.5. The mean annual bedload mass and volume amounted to $BL = 350 \cdot 10^3$ to and $BL_v = 132 \cdot 10^3$ m$^3$, respectively. This value is 32% higher than $BL_v = 100 \cdot 10^3$ m$^3$ reported by Studer (1925/26). However, this is a long-term average value, from which the 4 year average can significantly deviate. Moreover, the effective
bedload transport rates are supposed to have increased in the recent decades due to climate change effects. Therefore, the method applied herein to estimate the effective bedload transport rates can be assumed to result in realistic values.

Table 7.5 Estimated bedload transport masses and volumes in the Reuss from 2012 to 2015

<table>
<thead>
<tr>
<th>Year</th>
<th>2012</th>
<th>2013</th>
<th>2014</th>
<th>2015</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bedload transport mass $BL$ [$10^4$ to/yr]</td>
<td>457</td>
<td>366</td>
<td>145</td>
<td>432</td>
<td>350</td>
</tr>
<tr>
<td>Bedload transport volume $BL_v$ [$10^3$ m$^3$/yr]</td>
<td>173</td>
<td>138</td>
<td>55</td>
<td>163</td>
<td>132</td>
</tr>
</tbody>
</table>

7.2.4 Total sediment transport in the Reuss River

The total sediment transport in the Reuss is treated in this subsection. The total sediment load ($TL$) consists of bedload ($BL$) and suspended sediment load ($SSL$). The former was treated in Chapter 7.2.3. The latter was estimated based on two different methods (Chapter 3.2.7) since no monitoring of suspended sediment transport exists:

(I) Application of specific suspended sediment supply rate to the catchment,

(IIa) Application of a BL/SSL ratio of a comparable catchment to the estimated bedload masses, and

(IIb) Application of the BL/SSL ratio of the Reuss, obtained by a former study (VAW 1992) to the estimated bedload volumes.

(I) The specific annual suspended sediment supply of the Reuss Catchment was determined in Seedorf at the FOEN gauging station number 2056, located 25 km downstream of the SBT. The catchment of the Reuss in Seedorf is $A = 833$ km$^2$ and the measured mean annual suspended sediment load between 2004 and 2013 was $SSL = 155 \cdot 10^3$ to (BAFU 2013). The specific suspended sediment supply accordingly resulted in $SSL/A = 186$ to/km$^2$. By applying this value to the catchment area of the Reuss at Pfaffensprung ($A = 390$ km$^2$), the mean annual suspended sediment mass amounts to $SSL = 73 \cdot 10^3$ to. The corresponding bedload to suspended load ratio at the Pfaffensprung for the years 2012 to 2015 results in $BL/SSL = 1:0.2$.

(IIa) Typical $BL/SSL$ ratios for Swiss torrents vary from 1:1 to 1:2 (Rickenmann 2001, Turowski et al. 2010). By applying their average, i.e. $BL/SSL = 1:1.5$ to the mean estimated bedload mass at Pfaffensprung (cf. Chapter 7.2.3), the mean annual suspended sediment load from 2012 to 2015 amounts to $SSL = 525 \cdot 10^3$ to.

(IIb) The sediment transport during the flood event in August 1987 was investigated in a former study revealing that 45% of the sediment transported through the Reuss Valley was in
Results of the test case Pfaffensprung SBT

The resulting $BL/SSL$ ratio amounts to 1:0.8. By applying this ratio to the mean estimated annual bedload mass, the mean annual suspended sediment mass results in $SSL = 280 \cdot 10^3$ to.

Method I results in a 3 to 5 times lower mean annual $SSL$ compared to Method II. The corresponding $BL/SSL = 1:0.2$ obtained from Method I is significantly larger than typical values for Alpine rivers with $BL/SSL = 1:0.5 - 1:11$ (cf. Chapter 3.2.7), whereas the $BL/SSL$ ratios obtained from Method IIa and IIb vary between 1:1.5 and 1:0.8 and hence are in good agreement with literature data. Therefore, $BL/SSL = 1:1$, which lays in-between these two ratios, is applied in this study. The total sediment transport is thus twice the bedload transport, amounting to $TL = SSL + BL = 700 \cdot 10^3$ to for the period between 2012 and 2015.

### 7.3 Sediment transport through the Pfaffensprung SBT

The result of the bedload transport estimation in the SBT is presented in this subsection. It should be noted that suspended sediment transport is of minor importance regarding hydroabrasion and therefore not considered herein.

The possibility of SBT clogging due to sediment aggradation in the SBT was evaluated by comparing the bedload transport capacity of the SBT with that of the Reuss (cf. Chapter 7.2.2). The bedload transport capacity in the Pfaffensprung SBT was calculated by applying different bedload transport equations, considering their application ranges. These are: (SJ) Equation (3.49), (P) Equation (3.47), (NN) Equation (3.48), and (A) Equation (3.50). Figure 7.5 shows the resulting transport capacities in the SBT and the river as a function of discharge. Irrespective of the applied equation, the bedload transport capacity in the SBT considerably exceeds that in the river, so that SBT clogging is not expected. This result is in good agreement with the operator’s experiences, provided that large trunks and branches did not enter the SBT.

The design discharge capacity of the SBT was never exceeded during the observation period. This means that the entire bedload mass supplied by the river (Chapter 7.2.3) is supposed to be diverted by the SBT past the dam. Therefore, the estimated annual bedload transport masses listed in Table 7.5 were expected to represent the annual bedload masses in the SBT and used for the hydroabrasion analysis (Chapter 7.5 and 10.2).

The mean annual bypassed mass and volume amounts to $BL = 350 \cdot 10^3$ to and $BL_v = 132 \cdot 10^3$ m$^3$, respectively (Table 7.5). The rolling and suspension probability of the mean particle size was $P_R = 0.30$ (Equation (3.21)) and $P_{su} = 0.00$ (Equation (3.23)), respectively. This indicates that
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Saltation was the dominating transport mode. The corresponding particle hop length obtained from Equation (3.33) using the parameters listed in Table 7.2 was \( L_p = 2.70 \text{ m} \). This value was used to discuss the abrasion pattern in Chapter 7.5.1.

![Figure 7.5](image-url) Gravimetric bedload transport capacity in the SBT after Novak & Nalluri (NN), Auel (A) and Smart & Jäggi (SJ) and in the river as a function of discharge

### 7.4 Bypass and trap efficiency of the Pfaffensprung Reservoir

Continuous sediment monitoring does not exist at the Pfaffensprung Reservoir, but the sediments deposited in the reservoir are monitored and removed by flushing and excavation once a year. Therefore, the bypass efficiency \( BE \) of the Pfaffensprung Reservoir was determined based on the estimated annual sediment transport masses in the river and the measurement of the annually removed sediments. The latter were assumed to be equal to the annually accumulated sediment masses. The total sediment outflow was determined by the difference between the total inflowing sediment \( TL_{in} \) and the total accumulated sediment \( TL_{acc} \), i.e. \( TL_{out} = TL_{in} - TL_{acc} \). The bypass efficiency follows as \( BE = TL_{out}/TL_{in} = (TL_{in} - TL_{acc})/TL_{in} \).

Table 7.6 lists the annual sediment masses used to determine the bypass efficiency of the Pfaffensprung Reservoir with and without the SBT. The mean annual sediment inflow between 2012 and 2015 was \( TL_{in} = 700 \cdot 10^3 \) to. During the same period \( TL_{acc} = 13 \cdot 10^3 \) to per year were removed (SBB 2009, 2015). The resulting bypass efficiency of \( BE = 0.98 \) is relatively high compared to other reservoirs with a SBT in operation with \( BE = 0.60 - 0.90 \) (cf. Chapter 2.4.3). This is attributed to the location of the SBT intake at the reservoir head and the design of the guiding structure. The latter is a more than 2 m high masonry weir efficiently hindering the sediments to enter the reservoir. Moreover, the operation regime accounting for the initiation
Results of the test case Pfaffensprung SBT

of bedload transport in the Reuss contributes to this high bypass efficiency. The SBT is operated when the approach flow discharge exceeds $Q_0 = 40$ m$^3$/s, which is close to the threshold discharge for initiation of bedload transport $Q_c = 38$ m$^3$/s. As a result, the SBT is in operation several weeks per year, bypassing the sediments carried by the river without intermediate deposition and hence effectively avoiding reservoir sedimentation.

The corresponding mean trap efficiency between 2012 and 2015 amounts to $TE = 1 - BE = 0.02$. Comparison of the result with Brune’s (1953) criteria (cf. Equations (2.1), (2.2) and (2.3)) is not possible, since the $CIR = 2.6 \times 10^{-4}$ yr of the Pfaffensprung Reservoir is beyond the application range of those criteria.

The effect of the SBT on the bypass efficiency of the Pfaffensprung Reservoir with a capacity of $CAP = 170'000$ m$^3$ is assessed based on the sediment estimates between 2012 and 2015. For the sake of simplicity the bypass efficiency of the suspended sediments with and without SBT were assumed to be equal. This is an acceptable simplification due to the low $CIR = 2.6 \times 10^{-4}$ yr of the Pfaffensprung Reservoir associated with a low trap efficiency in particular for the suspended sediments. In contrast, all bedload particles are assumed to settle in the reservoir. Based on these assumptions, the bypass efficiency without the SBT results in $BE = 0.52$. The SBT hence considerably decreased the annual accumulation volume from $363'000$ to/yr / $1.8$ to/m$^3$/yr = $202'000$ m$^3$ to $13'000$ to/yr / $1.8$ to/m$^3$ = $7'000$ m$^3$/yr, increased the bypass efficiency by a factor of 2 and prolonged the reservoir life time by $202'000$ m$^3$/7'000 m$^3 = 28$ times from less than 1 to 24 years, assuming a bulk density of 1.8 to/m$^3$.

Table 7.6 Mean annual sediment inflow and outflow of the Pfaffensprung Reservoir and estimated bypass efficiencies between 2012 and 2015 with and without SBT

<table>
<thead>
<tr>
<th></th>
<th>Without SBT</th>
<th>With SBT</th>
</tr>
</thead>
<tbody>
<tr>
<td>SSL</td>
<td>350</td>
<td>337</td>
</tr>
<tr>
<td>BL</td>
<td>350</td>
<td>0</td>
</tr>
<tr>
<td>TL</td>
<td>700</td>
<td>337</td>
</tr>
</tbody>
</table>
7.5 Hydroabrasion

Annually conducted laser scans allow for high-resolution monitoring of the invert abrasion development over time and space. The high-resolution abrasion maps of the concrete test fields and of the granite pavements from 2012 to 2015 are presented and discussed in the following. Finally, the abrasion depths of the tested invert materials were cross-compared with the operating conditions in the SBT to determine their relative abrasion resistance.

The abrasion maps show two types of systematic errors: (I) a circular pattern with 2 m in diameter and (II) a regular repeating radial pattern (Figure 7.8, Figure 7.7, Figure 7.14 and Figure 7.13). These errors originate from beam deflection at the device casing and from repeating self-adjustment of the laser device. The magnitude of the errors amounts to ± 3 mm. Their effects on the determined abrasion depths is large for small abrasion depths close to the error range. However, the relative errors decrease with increasing abrasion depth and are even lower for spatially averaged values, because the errors are either limited to small areas (Error I) or are leveled out (Error II).

7.5.1 Concrete test fields

Close-up pictures of the test field C1 and C2 are shown in Figure 7.6a and b, respectively. It shows that the cement was washed away, the coarse aggregates were abraded, and a few break-offs of invert fragments and aggregates occurred. The observed abrasion pattern indicates that grinding, associated with the parallel component of the kinetic energy of the impacting particles, was the dominant harming process. This is in line with Auel’s (2014) physical scale model test results, revealing that the particle impact angle in supercritical flows in SBTs range from 2° to 8° only.

![Figure 7.6 Close-up view of the abraded surface of a) the C1 test field after four operational years and b) the C2 test field after two operational years](image-url)
Figure 7.7 and Figure 7.8 show the obtained high-resolution abrasion maps of the concrete test fields C1 and C2, respectively. The test fields were implemented in two and three steps, respectively. The working joints were visible after implementation and appear in the abrasion maps as thin lines at $x = 4.2$ m in Figure 7.7 and at $x = 5.8$ m and $x = 13$ m in Figure 7.8. Despite this, the material loss was not significantly higher at the working joints compared to the rest of the test field, indicating a proper implementation and curing procedure.

The abrasion maps of both concrete test fields clearly show an abrasion concentration on the orographic right side (Figure 7.7 and Figure 7.8). This is attributed to the effect of Prandtl’s first type of secondary currents (Chapter 3.1.3) due to the tunnel bend. The bending effect was expected to be strong at C2 as it is located in the tunnel bend (Figure 7.8). However, its effect propagated in flow direction until C1 located 180 m downstream from the bend near the outlet (Figure 7.7). This finding is of particular interest regarding the design of SBTs and optimal material usage of the invert lining (Chapter 11.6).

The abrasion at both concrete test fields show a characteristic undulating pattern, which might be governed by the hop lengths of saltating bedload particles (Auel 2014). Although the abrasion wave length of $l_w = 1.25 \pm 0.6$ m does not scale with the particle hop length $L_p = 2.70$ m (cf. Chapter 7.3), it does scale with approx. half of $L_p$. It is possible that the abrasion pattern was triggered by several surface irregularities resulting in a superposition of abrasion patterns reducing the observed abrasion wave length. Moreover, the hydraulic and sediment transport conditions in the SBT were not constant but fluctuated, so that variations in particle hop length are expected to affect the abrasion pattern. More research on the relation between particle motion and abrasion pattern is needed for a better understanding of the abrasion characteristics.

The super-elevated cross sectional abrasion profiles of the concrete test fields C1 and C2 averaged along the test field length are shown as a function of time in Figure 7.9a and b, respectively. The abrasion depths in both test fields increased over time. However, the abrasion depths of both concrete test fields is considerably higher in the first compared to the subsequent years. Despite differences were partly caused by different bedload masses, model uncertainties and measurement errors, the different abrasion rates in C1 and C2 for the same year indicate different abrasion behaviors depending on penetration depth. Aggregates can significantly enhance the concrete strength, so that their strength-increasing effect is inexistent at the topmost concrete layer but increases as the abrasion penetrates the core material (Kunterding 1991, Haroske 1998, Horszczaruk 2005, Vogel 2011).
Figure 7.7  High-resolution abrasion maps of the 10 m long C1 test field after a) one, b) two and c) four operational years (the gray colored areas could not be scanned due to ground water drainage and construction site installations)

Figure 7.8  High-resolution abrasion maps of the 20 m long C2 test field located at the tunnel bend after a) one and b) two operational years (the gray colored areas could not be scanned due to ground water drainage and construction site installations)
The maximum abrasion depth after the first operational year occurred at \( y = 1.6 \) m at both test fields indicating the location of high bed shear stresses (Auel 2014). This is attributed to the effect of Prandtl’s first type of secondary currents (Chapter 3.1.3). These currents initiated the abrasion and the formation of the abrasion channel, which in return stabilized the secondary currents. With increasing operation duration, more sediment was transported and concentrated in this channel, whereby the location of the maximum abrasion depth at test field C1 moved towards the right tunnel wall. These results reveal a self-intensifying process of hydroabrasion. Although the two concretes, i.e. C1 and C2, exhibit similar material properties they vary slightly, so that differences in the abrasion behavior due to different material properties cannot be excluded.

Figure 7.9 Super-elevated cross sectional abrasion profiles averaged along the test field length of the concrete test fields a) C1 and b) C2; view in flow direction

The mean aspect ratio was \( b/h = 6.3 \), so that Prandtl’s second type of secondary causing incision channels along the tunnel walls were expected in the straight tunnel section, where the test field C1 was installed. However, the abrasion profile in C1 does not show such incision channels. This is likely due to the relative strong effect of Prandtl’s first type of secondary currents on the flow propagating downstream and diminishing the effect of the Prandtl’s second type of secondary currents (Chapter 3.1.3, Auel 2014). Hence, the effect of Prandtl’s first but not second type of secondary currents dominated the abrasion pattern in C1.

The spatial dissimilarity of the concrete compression strength and its effect on abrasion were investigated at the test field C1. Figure 7.10 shows the 3 years cumulative abrasion depths at test field C1 as a function of the compressive strength, measured with a Schmidt Hammer (Figure 4.29). The data were subdivided into three categories according to the distance to the
right tunnel wall, i.e. $y = 0.7 \text{ m}$, $y = 1.7 \text{ m}$ and $y = 2.7 \text{ m}$, to account for the lateral distribution of bedload transport rate. The closer to the right wall, the higher the abrasion due to the bending effect (Chapter 3.1.3). Also, a slight decrease of abrasion values with increasing compressive strength for a given transverse distance from the wall is visible. However, the data largely scatter and a distinct relation between abrasion depth and compression strength cannot be extracted. A reason for this is the fact that the Schmidt Hammer measurement include a significant error because of the rough invert surface and the structural weakening of the abraded invert (cf. Chapter 4.4.3). Another reason might be the fact that abrasion is indeed triggered by weak spots but propagates both vertically and transversally in flow direction. As a result, the largest abrasion depths occur downstream of the weak spot.

![Figure 7.10 Abrasion depths as a function of compressive strength after three operational years](image)

The mean and maximum (= 95%-percentile) cumulative abrasion depths, i.e. $a_m$ and $a_{max}$, of the test field C1 and C2 are shown as a function of the cumulative bedload mass $BL$ for the years 2012 to 2015 in Figure 7.11a and b, respectively. The dotted lines represent the error margin including the bedload estimation error ($\pm 50\%$) and the abrasion measurement error ($\pm 2 \text{ mm}$). As expected, the mean and maximum abrasion depths increase with increasing bedload mass and follow a linear trend with $R^2 = 0.86$ and $R^2 = 0.78$, respectively. The ratio between maximum and mean abrasion depth defined by the gradients of the fits amounts to $43.8/25.0 = 1.8$, which is in agreement with Auel’s (2014) results from physical scale model tests.
7.5.2 Granite test fields

The abrasion patterns on both granite test fields are similar but different from those of the concrete test fields. Figure 7.12 shows close-up pictures of the G1 test field after four operational years. The material losses mainly occurred along the longitudinal joints and upstream edges of the granite blocks. The abrasion increased both vertically and transversally in flow direction and caused abrasion shadows downstream of the initial abrasion. Similar patterns were also observed in the Solis SBT (cf. Chapter 6.8) and Auel’s (2014) physical model investigations revealing that abrasion is triggered by surface irregularities.

Figure 7.12 Close-up view of the abraded surface of the G1 test field after four operational years showing abrasion concentration along joints with abrasion shadows in the downstream

Figure 7.13 and Figure 7.14 show the abrasion maps of the granite test fields G1 and G2, respectively. After the first operational year the measurements show implausible negative abrasion depths (colored in blue) because the abrasion depths were in the range of the measurement error. As a result, the measurements can hardly be interpreted. However, with
increasing operation duration the abrasion depths increased, so that this problem was overcome and the measurements confirmed the visual surveys, revealing strong abrasion concentrations along the joints. Moreover, abrasion depths at the orographic right side are slightly higher than at the left side and are in line with the lateral abrasion distributions observed at the concrete test fields.

Figure 7.13 High-resolution abrasion maps of the 10 m long and 4.4 m wide G1 test field after a) one, b) two and c) four operational years (the gray colored areas could not be scanned due to ground water drainage and construction site installations)
Results of the test case Pfaffensprung SBT

Figure 7.14  High-resolution abrasion maps of the 20 m long and 4.4 m wide G2 test field after a) one and b) two operational seasons (blue colored points mark theoretically negative abrasion depths, the gray colored areas could not be scanned due to ground water drainage and construction site installations)

Figure 7.15a and b show the cumulative mean and maximum (= 95%-percentile) abrasion depths, i.e. $a_m$ and $a_{max}$, of the test fields G1 and G2 as a function of cumulative bedload mass $BL$ for the years 2012 to 2015. The dotted lines represent the error of the bedload estimates and the abrasion measurement. The mean and maximum abrasion depths increase with both time and transported bedload mass and follow a linear trend with $R^2 = 0.87$ and $R^2 = 0.62$, respectively. The ratio between maximum and mean abrasion depth defined by the gradients of the linear fits amounts to $8.2/4.0 = 2.1$. This value is slightly higher compared to concrete due to the modular invert system leading to different abrasion characteristics and patterns.
7.5.3 Summary

The abrasion depths of both concrete and granite increased with increasing bedload transport mass, and the observed lateral abrasion distribution indicates a strong bedload transport concentration induced by the bending effect. The high-resolution abrasion maps and visual surveys in the SBT revealed different material-dependent abrasion characteristics. The concrete test fields showed an undulating abrasion pattern, while the granite pavements mainly suffered abrasions along the longitudinal joints and upstream edges of the blocks. Despite this, the ratios between maximum and mean abrasion depths of concrete and granite are in a comparable range.

Overall, the results show that the abrasion rates of the granite pavement was on average approx. 6 times lower compared to those of the concrete. However, reliable prediction of the long-term behavior of the tested materials is difficult based on such short observation periods of 2 to 4 years. Therefore, extended observation durations to analyze the effect of increasing structural weakening and bed roughness on the abrasion processes are recommended.
8 Results of the test case Runcahez SBT

In this chapter, the results of sediment transport and hydroabrasion investigations at the Runcahez SBT are reported and discussed. First, the discharge and the sediment transport in the Rein da Sumvitg are determined and discussed. Second, the sediment transport in the SBT is treated. Finally, the results of the abrasion monitoring and the relation between hydroabrasive impact and abrasion of the different invert materials are presented.

8.1 Hydrology and hydraulics

The approach flow discharges in the Rein da Sumvitg at Runcahez were determined by scaling the 15-minutes discharge measurements of the FOEN gauging station at Encardens situated 3.5 km upstream (Chapter 4.5.1). To account for the site-specific discharge characteristics, including flood events during SBT operations, the discharge scaling (Method II described in Chapter 4.5.1) was applied. Figure 8.1a shows the peak discharges in the Rein da Sumvitg at Runcahez as a function of the peak discharges at Encardens for the years 1996 to 1999 reported in Jacobs et al. (2001). The data correlate and the least square fit results in \( Q_0 = 4.4 Q^* \) with \( R^2 = 0.52 \) and \( \sigma = 13.8 \text{ m}^3/\text{s} \). The scaling ratio of 4.4 is 1.7 times larger than the catchment area-based ratio of 2.6 (Method I described in Chapter 4.5.1). This result indicates that catchment-based scaling would underestimate the discharges in Runcahez by 1.7 times, and therefore was not used in this study.

According to the operator, the SBT is operated at approach flow discharges of \( Q_0 \geq 35 \text{ m}^3/\text{s} \) (Axpo 2011). However, the measurements from 1996 to 1999 show considerable lower and higher values (Jacobs et al. 2001). On average, the SBT was in operation at \( Q_0 \geq 45 \text{ m}^3/\text{s} \) lasting for more than 2.5 hours. This criteria was applied herein to the whole observation period between 1996 and 2014. As long as the approach flow discharge was below the design discharge of the SBT, i.e. \( Q_0 \leq Q_d = 110 \text{ m}^3/\text{s} \), the entire approach flow discharge was assumed to be bypassed. Beyond this limit, the surplus discharge was assumed to enter the reservoir. The resulting operating regime applied herein is shown in Figure 8.1b.
Figure 8.1  a) Approach flow discharge in the Rein da Sumvitg at Runcahez as a function of discharge in the Rein da Sumvitg at the gauging station Encardens and b) relation between approach flow discharge and discharge in the SBT

The flow velocities and depths in the Rein da Sumvitg and in the Runcahez SBT were calculated based on 15 minutes hydrograph data using the Manning-Strickler flow equation (Equation (3.12)) and backwater calculation (with Equation (3.8) and (3.10)), respectively. The hydraulic and geometric parameters used are listed in Table 8.1. The Strickler value for bed roughness was calculated according to Equation (3.13). In contrast to the river, where the flow was assumed to be uniform, the flow is non-uniform in the SBT. Figure 8.2 shows the water depth, the flow velocity and the Froude number in the SBT determined for the SBT design discharge of $Q_d = 110 \text{ m}^3/\text{s}$. After the acceleration section, supercritical flow conditions are achieved. The flow slightly decelerates but remains supercritical along the SBT (Figure 8.2).

Table 8.1  Hydraulic and geometric parameters of the Rein da Sumvitg and the Runcahez SBT

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Rein da Sumvitg River</th>
<th>Runcahez SBT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cross section</td>
<td>Trapezoidal</td>
<td>archway (Chapter 4.5.1)</td>
</tr>
<tr>
<td>Bank slope [1:1]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Channel width [m]</td>
<td>15</td>
<td>3.8</td>
</tr>
<tr>
<td>Slope [%]</td>
<td>3.65</td>
<td>1.4</td>
</tr>
<tr>
<td>Roughness</td>
<td>$k_o = 23.5 \text{ m}^{1/3}/\text{s}$</td>
<td>$k_o = 7 \text{ mm}$</td>
</tr>
</tbody>
</table>
Results of the test case Runcahez SBT

Figure 8.2  Mean flow velocity, flow depths and Froude number along the SBT at the design discharge $Q_d = 110 \text{ m}^3/\text{s}$

The mean SBT operation duration and hydraulic conditions for the years 1996 to 2000 and 2001 to 2014 are listed in Table 8.2 and were used for the hydroabrasion analysis (cf. Chapter 10.2).

On average, the SBT was in operation for $\bar{T} = 1.5 \text{ d per year bypassing } \bar{Q}_{SBT} = 56 \text{ m}^3/\text{s}$. The mean flow depth, flow velocity and Shields parameter in the SBT were $\bar{h} = 2.0 \text{ m}$, $\bar{U} = 7.4 \text{ m/s}$ and $\bar{\vartheta} = 0.04$, respectively.

Table 8.2  Mean annual SBT operation duration and hydraulic conditions in the Runcahez SBT between 1996 - 1999 and 2000 - 2014

<table>
<thead>
<tr>
<th></th>
<th>1996 - 1999</th>
<th>2000 - 2014</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\bar{T}$</td>
<td>1.63</td>
<td>1.37</td>
<td>1.50</td>
</tr>
<tr>
<td>$\bar{Q}_0$</td>
<td>6.6</td>
<td>6.7</td>
<td>6.7</td>
</tr>
<tr>
<td>$\bar{Q}_{SBT}$</td>
<td>56.4</td>
<td>55.7</td>
<td>55.9</td>
</tr>
<tr>
<td>$\bar{h}$</td>
<td>2.01</td>
<td>2.00</td>
<td>2.00</td>
</tr>
<tr>
<td>$\bar{U}$</td>
<td>7.38</td>
<td>7.35</td>
<td>7.35</td>
</tr>
<tr>
<td>$\bar{\vartheta}$</td>
<td>0.04</td>
<td>0.04</td>
<td>0.04</td>
</tr>
</tbody>
</table>

8.2  Sediment transport in the river

The results of the sediment transport investigation in the river and the SBT are presented in this subchapter. First, the initiation of bedload transport was determined. The bedload transport in the river was calculated and is discussed next. The volumetric sediment loads were determined assuming a sediment density of $\rho_s = 2.65 \text{ t/m}^3$. 

8.2.1 Bedload transport initiation in the Rein da Sumvitg River

According to the different criteria introduced in Section 3.2.2, the Rein da Sumvitg exhibits an armor layer (Table 8.3). The critical Shields parameter for bedload transport was accordingly adapted using Equation (3.19) with $d_{m,substrat} = 230$ mm and the armor layer $d_{m,armor} = 300$ mm. The resulting critical Shields parameter for initiation of bedload transport is $\theta_c' = 0.058$. This value is in line with literature and therefore used to estimate the bedload transport in the Rein da Sumvitg (Meyer-Peter and Müller 1948, Fernandez Luque and Van Beek 1976, Prancevic and Lamb 2015). The corresponding critical discharge representing the start of bedload transport is $Q_c = 33$ m$^3$/s.

Even if hydraulic conditions remain below the threshold for the initiation particle motion, bedload transport can occur in case of sediment supply from upstream reaches or suction removal of particles from between the armor layer blocks. The former is not considered herein, for the latter Equation (3.20), introduced Sumer et al. (2001), is used to determine the critical Shields parameter for suction removal. For the Rein da Sumvitg it is $\theta_{su} = 0.31$. The ratio $\theta_{su}/\theta_c' = 5.3$ is considerably larger than $d_{m,armor}/d_{m,substrat} = 1.3$. Therefore suction removal does not take place and was not considered for bedload estimations herein.

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Value</th>
<th>Armor layer?</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gessler (1965)</td>
<td>$d_{84}/d_{50} &gt; 2.0$</td>
<td>2.4</td>
</tr>
<tr>
<td>Little and Mayer (1976)</td>
<td>$(d_{84}/d_{16})^{1/2} &gt; 1.3$</td>
<td>4.5</td>
</tr>
<tr>
<td>Chin (1985)</td>
<td>$d_{84}/d_{50} &gt; 1.5 &amp; d_{max}/d_{50} &gt; 1.8$</td>
<td>2.4 &amp; 7.1</td>
</tr>
<tr>
<td>Schöberl (1992)</td>
<td>$(d_{84}/d_{16})^{1/2} &gt; 1.35, d_{90}/d_{50} &gt; 1.55 &amp; d_{m}/d_{50} &gt; 1.05$</td>
<td>4.5, 3.1 &amp; 1.4</td>
</tr>
<tr>
<td>Günter (1971)</td>
<td>$(d_{84}/d_{16})^{1/2} &gt; 1.4$ to 1.6</td>
<td>4.5</td>
</tr>
</tbody>
</table>

8.2.2 Sediment transport in the Rein da Sumvitg River

Bedload transport is assumed to initiate when approach flow discharge in the river exceeds the threshold for armor layer scour, i.e. $Q_0 \geq Q_c = 33$ m$^3$/s. This result is in agreement with a former investigation reporting 30.1 m$^3$/s for the initiation of bedload transport (Jacobs et al. 2001).

The bedload transport capacity in the Rein da Sumvitg was determined by applying the bedload transport equations (4.3) to (4.6) presented in Section 4.1.3 to the estimated hydrograph at Runcahez. Jacobs et al. (2001) reported that the effective bedload transport in the Rein da Sumvitg at Runcahez amounts to 45% of the sediment transport capacity, i.e. $Q_{s}/Q_{s*} = 0.45$. This ratio was applied herein to determine the bedload transport rate in the river. Table 8.4 lists
the mean annual bedload mass for the years 1996 - 1999 of both the study of Jacobs et al. (2001) serving as a reference and the present study. The (SJ) equation resulted in the best agreement with the reference value with a 2% deviation. Therefore, (SJ) was applied to the hydrograph to determine the bedload transport in the Rein da Sumvitg between 1996 and 2014.

Table 8.4  Average annual bedload mass between 1996 and 1999 estimated based on different equations and the result of the previous study, serving as a reference

<table>
<thead>
<tr>
<th>Reference / Equation</th>
<th>Bedload mass ([10^3 \text{to}])</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reference study Jacobs et al. (2001)</td>
<td>9.9</td>
</tr>
<tr>
<td>(R)</td>
<td>14.3</td>
</tr>
<tr>
<td>(SJ)</td>
<td>10.1</td>
</tr>
<tr>
<td>(P)</td>
<td>23.6</td>
</tr>
<tr>
<td>(C)</td>
<td>1.3</td>
</tr>
</tbody>
</table>

The average annual bedload transport mass and volume in the Rein da Sumvitg between 1996 and 2014 is estimated to be \(BL = 10.6 \cdot 10^3 \text{to/yr}\) and \(BL_v = 4.0 \cdot 10^3 \text{m}^3/\text{yr}\), respectively. This is in a realistic range since it is close to the 4 years averaged value for 1996 to 1999 reported by Jacobs et al. (2001).

The suspended sediment load in the river was determined by applying a typical \(BL/SSL\)-ratio to the estimated bedload mass (Method I described in Chapter 4.1.6). Since the Pfaffensprung boundary conditions (topography, hydrology, geology) are comparable to those at Runcahez, a similar ratio of \(BL/SSL = 1:1\) was assumed (cf. Chapter 7.2.4). The resulting total sediment load in the river is thus twice the estimated bedload transport, i.e. \(TL = SSL + BL = 2 \times BL\). As a result, the mean total annual sediment transport mass and volume in the Rein da Sumvitg are estimated to be \(TL = 21.2 \cdot 10^3 \text{to/yr}\) and \(TL_v = 8.0 \cdot 10^3 \text{m}^3/\text{yr}\), respectively.

### 8.3 Sediment transport through the Runcahez SBT

Bedload transport in the SBT occurs when the SBT intake gate is open and the approach flow discharge exceeds the threshold for initiation of bedload transport (cf. Chapter 8.1 and 8.2.1). The approach flow discharge never exceeded the SBT discharge capacity during the observation period, so that the entire discharge including the sediments was assumed to be diverted through the SBT. However, bedload transport also takes place at approach flow discharges below the SBT operation threshold causing sediment depositions in front of the SBT intake. These sediments were re-entrained and transported through the SBT during the consecutive SBT
operation, so that the annual bedload mass supplied by the river equals that in the SBT. Figure 8.3 shows the annual bedload transport volumes and masses in the Runcahez SBT between 1996 and 2014. The mean annual bypassed bedload mass and volume are estimated to be $BL = 10.6 \times 10^3$ to/yr and $BL_v = 4.0 \times 10^3$ m$^3$/yr. For the years 1996 to 1999 the estimates correlate well with the reference data provided by Jacobs et al. (2001). As a further plausibility check, the mean estimated bedload volume was compared with the result of field surveys including the river bed morphology and excavation volumes of a gravel plant at the river mouth. Gravel extraction data from the gravel plant in Rabius (located 5 km downstream of the SBT, before the confluence with the Vorderrhein River, catchment area of $A = 85$ km$^2$) reveal an average annual extraction volume of $25.0 \times 10^3$ m$^3$/yr between 1960 and 2000 (TBA-GR 2015). During this period, the alluvial fan continuously decreased because the excavation volume exceeded the bedload supply estimated to $BL_v < 10.0 \times 10^3$ m$^3$/yr (Zarn 2015). The corresponding specific volumetric sediment supply rate amounts to $BL_v/A < 118$ m$^3$/km$^2$ and resulted in $BL_v < 6.6 \times 10^3$ m$^3$/yr for the Runcahez Reservoir ($A = 56$ km$^2$). This value is higher than the estimate but similar in magnitude and confirms the plausibility of the results. Therefore, the obtained bedload estimates were used for the hydroabrasion analysis as presented in Chapter 8.5 and 10.2.

![Figure 8.3](image-url)  
**Figure 8.3** Estimated bypassed bedload volumes and masses as well as reference data of Jacobs et al. (2001)

The rolling and suspension probability of the mean particle size in the SBT was $P_R = 0.33$ (Equation (3.21)) and $P_{su} = 0.00$ (Equation (3.23)), respectively. This indicates that two thirds of the particles were transported in saltation. The mean particle hop length $L_p = 2.28$ m was obtained by applying the parameters listed in Table 8.2 to Equation (3.33) and used to interpret the abrasion patterns in Chapter 8.5.
8.4 Bypass and trap efficiency of the Runcahez Reservoir

Sediment monitoring does not exist and no information about the sediment accumulation in the Runcahez Reservoir is available. Therefore, the bypass efficiency $BE$ of the reservoir was determined based on the estimated sediment masses transported in the river and the SBT (Table 8.5). The estimated annual bed load and suspended sediment mass supplied by the river amount to $BL = 10.6 \times 10^3$ to/yr and $SSL = 10.6 \times 10^3$ to/yr, respectively (Table 8.5). The trap efficiency of the suspended sediment $TESSL$ was determined based on Brune’s (1953) criteria using Equation (2.2) and resulted in $TESSL = 0.33$. Applying this value to the estimated suspended sediment transport in the river leads to an annual $SSL$ deposition volume of $3.5 \times 10^3$ to/yr (Table 8.5). The supplied bedload can be assumed to be completely accumulated in the reservoir without an SBT, resulting in a bypass efficiency of approx. $BE = 0.33$.

For the sake of simplicity the bypass efficiencies of the suspend sediment with and without the SBT were assumed to be equal, which is rather conservative as $SSL$ are bypassed to a great extent during SBT operation, so that the $SSL$ potentially depositing in the reservoir is decreased. The bedload transport is assumed to be bypassed by 100%, since the bedload particles are hindered to enter the reservoir because of the high guiding structure. As a result, the bypass efficiency with the SBT results in $BE = 0.83$. This value is within the typical range for reservoirs with a SBT in operation with $BE = 0.60 - 0.90$ and therefore assumed to be realistic (cf. Chapter 2.4.3). Based on these estimations and by assuming a bulk density of 1.8 to/m$^3$, the SBT reduced the annual accumulation mass and accordingly prolonged the reservoir life time by 4 times from 56 to 226 years.

Table 8.5 Mean annual sediment inflow and outflow of the Runcahez Reservoir and corresponding bypass efficiencies between 1996 and 2014 with and without SBT

<table>
<thead>
<tr>
<th></th>
<th>Without SBT</th>
<th></th>
<th></th>
<th>With SBT</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$SSL$</td>
<td>10.6</td>
<td>7.1</td>
<td>3.5</td>
<td>0.67</td>
<td>7.1</td>
<td>3.5</td>
<td>0.67</td>
</tr>
<tr>
<td>$BL$</td>
<td>10.6</td>
<td>0</td>
<td>10.6</td>
<td>0</td>
<td>10.6</td>
<td>0</td>
<td>1.0</td>
</tr>
<tr>
<td>$TL$</td>
<td>21.2</td>
<td>7.1</td>
<td>14.1</td>
<td>0.33</td>
<td>17.7</td>
<td>3.5</td>
<td>0.83</td>
</tr>
</tbody>
</table>
8.5 Hydroabrasion

The abrasion depths on the test fields were determined based on geodetic leveling measurements conducted in the 1990s and in 2014. The first topic in this subchapter is the observed abrasion, which is discussed by means of a representative test field. This is followed by a cross-comparison of the abrasion depths of the different test fields and their abrasion resistances.

Irregularities such as working joints and poorly compacted or improperly cured concrete are expected to represent spots of weakness and thus of initial damage. However, despite visible working joints, the abrasion patterns even after 19 years of operation were not affected by them, indicating proper installation and curing. The roller compacted concrete (RCC) represents an exception. Immediately after commissioning the RCC suffered a remarkable material loss near the walls (Appendix, Figure G.2), attributed to an improper compaction. As a result, the RCC test field was replaced after 4 years by a different concrete of unknown properties, so that only 4 years data are available from the RCC test field. Except this, the test fields generally behaved similar.

The abrasion map of the silica fume concrete (SC) test field as a representative after 4 and 19 operational years is shown in Figure 8.4a and b, respectively. The abrasion pattern is dominated by two incision channels along the tunnel walls. Figure 8.5a shows the longitudinally averaged cross-sectional profiles of the SC test field. The incision channels grow in depth and width over time, and the deepest point of the incision channels shifts towards the invert center. Similar abrasion channels as shown in Figure 8.5b were observed by Auel (2014) in physical scale model tests using a SBT model, despite the 50 times higher abrasion rate due to deviating test conditions (hydraulic conditions, sediment properties, invert material strength). Auel (2014) stated that the lateral abrasion pattern correlates with the bed shear stress distribution across the tunnel width. In narrow open channel flow, i.e. for $b/h < 4 - 5$ typically existing in SBTs, strong secondary currents of Prandtl’s second type cause high bed shear stresses close to the side walls (Chapter 3.1.3), resulting in high abrasion rates and incision channels. In return, these channels stabilize the secondary currents and hence cause a self-intensifying abrasion process. The mean aspect ratio in the Runcahez SBT was $b/h = 1.9$ and hence the observed abrasion patterns in the SBT are strongly related to secondary currents of Prandtl’s second type and confirm Auel’s (2014) laboratory results.

Although less pronounced, Figure 8.5a revealed an asymmetric cross sectional abrasion profile confirming visual surveys. The abrasion depths on the orographic left side are higher compared
to the right side. This is attributed to the downstream propagating effect of *Prandtl’s first type of secondary currents* due to the upstream located tunnel bend causing a bedload transport concentration. Although the bedload concentration gradually re-distributes across the tunnel width, its effect is still visible downstream of a bend (Chapter 3.1.3). For instance, the abrasion measurements in the Pfaffensprung revealed a significant effect of the bend even 180 m downstream (Chapter 7.5.1).

Figure 8.4 shows also a longitudinal undulating abrasion pattern with an amplitude an order of magnitude smaller than the incision channels. According to Auel (2014) such patterns are triggered by saltation of bedload particles. Bed irregularities cause sediment particles to saltate and initiate abrasion further downstream due to particle impingement, which in return act as abrasion triggers as well. As a result, a repetitive undulating abrasion pattern develops. The mean particle hop length $L_p = 2.28$ m obtained from Equation (3.33) is in the same range as the wave length $l_w = 2.5 \pm 0.5$ m, indicating a certain dependency between particle hop length and abrasion wave length. In contrast, the abrasion wave length at the Pfaffensprung SBT scales with half of the particle hop length. This might result from the superposition of different abrasion patterns caused by both a large range of hydraulic and sediment transport conditions, and surface irregularities. However, further systematic investigations including particle motion and invert abrasion are needed to clarify the effect of particle motion and invert irregularities on the abrasion pattern.

Figure 8.4 Abrasion depths of the SC test field a) in 1999 after four years of operation and b) in 2014 after 19 years of operation; circles represent measurement points, the raster of which is $0.4 \, \text{m} \times 0.5 \, \text{m}$
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Figure 8.5 Evolution of longitudinally averaged cross-sectional abrasion profiles at a) the SC test field in the Runcahez SBT with $F = 1.7$ and $b/h = 1.9$ and b) a flume invert tested in a physical model by Auel (2014) at $F = 1.8$ and $b/h = 2.8$, view in flow direction.

Figure 8.6a and b show the spatially averaged $a_m$, and the maximum abrasion depths $a_{max}$ ($= 95\%$ percentile) as function of bedload transport mass in the SBT. In the first four years, the abrasion depths were in the range of the measurement error. As a result, reliable quantification of those abrasion depths is difficult. This problem was overcome after 19 operational years when the abrasion depths significantly exceeded the measurement error. The abrasion depths increased with increasing bedload transport mass, and the mean annual abrasion rates were 0.9 - 1.4 mm/yr. The ratio between the maximum to mean abrasion depths was $a_{max}/a_m = 1.6 - 2.2$ and is in line with the ratios for concrete and granite $a_{max}/a_m = 1.8$ and $a_{max}/a_m = 2.1$, respectively, found in the Pfaffensprung SBT.

Figure 8.6 a) Mean and b) maximum abrasion depths as a function of cumulative bedload mass

Figure 8.7a and b show the mean abrasion rates of the tested concretes as a function of compressive and splitting tensile strength, respectively. The data largely scatter due to (I) the
Results of the test case Runcahez SBT

small data set, (II) the short testing period of the RCC, which had to be replaced after 4 years in operation, and (III) the different abrasion behavior of the polymer concrete (PC) because of the polymer instead of cementitious matrix. Despite the high splitting tensile strength, the Young’s modulus of the PC is about 3 times lower compared to the other materials (Table 4.14) associated with a higher elasticity and ductility. As a result, the interaction between impinging particles and invert material deviates from the other materials, which are dominated by a brittle behavior, leading to higher abrasion rates. Moreover, compressive or tensile strength alone might not be suitable to scale the abrasion rates, because other material properties also influence the abrasion resistance of the invert materials (Beyeler and Sklar 2010, Momber 2014, Scheingross et al. 2014, Lamb 2015). Nonetheless, the abrasion rates generally tend to decrease with increasing compressive and splitting tensile strengths.

Figure 8.7  Mean abrasion rates as a function of a) compressive strength and b) splitting tensile strength
9 Results of the underwater bedload tests

The results of the underwater bedload tests are presented in this chapter. Several cementitious invert materials installed in the Solis and Pfaffensprung SBTs were sampled to investigate their resistance against hydroabrasive stress by means of the Dresdner drum.

In the following, first the measured abrasion depths are presented and compared to determine the relative abrasion resistance of the tested materials. Second, the laboratory results are compared with the results from corresponding *in-situ* abrasion measurements to evaluate the scalability from laboratory to prototype scale.

9.1 Laboratory abrasion data

The following cementitious materials installed in the Pfaffensprung and Solis SBTs were tested in the Dresdner Drum: the high-strength concrete of the Pfaffensprung SBT (C2), as well as the High Alumina Cement concrete (HAC), the Low Shrinkage Concrete (LSC), the High-Strength Concrete (HSC), the Normal Concrete (NC) and the Ultra-High Performance reinforced Concrete (UHPC) installed in the Solis SBT. Figure 9.1 shows a sample of the UHPC and the LSC after the abrasion test. The borders between the drill core samples and the dummy plates are visible due to either the clipped steel fibers or the different colored aggregates. However, identical abrasion behavior at both the drill cores and the surrounding material was observed. This indicates a proper sample preparation, so that embedding of the drill core samples did not bias the test results.

Visual inspections of the abraded samples revealed different material-dependent abrasion characteristics resulting in different surface textures as seen in Figure 9.1. The cement matrix of the UHPC was subjected to higher abrasion rates compared to the steel fibers. Therefore, the surface of the abraded UHPC sample is studded with steel fibers (Figure 9.1a). In contrast, the surface of the LSC shows smoothly abraded aggregates and cement matrix because both the cement matrix and the aggregates suffered similar abrasion rates (Figure 9.1b).
Results of the underwater bedload tests

Figure 9.1 Samples of a) the UHPC and b) the LSC of the Solis SBT after testing, dotted lines indicate the boundary between drill core and dummy plate; courtesy of C. Bellmann (2016)

The abrasion measurements revealed that not only the abrasion patterns, but also the abrasion depths and rates vary among the materials. Figure 9.2 shows the spatially averaged abrasion depths (called “mean abrasion depths”) of the tested materials as a function of the exposure duration. It is noted that the exposure duration of the UHPC and the NC deviate from the norm of $t = 9.2 \text{ h}$. The exposure duration of the UHPC was $t = 9.54 \text{ h} (= 60’000 \text{ rotations})$ due to a lower rotation velocity of the drum between $t = 5.9 \text{ h}$ and $t = 7.9 \text{ h}$. The exposure duration of the NC was prolonged by mistake to $t = 9.76 \text{ h}$ instead of $t = 9.2 \text{ h}$. Therefore, the mean abrasion depth value of the NC listed in Table 9.1 was computed from a linear interpolation at $t = 9.2 \text{ h}$, which corresponds to the norm rotation number of 60’000. Moreover, the outstanding high abrasion depth of HAC at the end of the test is questionable and might result either from measurement errors or from test conditions that deviated from the norm due to an error in the test procedure, e.g. deviating rotation velocity of the drum. To reduce the biasing effect of such potential errors, the abrasion rates used for the analysis were determined by applying a linear fit to the measurement data as illustrated in Figure 9.2a. The inclination of the fits corresponds to the abrasion rates, which are listed in Table 9.1. The abrasion rates vary between $A_r = 2.16 \cdot 10^{-7} \text{ m/s}$ and $A_r = 4.36 \cdot 10^{-7} \text{ m/s}$. The highest value was obtained for the HAC followed by the LSC, the HSC, the NC, the C2 and the UHPC, in descending order. This means that the HAC exhibited the lowest resistance against the hydroabrasive stress regime in the Dresden Drum, followed be the LSC, the HSC, the NC, the C2 and the UHPC, in ascending order. For the sake of comparability with the other materials, the listed values of the C2 account for the abrasion of the sample center, i.e. $0.13 \times 0.13 \text{ m}$ (according to the position and size of the drill core samples), whereas the values in the brackets represent the data for the whole sample, i.e. $0.3 \times 0.3 \text{ m}$. 
As seen in Figure 9.2a, the mean abrasion depths increased with increasing exposure time for all materials. This is in agreement with theory, stating a linear correlation between abrasion depth and stress duration. However, some non-linear behaviors are also visible in Figure 9.2a. The abrasion rate of the C2 was highest at the beginning of the test, decreased afterwards and achieved a quasi-constant value, i.e. a constant slope, beyond $a_m = 3$ mm. The abrasion rates of the HAC, the LSC and the HSC did not achieve a quasi-constant value but revealed an increasing trend. This tendency might be caused by measurement errors or indicate variations of the material composition and quality in depths due to curing. Aggregates are absent at the surface, so that the abrasion behavior is dominated by the material properties of the cement matrix. The effect of the aggregates on the abrasion behavior increases as soon as the abrasion depths reach the aggregates and both the properties of the cement matrix as well as of the aggregates influence the abrasion behavior of the concrete. The curing can increase the material quality and hence the abrasion resistance, while its effect decreases with increasing distance to the surface. Furthermore, the increasing degree of structural weakening due to micro cracks could also lead to a decreasing abrasion resistance. To clarify the effects of all these aspects on the abrasion rates, extended test durations with larger abrasion depths are required.

Figure 9.2b shows the abrasion rate as a function of the splitting tensile strength. Despite significant differences in the properties of the tested materials, the abrasion rates deviated less than ±33% from the mean value. By applying the least mean square fitting to the log transformed data, $A_r \sim f_{st}^{-0.56}$ with $R^2 = 0.69$ was obtained. The exponent of the power law fit is approx. −0.5 instead of −2, which is expected according to the saltation abrasion models of Sklar and Dietrich (2004) and of Auel et al. (2017b) reported in Chapter 3.5.1 and 3.5.2, respectively. Applying a fit with −2 in the exponent results in a negative correlation coefficient and the data deviate ±200% from the fit. This indicates that the abrasion mechanism occurring in the Dresdner Drum deviates from that considered in the abrasion models due to different hydraulic and sediment transport conditions in the drum. Although the particle trajectories in the drum are not quantified in this study, the particle impact angle can be assumed to be considerably higher compared to the typical application range of the abrasion models (e.g. impact angles in SBTs are generally below 8° according to Auel (2014)). Increasing impact angle results in a significant increase of the concrete abrasion as seen in Chapter 3.3.3, Figure 3.7b. For instance, the abrasion rate can be doubled by increasing the particle impact angle from 15° to 30°. This increase of the abrasion rate is in the same range as the above revealed deviations and explains the different abrasion mechanisms occurring in the different setups.
Results of the underwater bedload tests

Therefore, the results obtained from the drum tests are not used to calibrate the abrasion prediction models.

![Figure 9.2](image)

**Figure 9.2** a) Mean abrasion depth as a function of exposure time and b) mean abrasion rate as a function of splitting tensile strength for all tested materials

The standard deviation $\sigma$ of the abrasion depth is a suitable parameter to describe the surface roughness. Figure 9.3 shows that in general $\sigma$ increases with increasing exposure time, for all materials resulting in an increasing bed non-uniformity, i.e. surface roughness. In particular, the rate of the increase for $\sigma$ decreases with increasing exposure duration. Overall, $\sigma$ is less than 25% of the mean abrasion depth (Table 9.1). The highest standard deviation of $\sigma = 5.0$ mm, corresponding to 48% of the $a_m$ was only obtained for the C2 considering the entire sample. This result is inherent to the Dresdner Drum. The hydroabrasive stress varies along the sample surface and causes the formation of a lateral ridge along the sample mounting, whereas the abrasion is more uniform at the center part of the sample (Mechtcherine et al. 2012). As a result, the standard deviation of the abrasion depths with $\sigma = 5.0$ mm as well as the ratio between the maximum (= 95%-percentile) and the mean abrasion depth of C2 with $a_{\text{max}}/a_m = 2.0$ exceeded $\sigma = 1.5 - 2.1$ mm and $a_{\text{max}}/a_m = 1.3 - 1.7$ obtained for the drill core samples, i.e. the NC, HSC, LSC, HAC and the UHPC (Table 9.1). Considering only the center part of the C2 samples, according to the position and size of the drill core samples, the standard deviation with $\sigma = 2.2$ mm and the $a_{\text{max}}/a_m$-ratio with $a_{\text{max}}/a_m = 1.7$ are in line with the values obtained from the drill core samples of the other materials.
Table 9.1    Abrasion data obtained from the Dresdner drum test

<table>
<thead>
<tr>
<th>Material</th>
<th>$a_m$ [mm]</th>
<th>$\sigma$ [mm]</th>
<th>$A_r$ $[10^{-7} \text{ m/s}]$</th>
<th>$a_{max}$ [mm]</th>
<th>$a_{max}/a_m$</th>
</tr>
</thead>
<tbody>
<tr>
<td>C2 (whole plate)</td>
<td>8.2 (10.4)</td>
<td>2.2 (5.0)</td>
<td>2.56 (3.27)</td>
<td>13.7 (21.0)</td>
<td>1.7 (2.0)</td>
</tr>
<tr>
<td>NC</td>
<td>9.7*</td>
<td>1.5</td>
<td>2.93*</td>
<td>12.7*</td>
<td>1.3</td>
</tr>
<tr>
<td>HSC</td>
<td>11.3</td>
<td>2.0</td>
<td>3.25</td>
<td>14.6</td>
<td>1.3</td>
</tr>
<tr>
<td>LSC</td>
<td>12.5</td>
<td>1.9</td>
<td>3.60</td>
<td>15.8</td>
<td>1.7</td>
</tr>
<tr>
<td>HAC</td>
<td>16.2</td>
<td>2.1</td>
<td>4.36</td>
<td>20.6</td>
<td>1.3</td>
</tr>
<tr>
<td>UHPC</td>
<td>7.8</td>
<td>1.8</td>
<td>2.16</td>
<td>11.8</td>
<td>1.5</td>
</tr>
<tr>
<td>Average</td>
<td>10.9</td>
<td>1.8</td>
<td>3.12</td>
<td>14.3</td>
<td>1.4</td>
</tr>
</tbody>
</table>

* Interpolated to the standard exposure duration of $t = 9.2$ h.

Figure 9.3    Standard deviation of the abrasion depth as a function of exposure time for all tested materials

The findings presented above indicate that the Dresdner Drum test is capable to determine the relative abrasion resistance of different invert materials. However, the results are not in agreement with the saltation abrasion models due to hydraulic and sediment transport conditions in the Dresdner Drum deviating from those considered by the models. Therefore, systematic laboratory studies are needed to determine the effect of hydraulic condition, particle motion and material properties on the abrasion rate occurring in the Dresdner Drum. Moreover, the scalability from laboratory to field scale is not clarified yet. To this end, the laboratory results were compared with the field measurements in Chapter 9.2.
9.2 Scalability of Dresdner Drum test results to prototype scale

The mean abrasion depths of the C2 test field in the Pfaffensprung SBT was 10.4 mm after 2 operational years, i.e. 116 days of operation with bedload transport (Chapter 7.1 and 7.5.1). The corresponding prototype abrasion rate was \( A_r = 1.04 \cdot 10^{-9} \) m/s. The mean abrasion rate of C2 in the Dresdner Drum test was \( A_r = 3.27 \cdot 10^{-7} \) m/s and hence was 300 times higher. Regarding the determination of abrasion resistance and life time estimations of different invert materials by means of laboratory tests, this is a promising result since it indicates that the abrasions of the C2 could be reproduced in the laboratory in a 300-fold accelerated time lapse. However, it must be noted that the scaling from laboratory to prototype scale strongly depends on the specific in-situ hydraulic and sediment transport conditions, the laboratory test conditions, and the invert material properties. Therefore, the scaling factor presented above holds true only for the specific combination of laboratory and prototype conditions. Application to other materials and SBTs is questionable due to deviating conditions and material properties.

The abrasion depths in the Solis SBT could not be reliably quantified due to small abrasion depths in the range of the measurement error of the laser scanner. Therefore, the in-situ abrasion depths were qualitatively classified by observation using the following scale: 0: no abrasion marks, 1: single abrasion marks, 2: <50% of the surface is abraded, 3: 50 - 90% of the surface is abraded, 4: 90 - 100% of the surface is abraded, 5: significant abrasions on the whole surface. Figure 9.4 shows the in-situ abrasion degree as a function of the mean abrasion depths obtained from the laboratory tests. The data considerably scatter and a reliable interpretation is hardly possible due to both too few data points and the non-objective determination of abrasion in the prototype. Investigation of the scalability from the laboratory to the field scale based on the abrasion depths in the Solis SBT is hence not possible and requires an extended observation period with significant in-situ abrasion depths.
Figure 9.4  *In-situ* invert abrasion degree observed in the Solis SBT as a function of laboratory abrasion depths obtained from the Dresdner Drum test

To conclude, the scalability of the abrasion rates from laboratory to prototype scale using the Dresdner Drum is questionable, because of a lack of reliable data and particularly the hydraulic and sediment transport conditions in the Dresdner Drum deviating from the prototypes. Systematic studies including Dresdner Drum tests and extended prototype experiments with abrasion depths reaching the core material are needed to investigate the scalability of the laboratory results to prototype scale.
10 Integrated analysis

An integrated analysis of the laboratory and prototype experiments is presented in this chapter. First the bypass efficiencies of the investigated SBTs are discussed with regard to their design and operating regime. Then, the calibration of the abrasion models based on the prototype results is presented and compared with literature data. Finally, the relative cost-effectiveness of the tested invert materials is treated.

10.1 Bypass efficiencies of SBTs

SBTs have demonstrated to be an effective countermeasure against reservoir sedimentation. The bypass efficiency of a reservoir equipped with a SBT (defined by the ratio of annually bypassed to annually inflowing sediment volume) depends on various parameters. These are: hydrology, sediment transport characteristics, and design and operating regime of the reservoir and the SBT. The latter is related to the type of SBT. The intake of type (A) SBTs is located at the reservoir head (Chapter 2.4). These SBTs are generally operated when the critical discharge for bedload transport is exceeded. As a result, the incoming sediments are directly conveyed past the dam without or only little deposition. Deposition of bedload may be hence completely prevented, whereas the deposition of the fine sediments depends on the operating regime and the design of the SBT. The higher the SBT design discharge capacity and thus the flood recurrence interval to be bypassed, the higher the share of the incoming suspended sediment load to be conveyed through the SBT and the smaller the amount of sediments entering the reservoir. The bypass efficiency of type (A) SBTs is generally $BE > 0.60$ (cf. Chapter 2.4.3), and the reservoir life time $RL$ is massively prolonged by the SBT as shown in Table 10.1. The Solis SBT, which is the only example of a type (B) SBT, has a significantly lower bypass efficiency of $BE = 0.31$ compared to type (A) SBTs due to (I) the intake location within the reservoir allowing for intermediate sediment deposition between the reservoir head and the SBT intake (cf. Chapter 6.6) and (II) a different operational regime. The Solis SBT is operated only during high flood discharge peaks resulting in an operation duration of a few hours per year, which is significantly lower than the typical operation durations of type (A) SBTs.
Table 10.1  Bypass efficiencies of existing SBTs with the (remaining) reservoir life time of the reservoir with and without a SBT

<table>
<thead>
<tr>
<th>SBT type</th>
<th>Name</th>
<th>BE [-]</th>
<th>Remaining RL without SBT [yr]</th>
<th>Remaining RL with SBT [yr]</th>
<th>RL prolongation [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>(A)</td>
<td>Pfaffensprung</td>
<td>0.98</td>
<td>&lt; 1</td>
<td>24</td>
<td>23</td>
</tr>
<tr>
<td>(A)</td>
<td>Runcahez</td>
<td>0.83</td>
<td>56</td>
<td>226</td>
<td>3.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SBT type</th>
<th>Name</th>
<th>BE [-]</th>
<th>Remaining RL without SBT [yr]</th>
<th>Remaining RL with SBT [yr]</th>
<th>RL prolongation [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>(A)</td>
<td>Asahi</td>
<td>0.77</td>
<td>190</td>
<td>636</td>
<td>2.3</td>
</tr>
<tr>
<td>(A)</td>
<td>Nunobiki</td>
<td>0.81 - 0.95*</td>
<td>52</td>
<td>1256</td>
<td>23</td>
</tr>
<tr>
<td>(B)</td>
<td>Solis</td>
<td>0.31</td>
<td>22</td>
<td>27</td>
<td>0.23</td>
</tr>
</tbody>
</table>

* including the effect of the Sabo dams; (Sumi et al. 2012, Auel et al. 2016c)

The bypass efficiency BE might not be a suitable indicator to determine the performance of a SBT, while considering the annual sediment mass balance of a reservoir, because it does not account for the operational regime of the SBT. Therefore, the operational bypass efficiency \( BE_{op} \) determined by the ratio of bypassed to inflowing sediment mass during the SBT operation is introduced. For type (A) SBTs, \( BE_{op} \) can reach a maximum of \( BE_{op} = 1.00 \) since the bypassed sediment mass is limited by the sediment supply. In contrast, type (B) SBTs, represented by the Solis SBT, can reach even higher values due to the re-entrainment of sediment deposits (cf. Chapter 6.6). Sediment transport in type (B) SBTs depends on the sediment supply and on the sediment transport processes within the reservoir as governed by the bed shear stress. The latter is controlled by the reservoir water level \( RWL \) and the approach flow discharge \( Q_0 \). As a result, a lower \( RWL \) at a given approach flow discharge results in higher bed shear stresses and hence higher sediment transport rates in the SBT. These findings can contribute to improved SBT operational regimes with regard to the reservoir life time, in particular for type (B) SBTs. Operational recommendations are provided in Chapter 11.6.

10.2 Abrasion model calibration

The focus of the present study is on the abrasion prediction of invert materials used in SBTs and hydraulic structures operated under high-speed flow conditions with bedload transport. The harming process of abrasion is dominated by the impingement of saltating bedload particles. The Saltation Abrasion Model (SAM) and the Saltation Abrasion Model adapted by Auel (SAMA) account for this mechanical process and are therefore discussed herein (Chapter 3.5.1 and 3.5.2). The analysis further includes the abrasion model introduced by Ishibashi (1983),
which was recently found to yield realistic abrasion estimates for SBTs (Auel et al. 2017b, Chapter 3.5.3).

All models include abrasion coefficients, i.e. the abrasion coefficient $k_v$ or material parameters $c_1$ and $c_2$, which have been partly but not clearly determined for various materials before. Therefore, these values were calibrated based on the prototype data from the Pfaffensprung and Runcahez SBTs (Chapter 7 and 8), the Mud Mountain SBT (Chapter 2.4.2 and 2.4.4), and the data provided by the operators of various Swiss SBTs (Chapter 2.4.2 and 2.4.4). However, the latter include a considerable level of uncertainty. Operator information often consist of non-objective estimations, whereas measurement data are scarce and poorly documented. As a result, these data are less reliable compared to the rest, so that only the data of the Val d’Ambra SBT was used herein. The abrasion depths on the invert of the Solis SBT were not measurable due to too low magnitude and hence were not used (Chapter 6.8). The model input parameters are listed in Appendix H, Table H.1, Table H.4 and Table H.7.

The saltation abrasion models (SAM and SAMA) and the abrasion model of Ishibashi are calibrated and compared to literature data in the following subchapter. This is followed by a discussion and hints on abrasion estimation in SBTs.

### 10.2.1 Saltation abrasion models (SAM and SAMA)

The abrasion rates of the different SBT invert materials are plotted as a function of splitting tensile strength in Figure 10.1. The data scatter considerably despite the small number of data points. The correlation coefficient of $R^2 = 0.27$ is low due to (I) the different materials, i.e. different concrete types, granite and steel and (II) different site-specific conditions, i.e. hydraulic conditions, sediment transport conditions, and sediment properties resulting in varying hydroabrasive stress regimes, and (III) model uncertainties. However, the data follow $A_r = 2.2 \cdot 10^{-7} f_{st}^{-2.0}$ with $R^2 = 0.27$. Neglecting steel, which has a different abrasion behavior due to its ductility, results in $A_r = 1.9 \cdot 10^{-7} f_{st}^{-2.0}$ with a correlation coefficient of $R^2 = 0.42$. However, the fits are comparable and are in agreement with the saltation abrasion models. Therefore, the SAM and the SAMA are assumed to be applicable for the abrasion prediction in SBTs.
The abrasion coefficient $k_v$ was determined for (I) the *Saltation Abrasion Model* (SAM) assuming a constant Young’s modulus $Y_M = 50 \text{ GPa}$, (II) the *Saltation Abrasion Model* using the effective Young’s moduli (SAM*) and (III) the *Saltation Abrasion Model adapted by Auel* (SAMA) also accounting for the effective Young’s moduli. The numerical $k_v$ values are presented in Appendix H, Table H.2, Table H.5 and Table H.8, and plotted in Figure 10.2 as a function of splitting tensile strength. The results for the SAMA are generally one order of magnitude smaller compared to those for the SAM and SAM* due to different formulas for the particle motion trajectories. Regardless, the data scatter was not significantly reduced as seen in Figure 10.2. The data scatter originates from model uncertainties, measurement errors as well as a certain material dependency, which is not accounted for in the models. The data scatter is considerably reduced by taking the latter into account. The values obtained from the Pfaffensprung SBT, the Runcahez SBT and the Val d’Ambra SBT are in a good agreement for concrete. Only the value for the polymer concrete (PC) is much lower. The polymer matrix increases the material ductility and changes the abrasion behavior. Therefore, this data point was not considered for the calibration of $k_v$.

The obtained abrasion coefficients fluctuate around $k_v = 1.3 \cdot 10^6$ for the SAM, $k_v = 8.8 \cdot 10^5$ for the SAM* and $k_v = 1.9 \cdot 10^5$ for the SAMA, respectively (Figure 10.2). These values are in good agreement with both $k_v = 10^6$ proposed by Sklar and Dietrich (2001) for the SAM and with $k_v = 1.9 \cdot 10^5$ proposed by Auel *et al.* (2017b) for the SAMA.

The $k_v$ values of granite, i.e. $k_v = 1.4 \cdot 10^7$ for the SAM, $k_v = 1.6 \cdot 10^7$ for the SAM* and $k_v = 2.4 \cdot 10^6$ for the SAMA, are roughly one order of magnitude higher compared to those of concrete.
Integrated analysis

(Figure 10.2). However, the former two are still in line with Sklar and Dietrich’s (2004/2012) laboratory results of roughly $k_v \approx 10^7$ for quartzite.

![Figure 10.2](image)

$\nu$ as a function of splitting tensile strength of various SBT invert materials for a) the SAM, b) the SAM* and c) the SAMA.

For all models the $k_v$ value determined for the steel lining at the Mud Mountain SBT is more than one order of magnitude lower than the values for concrete. This difference comes from the fact that steel is a ductile material, whereas the abrasion models were developed for brittle materials such as rock, concrete and mortar exhibiting a different abrasion behavior (cf. Chapter 3.3.3). Figure 10.3 shows the schematic stress-deformation curve from a direct tension test for brittle and ductile materials, respectively. The specific fracture energy is defined by the integral of the stress-deformation curve and assumed to correlate with the energy required to detach a unit volume. The fracture energy for brittle materials with a linear-elastic stress-strain behavior following Hook’s law is defined as:

$$E_{frac} = \frac{1}{2} \frac{f^2}{Y_M} \quad [J/m^2] \quad (10.1)$$

The plasticity of the ductile materials shown in Figure 10.3b significantly raises the fracture energy and presumably the abrasion resistance. This leads to a significantly underestimated abrasion resistance considering the elasto-plastic behavior of steel. Assuming an elastic behavior up to a yield strain of $\varepsilon = 0.001$ and a plastic behavior until the rupture strain of $\varepsilon = 0.2$, Equation (10.1) considering only the elastic behavior leads to an underestimation of the fracture energy by 400 times. This results in $k_v$-values for the SAM* and SAMA that are approx. 400 times too low. Moreover, using a Young’s modulus of $Y_M = 50$ GPa for the SAM with steel is
misleading because the Young’s modulus of steel is roughly 4 times higher. This means that the computed $k_v$-values for the SAM are about 1600 times lower. The results for steel and granite are in a comparable range (deviation less than ±60%) when these factors are recognized.

The abrasion coefficient was further computed for various materials based on abrasion data collected from literature. The data basis encompasses abrasion data from field and laboratory experiments. The $k_v$-values were determined using the SAMA, accounting for the supercritical flow conditions in the SBTs as well as in the prior cited studies, except from the abrasion mill experiments (cf. Chapter 3.4.1) conducted by Sklar and Dietrich (2004) and Scheingross et al. (2014). The hydraulic conditions and particle motions in these experiments considerably deviate from those in open channel flow. Therefore, the following input parameters were selected for the abrasion mill data sets: (I) the bedload active width was 0.075 m according to Sklar and Dietrich (2001), (II) the cover effect term was neglected for single particle experiments without particle interaction and (III) the sediment transport rate of the experiments using 150 g of sediments with $d = 6$ mm was exceeded by the sediment transport capacity by a factor of 7, so that $q_s/q_s^* = 1/7$ was used herein.

Figure 10.4 shows the abrasion coefficient $k_v$ as a function of splitting tensile strength $f_{st}$ for a range of invert materials. A general trend of increasing $k_v$ with increasing $f_{st}$, as revealed by Auel et al. (2017b), is visible despite a considerable data scatter. The foam experiments of Scheingross et al. (2014) confirmed this trend, but resulted in abrasion coefficients of roughly two orders of magnitudes below the mean trend. This is attributed to the low Young’s modulus of $Y_M = 3.9 - 330$ MPa and the low material density of $\rho_c = 87 - 960$ kg/m$^3$ of the tested foams, which reduce the abrasion resistance (Beyeler and Sklar 2010, Momber 2014, Scheingross et al. 2014, Lamb et al. 2015). The foam data are of minor interest regarding practical applications and are no longer considered herein.
The data in Figure 10.4 show a certain material-dependency of $k_v$. The highest $k_v$-values were obtained for rock, followed by concrete, mortar / soft rock and steel. Based on the abrasion data of the Pfaffensprung SBT $k_v = 2.4 \cdot 10^6$ was found for granite. This agrees with Sklar and Dietrich’s (2001, 2004) data for hard rocks such as granite, quartzite and marble. The values for concrete (denoted by circular symbols) obtained from literature data mostly agree with the prototype results and confirm $k_v = 0.19 \cdot 10^6$ for concrete, found by Auel et al. (2017b). The $k_v$ computed using the data set provided by Kryžanowski et al. (2012) are one order below the mean trend. This deviation is caused by the test location in the stilling basin of a weir, which tends to retain sediment in a recirculating flow field. As a result, the same sediments repeatedly impact the invert (Spörel et al. 2015). This special hydraulic condition causes significant invert material loss resulting in decreasing $k_v$-values. The value for steel is also considerably below the mean trend due to the ductile behavior of steel, which the SAMA does not account for.

Auel et al. (2017b) recently found a certain stabilization of the abrasion coefficient around $k_v \approx 10^5$ for hard materials with $f_{st} > 1$ MPa, which cannot be identified in Figure 10.4 due to the large scatter. This scatter is supposed to be reduced by accounting for the effect of further invert material properties (e.g. Young’s modulus, crystal and clast size, porosity and density), and sediment abrasiveness (i.e. sediment shape and lithology) as reported in Chapter 3.5.1. However, the effect of those parameters on the abrasion rate could not be investigated due to a lack of data. Porosity as well as crystal and clast size of the invert material are unknown. Information about particle shape and lithology of the sediment in the different catchment areas present too high uncertainties and variabilities in the same range. A further scaling parameter might be the characteristic length $l_{ch} = Y_M E_{frac} f_{st}^2$ introduced by Hillerborg et al. (1976). However, the effect of the characteristic length on the scaling could not be investigated due to a lack of data. The therefore required parameters need to be either assumed or derived from the measured material parameters and hence are again, as $f_{st}$, a function of the compressive strength. Consequently, the data scatter observed in Figure 10.4 could not be enhanced by using those parameters and hence is not treated herein.
To conclude, different $k_v$ values were found for concrete, granite and steel. By accounting for the effective Young’s modulus $Y_M$ and the particle motion in supercritical flow with the SAMA, the $k_v$ values were found to be roughly one order of magnitude smaller than those determined with the SAM and SAM*. However, the data scatter was not significantly reduced due to low variations in the Young’s modulus, which only slightly affects the model accuracy. The model accuracy is rather dominated by other uncertainties and errors, such as determination of the representative hydraulic and sediment transport conditions, bedload estimation, abrasion measurements, the abrasiveness of the sediment and the abrasion resistance of the invert material. More comprehensive field and laboratory data, including detailed information on the invert material and sediment properties, are needed to clarify the effect of the latter two on the abrasion rate and to enhance the model accuracy.

### 10.2.2 Ishibashi abrasion model

The material parameters $c_1$ and $c_2$ used with the Ishibashi (1983) abrasion model were obtained from SBT abrasion data. These parameters are listed in Appendix Table H.3, Table H.6 and Table H.9, and illustrated in Figure 10.5 as a function of splitting tensile strength $f_{st}$. The results obtained from the Runcahez SBT and the Val d’Ambra SBT are in a good agreement. The values obtained from the Pfaffensprung SBT are lower. The weighted average value for concrete amounts to $c_1 = 1.4 \cdot 10^{-7}$, which agrees with $c_1 = 1.189 \cdot 10^{-7}$ proposed by Ishibashi (1983). The value for granite was two orders of magnitudes lower with $c_1 = 7.2 \cdot 10^{-9}$, and the abrasion data of the Mud Mountain SBT resulted in $c_2 = 8.4 \cdot 10^{-10}$ for steel. The latter value is
one order of magnitude higher than proposed by Ishibashi (1983). One reason for this deviation is the model uncertainty, which causes a significant data scatter shown in Figure 10.5. Moreover, the application of Ishibashi’s (1983) laboratory results to prototype scale might be misleading due to potential model and scale effects. Apart from this discrepancy, the findings confirm the material-dependency which was previously cited by Ishibashi (1983). However, the material dependency might not be sufficiently accounted for by applying the provided constants for steel and concrete. Therefore, additional laboratory and field abrasion data from literature were used to determine $c_1$ and $c_2$ for various materials.

Figure 10.6 shows the resulting $c_1$ and $c_2$ values as a function of splitting tensile strength $f_{st}$ for a range of invert materials. The data scatter is larger compared to that of the SAMA shown in Figure 10.4. Therefore, the accuracy of the Ishibashi model is presumably lower compared to that of the SAMA. Despite this large scatter, there is a clear trend of decreasing $c_1$ with increasing $f_{st}$. The data obtained from the abrasion mill by Sklar and Dietrich (2001, 2004) and Scheingross et al. (2014) confirm this trend, but result in values two to three orders of magnitude lower compared to the SBT data. One reason for this deviation is found in the application of Ishibashi’s (1983) particle motion equations, which might not be suitable to describe the particle motion in an abrasion mill. Further reasons for this deviation and the large data scatter shown in Figure 10.6 are: model uncertainties, lack of the sediment abrasiveness, and the insufficient determination of the abrasive resistance of the invert material as previously discussed in Chapter 10.2.1. To increase the prediction accuracy more information on the sediment, i.e. hardness and angularity, and invert material properties, is required. This should be a subject of further studies.
10.2.3 Abrasion prediction for SBTs

The SAM, SAM*, SAMA and Ishibashi’s model can be used for abrasion estimation in SBTs. However, these models were developed for specific application ranges, which should be respected. The SAM and SAM* should be applied for subcritical to low supercritical flow regimes and the SAMA and Ishibashi’s formula should be applied for (highly-)supercritical flow regimes. Moreover, the SAMA should be preferred over the Ishibashi model in particular for brittle materials due to its presumably higher model accuracy (Chapter 10.2.3).
The models include abrasion coefficients, i.e. $k_v$, $c_1$ and $c_2$. These parameters were determined for various invert materials based on \textit{in-situ} investigation in SBTs. The mean, minimum and maximum values are summarized in Table 10.2 and Table 10.3 together with literature data. There is a considerable scatter, irrespective of the model, because of model uncertainties, measurement errors and the fact that various material parameters, which are known to have an effect on the abrasion, are not accounted for by the models. The mean values balance out some of these effects and therefore should be used for long-term abrasion estimation. However, they are still afflicted with some uncertainty. Table 10.4 presents the relative error $\sigma_{r,i}$ of the input parameters and the relative error of the abrasion coefficients resulting from the error propagation analysis assuming the variables to be independent (cf. Chapter C). The uncertainty of $k_v$ amounts to $\sigma_{r,kv} \approx 110\%$, whereas it is $\sigma_{r,c1} \approx \sigma_{r,c2} \approx 146\%$ for $c_1$ and $c_2$. The uncertainty of $c_1$ and $c_2$ is considerably higher compared to $\sigma_{r,kv}$ due to high uncertainties of the deformation and friction work $E_k$ and $W_f$, respectively.

The values listed in Table 10.2 and Table 10.3 for concrete and granite partly correlate with literature and can be assumed to yield realistic estimates. However, application of the material-specific values instead of the constant value proposed by Sklar and Dietrich (2004, 2012) considerably enhances the prediction accuracy and is therefore strongly recommended.

Although the saltation abrasion models were developed for abrasion prediction of brittle materials, they are applicable for steel as well provided the corresponding $k_v$ value in Table 10.2 are used. The abrasion coefficient $c_1$ for concrete is in line with Ishibashi (1983), whereas the obtained abrasion coefficient $c_2$ for steel deviates from literature. The results obtained from the prototype experiments presumably lead to more realistic results since they are not subjected to potential model and scale effects. Therefore, the values obtained from the prototype experiments are recommended.

It is noted that the abrasion coefficients depend on the abrasion resistance of the invert material and the abrasiveness of the sediment, which are inadequately or not accounted for by the models. Therefore, application of the values listed in Table 10.2 are useful in a first step, but might result in incorrect estimates. At existing facilities, the accuracy and reliability of the estimates can be significantly enhanced by using site-specific values obtained from \textit{in-situ} calibration experiments instead of using the values listed in Table 10.2.
Table 10.2  Abrasion coefficients $k_v$ for the SAM, the SAM* and the SAMA with values provided by literature (minimum and maximum values in brackets)

<table>
<thead>
<tr>
<th>Model</th>
<th>Rock</th>
<th>Concrete</th>
<th>Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>SAM</td>
<td>14 (7.8 - 32)</td>
<td>1.3 (0.31 - 4.9)</td>
<td>0.0063 (0.0061 - 0.018)</td>
</tr>
<tr>
<td>Sklar and Dietrich (2004, 2012)</td>
<td>1.0 (1.4 - 9.1)*</td>
<td>1.0 (1.4 - 9.1)*</td>
<td></td>
</tr>
<tr>
<td>SAM*</td>
<td>16 (9.2 - 38)</td>
<td>0.88 (0.10 - 3.8)</td>
<td>0.026 (0.0026 - 0.075)</td>
</tr>
<tr>
<td>SAMA</td>
<td>2.4 (1.5 - 5.8)</td>
<td>0.19 (0.08 - 0.51)</td>
<td>0.0081 (0.0079 - 0.023)</td>
</tr>
<tr>
<td>Auel et al. (2017b)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* For rock and mortars

Table 10.3  Material parameters $c_1$ and $c_2$ for Ishibashi’s (1983) abrasion prediction model with values provided by literature (minimum and maximum value in brackets)

<table>
<thead>
<tr>
<th>Rock</th>
<th>Concrete</th>
<th>Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>This study</td>
<td>0.072 (0.026 - 0.100)</td>
<td>1.38 (0.142 - 2.46)</td>
</tr>
<tr>
<td>Ishibashi (1983)</td>
<td>-</td>
<td>1.189</td>
</tr>
</tbody>
</table>

Table 10.4  Uncertainties of input parameters and resulting $k_v$, $c_1$ and $c_2$

<table>
<thead>
<tr>
<th>SAM</th>
<th>SAM</th>
<th>Ishibashi (1983)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameter</td>
<td>$\sigma_{r,i}$ [%]</td>
<td>Parameter</td>
</tr>
<tr>
<td>$A_r$</td>
<td>40</td>
<td>$A_r$</td>
</tr>
<tr>
<td>$Y_M$</td>
<td>13</td>
<td>$Y_M$</td>
</tr>
<tr>
<td>$f_{st}$</td>
<td>42</td>
<td>$f_{st}$</td>
</tr>
<tr>
<td>$q_s$ and $q_i$</td>
<td>50</td>
<td>$q_s$ and $q_i$</td>
</tr>
<tr>
<td>$\theta$ and $\theta_c$</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>$U_*$</td>
<td>6</td>
<td>$k_v$</td>
</tr>
<tr>
<td>$k_v$</td>
<td>107</td>
<td>$k_v$</td>
</tr>
</tbody>
</table>

The abrasion depths of the invert materials installed in the Solis SBT could not be quantified by means of a laser scanner. Therefore, the abrasion rates and depths between 2012 and 2016 were estimated by applying the SAMA and the Ishibashi abrasion model with $k_v$ and $c_1 / c_2$ values from Table 10.2 and Table 10.3, respectively. The hydraulic and sediment transport conditions in the SBT were quantified in Chapter 6.1 and 6.5.2, respectively. The resulting abrasion depths are listed in Table 10.5. The SAMA resulted in lower abrasion depths for the brittle materials, and higher abrasion depths for steel compared to Ishibashi (1983). However,
the estimated abrasion depths from the latter are in a comparable range and vary from \( a = 0.001 \text{ mm} \) for steel to \( a = 0.278 \text{ mm} \) for the high-strength concrete (HSC). This result confirms the qualitative field observations of small abrasion depths, which were too small to be quantified by means of the laser scanner (cf. Chapter 6.8 and last row in Table 10.5).

The visual inspection of the abrasion depths qualitatively agrees with the estimated abrasion depths. Only for the high alumina cement concrete (HAC) the observations do not match with the estimates. While the estimated abrasion depths of the HAC are comparable to the estimates for the NC, the HSC and the LSC, the HAC performed significantly better than these materials in the field. This might be attributed to the fact that the abrasion models only account for the Young’s modulus and tensile strength, although further parameters e.g. sediment shape and hardness or invert material properties, are known to affect the abrasion behavior. A reliable conclusion on the abrasion behavior and a robust evaluation of the abrasion models are difficult at the present stage, requiring an extended observation period with abrasion depths on the order of centimeters.

### Table 10.5 Estimated and observed abrasion depths in the Solis SBT between 2012 and 2016

<table>
<thead>
<tr>
<th></th>
<th>SAMA</th>
<th>Ishibashi (1983)</th>
<th>In-situ observation</th>
</tr>
</thead>
<tbody>
<tr>
<td>NC</td>
<td>0.163</td>
<td>0.269</td>
<td>Wavy abrasion pattern partially exposing the aggregates</td>
</tr>
<tr>
<td>HSC</td>
<td>0.128</td>
<td>0.278</td>
<td>Wavy abrasion pattern partially exposing the aggregates</td>
</tr>
<tr>
<td>LSC</td>
<td>0.174</td>
<td>0.276</td>
<td>Wavy abrasion pattern partially exposing the aggregates</td>
</tr>
<tr>
<td>HAC</td>
<td>0.155</td>
<td>0.275</td>
<td>Small abrasion marks at working joints</td>
</tr>
<tr>
<td>UHPC</td>
<td>0.060</td>
<td>0.250</td>
<td>Single notches and scratches</td>
</tr>
<tr>
<td>Cast basalt</td>
<td>0.002</td>
<td>0.010</td>
<td>Abrasion marks at the upstream edges of the tiles</td>
</tr>
<tr>
<td>Steel</td>
<td>0.027</td>
<td>0.001</td>
<td>Single impact dips</td>
</tr>
</tbody>
</table>

### 10.3 Cost-effectiveness of invert materials

The cost-effectiveness of various invert materials accounting for abrasion resistance and cost is presented in this subsection. To this end, the net present value \( NPV \) of a range of invert materials was computed based on estimated abrasion rates and is discussed in a first step. Second, the cost-effectiveness of the invert materials installed in the prototype SBTs was determined based on the actual abrasion rates observed \textit{in-situ}.
10.3.1 Modelling cost-effectiveness

Different invert materials exhibit different abrasion resistances. This results in different replacement intervals. To account for that, the net present value (NPV) was used to determine the costs of a range of invert materials. The NPV is defined as:

\[
NPV = -J + \sum_{t=1}^{T} \frac{C_t}{(1+r)^t} \quad [\text{CHF}]
\]  

(10.2)

Where \( J \) = initial investment, \( C_t \) = net cash flow at a point in time \( t \), \( r \) = interest rate and \( T \) = accounting period. Disposal costs were assumed to be equal irrespective of the material, so that the NPV was determined based on initial investment \( J \) (including planning, construction management, material, and implementation) and maintenance and potential refurbishment cost \( C_t \), resulting in negative NPVs as only costs were considered.

The following assumptions were made for this analysis: invert material thickness of 0.30 m except for steel with 0.025 m, material replacement at a mean abrasion depth of 2/3 of the initial invert thickness, interest rate of \( r = 0.03 \) and accounting period of \( T = 80 \) yr according to a typical concession period in Switzerland. The abrasion rates were computed by using the SAMA (Equation (3.59)) with the abrasion coefficients listed in Table 10.2 in Chapter 10.2.3. The hydroabrasive impact, strongly affecting the abrasion rates, may be tentatively quantified by introducing the hydroabrasion index \( HAI \):

\[
HAI = d_m \cdot BL \cdot P_{sa} \quad [\text{m-to/yr}]
\]  

(10.3)

It depends on the mean particle diameter \( d_m \), the annual bedload mass \( BL \) and the saltation probability \( P_{sa} \). The latter follows from the suspension probability \( P_{su} \) (Equation (3.21)) and the rolling probability \( P_R \) (Equation (3.23)) as \( P_{sa} = 1 - P_{su} - P_R \). It is noted that Equation (10.3) is tentative, and presumably has to be revised after more data on sediment abrasiveness, i.e. shape and hardness, become available.

As a first approach, three different scenarios were distinguished, i.e. low (\( HAI < 100 \) [m-to/yr]), moderate (100 \( \leq HAI \leq 2'000 \) [m-to/yr]) and severe abrasion conditions (\( HAI > 2'000 \) [m-to/yr]), for which the Solis, the Runcahez and the Pfaffensprung SBT, respectively, are examples. The corresponding \( HAi \)s as well as the operating conditions in these SBTs and the parameters required for the abrasion estimation are listed in Table 10.6.
Table 10.6 Abrasion scenarios

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Name</th>
<th>HAI [m \cdot to/yr]</th>
<th>SBT example</th>
<th>HAI [m \cdot to/yr]</th>
<th>(d_m) [m]</th>
<th>BL [to/yr]</th>
<th>(P_{sa}) [-]</th>
<th>Operation [d/yr]</th>
<th>(q_s) [to/(s \cdot m)]</th>
<th>(q_s/q_s^*) [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>&lt; 1000</td>
<td>Solis 2012-2016</td>
<td>3</td>
<td>0.006</td>
<td>740</td>
<td>0.58</td>
<td>0.88</td>
<td>2.2</td>
<td>0.005</td>
<td></td>
</tr>
<tr>
<td>Moderate</td>
<td>1000-10'000</td>
<td>Runcahez 1996-2014</td>
<td>1’630</td>
<td>0.23</td>
<td>10’600</td>
<td>0.67</td>
<td>1.2</td>
<td>21.5</td>
<td>0.12</td>
<td></td>
</tr>
<tr>
<td>Severe</td>
<td>&gt; 10'000</td>
<td>Pfaffensprung 2012-2015</td>
<td>61’250</td>
<td>0.25</td>
<td>350’000</td>
<td>0.70</td>
<td>75.75</td>
<td>12.2</td>
<td>0.04</td>
<td></td>
</tr>
</tbody>
</table>

The annual maintenance cost under severe abrasion conditions was assumed to be 25 CHF/m², corresponding to approx. 5% of the investment cost of the concretes installed in the Solis SBT. The net present values \(NPVs\) per square meter for a range of invert are shown in Figure 10.7 and listed in Table 10.7 assuming severe abrasion conditions, as existing at the Pfaffensprung SBT with a \(HAI = 61’250 \ m \cdot to/yr\). Variations of the interest rate \(r = 0.03 \pm 50\%\) resulted in \(NPV\) variations of less than 6% and hence indicate that the results are not sensitive to \(r\). Therefore, the \(NPVs\) obtained with \(r = 0.03\) were considered in the following.

The highest \(NPV\) and hence the highest cost-effectiveness was obtained for the granite (G1), followed by high-strength concrete (C1) both installed in the Pfaffensprung SBT. Slightly lower \(NPVs\) were computed for the high-strength concrete (HSC, Solis SBT), the low shrinkage concrete (LSC, Solis SBT) and the normal concrete (NC, Solis SBT), whereas significantly lower values resulted for the ultra-high performance concrete (UHPC, Solis SBT), the polymer concrete (PC, Runcahez SBT), the steel fiber concrete (SF, Runcahez SBT), the silica concrete (SC, Runcahez SBT), the high-performance concrete (HPC, Runcahez SBT), the high alumina cement concrete (HAC, Solis SBT) and the roller compacted concrete (RCC, Runcahez SBT). The lowest \(NPVs\) and hence the lowest cost-effectiveness was obtained for the steel (S235, Solis SBT). In general, the materials installed in the Runcahez SBT resulted in lower \(NPVs\) than those installed in the Solis SBT, due to the usage of basaltic aggregates. This type of aggregate raised the material costs, whereas their positive effect on the abrasion resistance caused by the high material hardness might be underestimated by the SAMA. As a result, the cost-effectiveness of these materials might be underestimated.

As seen from Figure 10.7 the relative cost-effectiveness varies with time. For instance, in the long-term, i.e. \(T > 70\ yr\), the granite G1 exhibits the highest \(NPV\), whereas in the short to medium-term the \(NPVs\) of other materials exceed that of G1 due to their relatively low
investment costs. This means that the target life time has to be considered for determining the most cost-effective material.

Over all, the findings presented herein indicate that in the long-term, granite exhibits the highest cost-effectiveness for severe abrasion conditions and hence confirm the use of granite as an invert material as installed in the Pfaffensprung SBT and the Mud Mountain SBT with a $HAI = 61'250$ and $HAI = 2'700$, respectively (Müller and Walker 2015, Auel et al. 2017d).

![Figure 10.7 Costs expressed by the net present value for a range of invert materials considering investment and maintenance costs assuming severe abrasion conditions with $HAI = 61'250$ m·to/yr](image)

![Table 10.7 Net present value for a range of invert materials assuming severe abrasion conditions for different interest rates](table)

<table>
<thead>
<tr>
<th>$\overline{NPV}$ [10^3 CHF/m²]</th>
<th>S235</th>
<th>G1</th>
<th>NC</th>
<th>HSC</th>
<th>LSC</th>
<th>HAC</th>
<th>UHPC</th>
<th>S235</th>
<th>SC</th>
<th>RCC</th>
<th>HPC</th>
<th>SF</th>
<th>PC</th>
</tr>
</thead>
<tbody>
<tr>
<td>$r = 0.015$</td>
<td>4.1</td>
<td>3.8</td>
<td>5.6</td>
<td>5.1</td>
<td>5.5</td>
<td>12.4</td>
<td>7.5</td>
<td>18.3</td>
<td>9.8</td>
<td>14.2</td>
<td>13.6</td>
<td>9.6</td>
<td>7.5</td>
</tr>
<tr>
<td>$r = 0.03$</td>
<td>4.4</td>
<td>3.9</td>
<td>6.0</td>
<td>5.5</td>
<td>5.8</td>
<td>13.1</td>
<td>8.0</td>
<td>19.4</td>
<td>10.3</td>
<td>15.1</td>
<td>14.5</td>
<td>10.2</td>
<td>7.9</td>
</tr>
<tr>
<td>$r = 0.045$</td>
<td>4.7</td>
<td>4.1</td>
<td>6.4</td>
<td>5.8</td>
<td>6.2</td>
<td>13.9</td>
<td>8.5</td>
<td>20.6</td>
<td>11.0</td>
<td>16.0</td>
<td>15.4</td>
<td>10.8</td>
<td>8.3</td>
</tr>
</tbody>
</table>

The $NPVs$ per square meter for a range of invert materials for moderate abrasion conditions, as existing in the Runcahez SBT, are shown in Figure 10.8 and listed in Table 10.8. The annual maintenance costs for moderate abrasion conditions were assumed to be one fifth of those for severe conditions, i.e. 5 CHF/m². Variations of the interest rate $r = 0.03 \pm 50\%$ resulted in $NPV$ variations of less than 4% and hence indicate that the results are even less sensitive to $r$ compared to the severe abrasion scenario.
The annual abrasion rates vary between 0.04 mm/yr for granite (G1) and 1.21 mm/yr for the RCC and hence are relatively low due to the relative low hydroabrasion impact compared to the severe abrasion conditions. As a result, the replacement interval is larger than 160 years, so that no replacement takes place within the considered period of $T = 80$ yr. The highest cost-effectiveness was obtained for the LSC followed by the C1, the NC and the HSC. Slightly lower NPVs resulted for the RCC, the SF and the SC, whereas considerably lower NPV were computed for the steel (S235), the HAC, the granite and the UHPC, and the PC. The costs expressed by the NPV of G1 was twice that of the C1, whereas it was 11% lower than that of the C1 for severe abrasion conditions after 80 years. This indicates that cost-effectiveness analyses need to account not only for the target life time but also for the on-site abrasion conditions.

Figure 10.8 Costs expressed by the net present value for a range of invert materials considering investment and maintenance costs assuming moderate abrasion conditions with $HAI = 1'630$ m∙to/yr

Table 10.8 Net present value for a range of invert materials assuming moderate abrasion conditions for interest rates

<table>
<thead>
<tr>
<th>$-^{NPV}$ [10^3 CHF/m²]</th>
<th>C1</th>
<th>G1</th>
<th>NC</th>
<th>HSC</th>
<th>LSC</th>
<th>HAC</th>
<th>UHPC</th>
<th>S235</th>
<th>SC</th>
<th>RCC</th>
<th>HPC</th>
<th>SF</th>
<th>PC</th>
</tr>
</thead>
<tbody>
<tr>
<td>$r = 0.015$</td>
<td>0.9</td>
<td>2.1</td>
<td>0.9</td>
<td>0.9</td>
<td>2.1</td>
<td>2.1</td>
<td>1.5</td>
<td>1.3</td>
<td>1.2</td>
<td>1.2</td>
<td>1.2</td>
<td>1.2</td>
<td>3.0</td>
</tr>
<tr>
<td>$r = 0.03$</td>
<td>1.0</td>
<td>2.1</td>
<td>1.0</td>
<td>1.0</td>
<td>0.9</td>
<td>2.1</td>
<td>2.2</td>
<td>1.6</td>
<td>1.3</td>
<td>1.2</td>
<td>1.3</td>
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<tr>
<td>$r = 0.045$</td>
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<td>2.2</td>
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<td>1.6</td>
<td>1.3</td>
<td>1.2</td>
<td>1.3</td>
<td>1.3</td>
<td>3.1</td>
</tr>
</tbody>
</table>

The obtained NPVs for severe and moderate abrasion conditions are illustrated as a function of splitting tensile strength $f_{st}$ in Figure 10.9. The data collapse well, with the exception of steel due to its high splitting tensile strength. However, completely different scenario-dependent magnitudes and trends are visible. The magnitudes for severe abrasion conditions are
considerably higher compared to those for moderate abrasion conditions due to relatively high abrasion rates requiring several material replacements within the accounting period. The \( NPV \) increases with increasing \( f_{st} \) for severe abrasion conditions, and the least square fitting applied to all data except for steel results in \( NPV = -2.5 \cdot 10^5 f_{st}^{-1.55} \) with \( R^2 = 0.48 \). In contrast, \( NPV = -2.1 \cdot 10^2 f_{st}^{0.89} \) with \( R^2 = 0.28 \) was obtained for moderate abrasion conditions. This result confirms that the use of materials with high tensile strength for long-term applications under severe abrasion conditions is beneficial, despite higher investment cost. However, for moderate to low abrasion conditions or short-term applications, less expensive materials exhibit the highest cost-effectiveness despite lower abrasion resistances.

The results presented above give an overview of the selection of invert materials based on their cost-effectiveness under different abrasion scenarios. In the following, the cost-effectiveness of different invert materials installed in the prototype SBTs were analyzed based on the actual abrasion rates determined in-situ. The results of the underwater bedload drum tests were not used for this analysis, due to the questionable upscaling from laboratory to prototype scale (cf. Chapter 9.2).

### 10.3.2 Cost-effectiveness of the Pfaffensprung invert materials

The \( NPV \) of the concrete C1 and the granite G1 installed in the Pfaffensprung SBT were computed based on the actual abrasion rates from 2012 to 2015 for an accounting period of \( T = 80 \) yr. The \( NPV \) was determined based on the investment cost and estimated annual maintenance cost of 25 CHF/m². The initial material thickness was 30 cm and the material replacement was assumed to take place as soon as cumulative abrasion depth reached 2/3 of the initial material thickness.
Figure 10.10 illustrates the $NPV$ of the C1 and G1 over time. The $NPV$ of the granite was below that of concrete due to its high investment costs over a 75 year period. The first replacement of the granite liner is expected after 150 years of operation due to its low abrasion rates, whereas the replacement interval for C1 amounts to 25 years. As a result, the concrete exhibits a higher $NPV$ and hence cost-effectiveness in the short- to medium-term, whereas the granite exhibits a higher $NPV$ in the long-term, i.e. over 75 - 150 years. This finding confirms the operator’s decision for granite as an invert material, in agreement with the result of the cost-effectiveness analysis above for severe abrasion conditions (Chapter 10.3.1).

![Figure 10.10 Costs expressed by the net present value of the concrete and granite installed in the Pfaffensprung SBT based on the abrasion rates determined between 2012 and 2015](image)

Although this evaluation is based on prototype-scale, the time series of only four years is rather short compared to the design life time of the Pfaffensprung SBT, which is generally 50 to 100 years or even longer. The estimation of long-term performance of the invert materials is afflicted by significant model uncertainties and thus is difficult to reliably predict. Hence, continued abrasion monitoring is necessary to confirm the present findings. Moreover, this evaluation is site-dependent and is only indicative for other sites. Direct application to other projects is only possible for similar hydraulic and sediment transport conditions as well as sediment and material properties.

### 10.3.3 Cost-effectiveness of the Runcahez invert materials

The $NPV$ of the concretes installed in the Runcahez SBT were calculated based on the actual abrasion rates between 1996 and 2014, for an accounting period of $T = 80$ yr. The $NPV$ was determined based on the investment cost and estimated annual maintenance cost of 5 CHF/m². The initial material thickness is 30 cm and the material replacement is assumed to take place as soon as cumulative mean abrasion depth reaches 2/3 of the initial material thickness.
Figure 10.11 illustrates the resulting NPV of the concretes installed in the Runcahez. The mean annual abrasion rates of the invert materials after 19 years (except from the RCC, which existed only 4 years) was $a_m = 1.15 \pm 0.25$ mm, and the replacement intervals are between 140 and 220 years so that no replacements take place within the considered period of $T = 80$ yr. The tested materials exhibited similar NPVs, except the polymer concrete (PC), which has a considerable lower NPV due to high investment. The highest NPV and hence highest cost-effectiveness was obtained for the roller compacted concrete (RCC). However, the abrasion rate of the RCC was computed based on only 4 years of data from the properly compacted zone. A direct comparison with the other abrasion rates obtained from a 5 times longer time series hence includes an uncertainty. Apart from the RCC, the high performance concrete (HPC) exhibits the highest NPV, followed by the steel fiber reinforced concrete (SF), the silica fume concrete (SC) and the PC. However, similar investment costs and the absence of material replacements resulted in similar NPVs, except for PC.

![Graph showing NPV vs. time for different concretes]

Figure 10.11 Costs expressed by the net present value of the concretes installed in the Runcahez SBT based on the abrasion rates determined between 1996 and 2014

Figure 10.12a shows the NPV as a function of splitting tensile strength $f_{st}$. The NPV decreases with increasing $f_{st}$ and follows $NPV = -847.4 f_{st}^{0.19}$ with $R^2 = 0.91$, neglecting the PC. A similar trend was observed in the cost-effectiveness analysis for the moderate abrasion conditions (Figure 10.9).
Figure 10.12  a) Net present value as a function of splitting tensile strength for the invert materials installed in the Runcahez SBT

\[ NPV = -847 \cdot f_{a}^{0.19} \]

\[ R^2 = 0.91 \]
11 Design and operation recommendations

No design guideline with applicable standards for SBTs and hydraulic structures exposed to hydroabrasion has been available so far. The German standard DIN 206-1 deals with hydroabrasion at concretes, but only in concept. It distinguishes three different exposure classes (medium, strong, very strong) and describes some requirements considering material compositions, curing and properties. However, the classification is rough and does not consider the amount of sediment, the stress duration or the hardness and shape of the sediment, which are all important factors regarding hydroabrasion.

Therefore, this knowledge gap shall be reduced in the present study by providing a state-of-the-art summary that includes the findings presented herein concerning design and operation of SBTs with a focus on hydroabrasion of the invert and bypass efficiency of the SBT. The design recommendations regarding invert abrasion also apply to other hydraulic structures exposed to sediment-laden high-speed flows provided that the hydraulic conditions are comparable to SBT flows.

11.1 Tunnel design: Course in plan view, longitudinal and cross-sectional profiles

Bends on the tunnel axis in plan view induce secondary currents (cf. Chapter 3.1.3). These currents cause high bed shear stresses and sediment transport concentrations on the inner side of a bend, and hence initiate the self-intensifying abrasion process. Their effects are not only visible in the bends, but propagate further downstream of a bend. At the Mud Mountain SBT ($R \approx 54$ m, $\alpha \approx 50^\circ$), abrasion measurements showed that a uniform flow field with corresponding bedload transport characteristics and abrasion patterns was only re-established about 150 m downstream of the bend (cf. Chapter 2.4.4). At the Pfaffensprung SBT ($R \approx 140$ m, $\alpha \approx 40^\circ$) the abrasion pattern indicates that the bending effect on the flow field did not disappear until the tunnel outlet located 180 m downstream of the bend. Similarly, the bedload measurements at the Solis SBT ($R = 145$ m, $\alpha = 46^\circ$) showed strong bedload transport concentrations close to the right side of the tunnel 105 m downstream of a right-hand bend (Hagmann et al. 2015a). Therefore, to mitigate high local abrasion rates, horizontal flow redirection, e.g. bends or changes of the cross section geometry, should be avoided in the design.
of a SBT whenever feasible. If not avoidable, local invert strengthening is recommended not only at the bend but also downstream.

There is a trade-off regarding the SBT slope. While the slope must be sufficiently high to avoid sediment deposition in the tunnel, it should be as low as possible to minimize shear stresses and the extent of invert abrasion (Hagmann et al. 2016). These requirements generally result in SBT slopes between 1 and 4% (Auel and Boes 2011).

From the hydraulic point of view, large cross sections providing high design discharges are preferred. However, economical and technical limitations restrict the cross section geometry and size. Flat bed design is favored over concave bed shapes in order to avoid high stress concentrations (Boes et al. 2014).

### 11.2 Hydraulics and sediment transport

Free-surface flows are preferred in a SBT, but pressurized flow are also possible depending on the location of the intake and the gates that determine the control section (Hagmann et al. 2016, Boes et al. 2017). For free-surface flow, choking should be avoided to achieve safe operating conditions with a sufficient cross-section for aeration. The SBT design discharge depends on site-specific hydrology, bedload transport capacity of the river, sediment supply, water demand and capacity of spillway and bottom outlets (Kashiwai et al. 1997). These requirements usually lead to a design discharge between a 1 and a 10 year flood event (Vischer et al. 1997, Sumi et al. 2004, Hagmann et al. 2016). Changes in the invert surface roughness caused by hydroabrasion during the lifespan of a SBT must be accounted for when determining the hydraulic operating conditions.

The sediment transport capacity should be determined by using bedload transport equations for fixed beds (Equations (3.47) to (3.50)) and selecting a conservative approach, i.e. using the most conservative bedload transport equation and taking into account that sediment transport capacity always has to exceed the possible sediment supply to avoid sediment deposition and clogging of the SBT. To achieve this, supercritical open channel flow in the tunnel is generally required (Hagmann et al. 2015b).

Depending on flow conditions the sediment particles are transported in sliding, rolling or saltation mode and cause grinding, rolling or saltating impact stresses and hence wear on the invert. Hence, the stress regime is usually a combination of these three stress types because of a certain grain size distribution of the sediment transport and due to natural variations in the
hydraulic and sediment transport conditions. However, the governing process causing hydroabrasion is saltation, whereas sliding or rolling do not cause significant wear (Sklar and Dietrich 2004, Beer et al. 2014, Lamb et al. 2015). The energy transmitted to the invert and hence the extent of the abrasion depends on the rolling, saltation and suspension probability of the transported sediment. They depend on the transport stage, i.e. the ratio of Shields number to critical Shields number for initiation of sediment transport, and the transport mode (cf. Chapter 3.2.3). Therefore, for an optimal SBT design, hydraulic conditions, sediment grain size distribution and hence sediment transport modes have to be determined (Boes et al. 2014). The particle impact velocity, which is decisive regarding abrasion mechanics, increases with increasing flow velocity (Auel et al. 2014b). The impact energy furthermore increases with particle mass, so that a combination of large particle mass and high flow velocity cause high invert abrasions. Therefore, regarding the flow velocity there is a trade-off between minimizing the impact energy and hence abrasion rate, and providing a sufficient sediment transport capacity to prevent clogging.

Laboratory investigations revealed that the 2D or 3D flow field in SBTs is governed by the tunnel width to flow depths ratio, $b/h$ and strongly affects the bed shear stress distribution (Auel, 2014). In narrow cross sections with $b/h < 4 - 5$, i.e. 3D flow, strong secondary current cells develop in straight sections and lead to 20 - 50% higher bed shear stresses close to the side walls. This is where higher sediment transport rates and hence higher abrasion rates occur. As a result, incision channels form, which grow in depth and width along the side walls. This effect disappears for $b/h > 5$, i.e. 2D flow, but such wide SBT cross section geometries are not economic and hence bed shear stress concentrations are unavoidable. The formation of incision channels can be prevented by the installation of higher abrasion-resistant materials along the tunnel walls compared to the material in other tunnel sections (cf. Chapter 11.6).

Another measure to reduce the invert abrasion is to reduce the mass and particle size of the bypassed sediment by means of sediment trapping facilities as reported in the following chapter.

### 11.3 Sediment trapping facilities

Most SBTs are designed and operated to pass all incoming sediments (Hagmann et al. 2016). However, bypassing of coarse sediment particles can cause massive abrasion on tunnel inverts and result in cost-intensive maintenance. Therefore, upstream sediment traps may be a suitable measure to reduce SBT invert abrasion (Kashiwai et al. 1997, Vischer et al. 1997, Sumi et al. 2004b, Fukuroi 2012). While sediment traps near the reservoir head mainly retain the coarser
part of the sediment, check dams decrease the sediment yield by reducing the channel slope and the hill slope erosion rate (Kondolf et al. 2014, Auel et al. 2016c). The installation of check dams in a cascade along a river reach is particularly useful in this regard. Both methods have been successfully applied. The coarse sediments were trapped and dredged near the inlet of the Miwa SBT. This reduced not only the reservoir sedimentation, but also the abrasion on the tunnel invert (Sumi et al. 2012, Sakurai and Kobayashi 2015). A further example is the Nunobiki dam, at which the upstream check dams (also called Sabo dams) significantly reduced the sediment inflow to the Nunobiki dam by 14%, (Auel et al. 2016c). The trapped sediments became a raw material as by recycling and thus can be used for new construction projects or can be used for replenishment in the tailwater. Like that, the sediment continuity not only for the fine sediments but also for the coarser fractions can be restored and the ecomorphology of the river system can be enhanced to pre-dam status and thus natural-like conditions (Friedl et al. 2017).

The drawback of this approach is that the retention facilities may be filled with sediment within only one flood event and require frequent maintenance to ensure proper function and to reduce the risk of a large sediment pulse in case of failure (Wang and Kondolf 2014).

### 11.4 Guiding structure, intake and outlet

The guiding structure is used to separate muddy, sediment-laden flow from the clear water and to divert them towards the SBT intake. Furthermore, it ensures a controlled release of the surplus flow that exceeds the tunnel discharge capacity.

Generally, the guiding structure is located next to the bypass intake and crosses the reservoir (Vischer et al. 1997, Sumi et al. 2004b). Different diversion types are required depending on the sediment size of the inflowing sediment (Kashiwai et al. 1997, Parsons et al. 2015). While wash load is uniformly distributed within the water column, suspended sediment and bedload are found in higher concentrations near the river bed. Hence, the diversion facility needs to guide the lower part of the water column to the intake if the sediment inflow is dominated by bedload transport, while the upper clean part can be allowed to enter the reservoir. In contrast, the bypass efficiency of wash load increases proportionally to the ratio of the bypassed discharge to the inflow discharge, requiring a higher discharge capacity.

Two different locations are generally possible for the SBT intake. Both affect the design of the entire SBT as well as of the operating regime of the SBT and the reservoir during sediment
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Routing (Auel and Boes 2011). The common location for the SBT intake is at the reservoir head (= type (A) SBT). Alternatively the SBT intake can be within the reservoir between the reservoir head and the dam (= type (B) SBT) as at the Solis Reservoir. The flow conditions at the intake of type (A) SBTs is free-surface, and the tunnel invert is constructed plain to the river bed. The flow in the SBT is accelerated to supercritical flow conditions by a short and steep acceleration section downstream of the intake gate. The flow at the intake of type (B) SBTs is generally pressurized due to a large energy head. Supercritical free-surface flow conditions exist downstream of the intake gate, representing the control section. Therefore, no acceleration section is required. Some SBTs are gate-controlled at the downstream end for various reasons, mainly due to site constraints. These SBTs typically feature the whole flow regime spectrum from free-surface via transitional to pressurized flow, depending on the discharge, head and invert slope. The tunnel lining design has to account for the pressure pulsations stemming from the transitional flow regime with flow choking and pulsating flow (Boes et al. 2017).

If the reservoir is located below the timberline, considerable amounts of driftwood can be mobilized during flood events. Hence, measures to hinder driftwood entering the SBT and provoking clogging for instance by a skimming wall or upstream located trash racks are advisable. However, vertical pile racks can interrupt sediment transport due to the retained driftwood and hence should be carefully applied (Hagmann et al. 2016).

The outlet structure safely releases the bypassed water-sediment mixture into the downstream river reach. The outlet is generally located above the river bed, to avoid sediment depositions in the vicinity of the SBT outlet, which can reduce the SBT discharge capacity due to backwater effects. Scour of the embankments or the river bed in the tailwater can be prevented by dissipating the hydraulic energy in natural or artificial energy dissipaters like plunge pools with rock bed or stilling basins, respectively.

11.5 Hydroabrasion prediction

Invert abrasion rates can be predicted based on various available abrasion models (cf. Chapter 3.5). Amongst others, the saltation abrasion model (SAM) introduced by Sklar and Dietrich (2004), the saltation abrasion model adapted by Auel et al. (2017b) (SAMA) and the abrasion model introduced by Ishibashi (1983) are applicable for the hydroabrasion prediction in SBTs. However, the application range, which the models were developed for should be considered. The SAM was developed based on particle motion characteristics for sub- to low supercritical flow regimes, whereas the other two focus on supercritical to highly supercritical flow regimes.
Moreover, the SAMA is preferred over the Ishibashi model in particular for brittle materials due to its presumably higher model accuracy (Chapter 10.3.3).

The SAM and the SAMA were developed for brittle materials, but they can be also applied to ductile materials, with a certain model uncertainty due to the different abrasion behavior of materials (cf. Chapter 3.3.3). In contrast, the Ishibashi model applies to concrete and steel. It is expected to yield more realistic results compared to the SAM and SAMA for ductile materials such as steel. However, the model accuracy for steel could not be evaluated at this time due to lack of data. The abrasion estimation for steel with both the Ishibashi model and the SAMA therefore need careful interpretation.

The prediction accuracy of the models strongly depends on the accuracy of the model input parameters, i.e. measurable / computable hydraulic and sediment transport parameters, invert material properties and the selection of the abrasion coefficients, i.e. $k_v$, $c_1$ and $c_2$. Therefore, the material properties determined by means of material tests should be preferred over standard or calculated values. The abrasion coefficients are related to the efficiency of energy transfer from particles impacting on the invert material and depend on various invert material and sediment properties. These parameters were experimentally determined herein by means of prototype investigations. The mean abrasion coefficients listed in Chapter 10.2.3, Table 10.2 and Table 10.3 can be used for abrasion estimation of concrete, rock and steel. These values are proposed for long-term estimations. However, the range of the corresponding parameters induce a considerable prediction error, in particular for short-term analysis, due to model input parameter uncertainties (Chapter 10.2.3). For example, the minimum and maximum abrasion coefficients determined for each operational year at the Pfaffensprung SBT deviated by $-40$ to $+240\%$, respectively, from the four-year averaged value (Table H.2). To reduce the uncertainty, the particular material-specific values obtained from the SBTs are recommended (cf. Appendix H).

All three abrasion models are one-dimensional models, which are able to predict spatially averaged abrasion rates and depths. However, abrasion rates are generally unevenly distributed resulting in bumpy abrasion patterns, potholes or incision channels depending on the tunnel aspect ratio (cf. Chapter 3.1.3). Moreover, the abrasion pattern depends on the material type. Concrete tends to show an undulating abrasion pattern. Modular materials such as granite or cast basalt pavements exhibit abrasion concentrations in the form of grooves along the joints. This spatial variation of abrasion rates needs to be considered in the design of the invert material thickness, which depends on the expected maximum rather than the spatially averaged abrasion
depths. Based on the prototype experiments, the ratio between maximum and spatially averaged abrasion depths for concrete and granite inverts ranges between 1.6 and 2.2 confirming Auel’s (2014) laboratory observations. However, larger ratios might be possible due to dissimilarities of the invert material quality or due to the self-intensifying process of hydroabrasion. Therefore, these ratios serve only as a reference and could be even higher for a conservative design of the invert material thickness.

11.6 Invert material

SBTs are exposed to massive hydroabrasive impact due to their primary function of bypassing sediment-laden discharges. Supercritical flow conditions and high sediment transport rates dominate SBT operating conditions and can cause severe abrasion mainly on the invert and on the lower part of the walls (Vischer et al. 1997, Fukuroi 2012, Boes et al. 2014). This can result in high maintenance cost, in particular if abrasion was not considered during the design phase (Vischer et al. 1997, Sumi et al. 2004b, Inoue 2009, Auel and Boes 2011). Hence, to ensure sustainable use of a SBT, tunnel invert abrasion, i.e. site-dependent abrasion resistance as well as the cost-effectiveness of the invert materials have to be considered in the planning stage (Sumi et al. 2004b, Boes et al. 2014).

The abrasion resistance of an invert material can be estimated by means of the abrasion models with the abrasion coefficients listed in Chapter 11.5. These models account for abrasion by particle impingement, which is the governing process causing hydroabrasion. However, the models include significant uncertainties so that the abrasion resistance can be considerably under- or overestimated. To reduce the uncertainty, the abrasion resistance of invert materials can be experimentally determined. Since the scalability of laboratory results to prototype scale is not clarified yet, prototype experiments performed under the real site-specific operating conditions are recommended. However, such experiments are often not possible due to technical, financial or temporal reasons. Hence, the experiences of comparable facilities deliver important information and can help to evaluate the predicted abrasion resistance obtained from the abrasion models.

Prototype experiments in the Pfaffensprung SBT revealed that the installed granite exhibits an approx. 6 times higher abrasion resistance compared to the installed high-strength concrete (Chapter 7.5.3). This result suggests that granite is the better material for the Pfaffensprung SBT. However, granite pavements require specialized structural design and implementation, which might be challenging to ensure. Moreover, visual surveys revealed that natural stone, i.e.
granite and in particular cast basalt pavements, indeed have a higher abrasion resistance against mainly grinding action, but tend to fracturing by impingements of large particles because of their brittleness. This problem was successfully overcome in the Pfaffensprung SBT by an optimized geometry of the granite blocks, i.e. 1 m × 1 m × 0.3 m, and a proper implementation.

Considering concrete, the results from the Runcahez SBT revealed that the abrasion resistance of concrete generally increases with increasing compressive and splitting tensile strength (Chapter 8.5). Furthermore, the abrasion resistance was reported to increase with increasing invert material strength, increasing elasticity, increasing fracture energy, increasing density, decreasing porosity, increasing hardness and decreasing crystal and clast size (Bania 1989, Kunterding 1991, Haroske 1998, Sklar and Dietrich 2001, 2004, Beyeler and Sklar 2010, Momber 2014, Scheingross et al. 2014, Lamb et al. 2015). Although the effects of these parameters were rarely quantified, in particular for the hydroabrasive stress regime existing in SBTs, this qualitative trends can serve as a guideline in the design of the invert material.

Although there is no design guideline with applicable standards for SBTs and other hydraulic structures exposed to hydroabrasion, minimum requirements in particular regarding the material strength were reported. Based on laboratory as well as prototype experiments literature provides a minimum concrete compressive strength of $f_c \geq 60$ MPa for a stress regime defined by a mean flow velocity of $U \geq 5$ m/s and specific sediment transport rate $q_s \geq 1$ kg/s (Jacobs 1994, Haroske 1998, Nakajima et al. 2015), which is comparable to the conditions in the SBTs. This value was recently confirmed by Hagmann et al. (2016) based on prototype experiences from Swiss and Japanese SBTs resulting in a minimum compressive strength of $f_c = 50 - 70$ MPa for SBTs.

Regarding the abrasion resistance, the minimum required material strength mentioned above might serve as a reference. However, the cost-effectiveness depends not only on the abrasion resistance, but also on the site-specific abrasion conditions and the target life time. It was found that for long-term applications under severe abrasion conditions, i.e. a hydroabrasion index $HAI > 2000$ m·to/yr (Equation (10.3)), materials with a high tensile strength exhibit the highest cost-effectiveness, despite considerably higher investment costs (cf. Chapter 10.3). However, for moderate to low abrasion conditions or short-term applications, less expensive materials exhibit the highest cost-effectiveness despite lower abrasion resistances. As a result, for long-term applications under severe abrasion conditions hard rock, steel or (ultra-)high-performance concretes are recommended. For the latter a compressive strength of $f_c \geq 100$ MPa might be
recommendable for facilities exposed to coarse sediment transport (cf. prototype results from the Pfaffensprung SBT with $d_m = 250$ mm, Chapter 10.3.2).

The abrasion protection layer should be as thin as possible while respecting the target lifetime and accounting for the characteristic abrasion patterns in order to optimize the material usage and hence the investment cost (cf. Chapter 11.5). Furthermore, the tunnel cross section may be divided into areas of different hydroabrasive impacts, which are accordingly equipped with materials of different hydroabrasion resistance. This approach has already been applied at a number of SBTs (Kashiwai et al. 1997, Jacobs et al. 2001, Baumer and Radogna 2015, Müller and Walker 2015, Nakajima et al. 2015, Oertli and Auel 2015). In general, the SBTs consist of commonly used materials, mostly concrete or natural stones, and feature more resistant materials at the invert, e.g. high-strength concrete, cast basalt or steel. In some SBTs, even the lower part of the tunnel walls is strengthened. By introducing additionally a transversal subdivision of the invert into highly and less exposed areas (according to the spanwise bedload transport or the bed shear stress distribution, cf. Chapter 11.2) the material use and hence the investment cost could be further optimized.

Since hydroabrasion is a self-intensifying process triggered and intensified by irregularities, joints, notches, gaps, offsets and cracks on the invert should be avoided. The installation of modular invert systems such as cast basalt or granite pavements should carefully be planned and executed as follows: (1) staggered installation to avoid overlaps of downstream propagating “abrasion shadows” provoked by longitudinal joints, (2) flush installation without joints and offsets to minimize spots of weaknesses and (3) strong bonding between the bedding and invert as well as tight placement of modular lining systems without joints to withstand both particle impact and hydraulic uplift forces.

### 11.7 Operation

The main aim of SBTs is to efficiently divert the incoming sediments past the dam to prevent reservoir sedimentation, and additionally to restore sediment and water regimes in the downstream river reach to pre-dam status and thus natural conditions. The operational regime should hence achieve an optimum bypass efficiency, which strongly depends on the site-specific hydrology, reservoir size, hydraulic conditions and sediment size distribution. Moreover, the operational regime has to be optimized regarding the reservoir purposes while respecting legal restriction. Therefore, it differs from site to site and results in operational durations between a few hours to a few months per year (Chapter 2.4.2).
The SBT operation is mostly independent of the reservoir operation in particular for SBTs with an intake at the reservoir head, i.e. type (A) SBTs. In contrast, the hydraulic and sediment transport conditions in SBTs with an intake within the reservoir, i.e. type (B) SBTs, strongly depend on the reservoir water level. Therefore, the operation of the type (B) SBTs needs to be carefully coordinated with that of the reservoir. Decisive parameters are the reservoir water level and the approach flow discharge, because they determine the bed shear stress and hence sediment transport in the river, the reservoir and finally in the SBT. The reservoir water level needs to be lowered prior to a SBT operation to ensure the inflowing sediments remain in motion and enter the SBT. The prototype result from the Solis SBT revealed that the sediment transport in the SBT increases with decreasing reservoir water level (Chapter 6.7). Hence, the reservoir water level should be kept as low as possible in order to optimize the bypass efficiency. To this end, the amount of water released through the outlet works should be maximized during flood peak discharges. Regarding the operation of the turbines, the critical water level avoiding aeration from air-entraining intake vortices should be respected and, if not existing, the installation of an independent cooling system of the alternator, i.e. not being fed from the power waterway to prevent clogging of filters, might be advisable.

The lowering of the reservoir water level results in a release of water otherwise used for electricity production and causes hence revenue losses. This results in a trade-off between maximum bypass efficiency and minimum financial loss. The optimal operating conditions can be determined by means of numerical investigation and / or physical model tests in the design. In general, a shorter operation duration with higher sediment transport rates is preferable over a longer operation duration with lower sediment transport rates. Therefore, the operation duration for type (B) SBTs is generally significantly lower compared to those for type (A) SBTs, which do not require a reservoir water level lowering for efficient sediment bypass. In order to optimize the performance of the facility irrespective of the SBT type, it is recommended to real-time monitor the bypass efficiency.

### 11.8 Monitoring

Optimization of bypass and reservoir operations with regard to bypass efficiency, invert abrasion and ecomorphology of the downstream river reach requires knowledge on the site-specific relations between inflow, outflow, reservoir water level (in particular for type (B) SBTs, cf. Chapter 11.6) and sediment transport. To this end, a comprehensive monitoring of
the hydraulic and sediment transport conditions in the river, the reservoir and the SBT, ideally in real-time, is needed.

For real-time bedload transport monitoring, indirect measurements such as geophones (cf. Chapter 4.2.1) or hydrophones (Koshiba et al. 2016) are recommended due to their robust designs. Both devices register the impacts of passing bedload particles. The main difference of these two techniques is the detectable threshold particle size, which is about 2 cm for the Swiss plate geophone and 4 mm for the hydrophone. Hence, according to the expected GSD, either one or the other or both techniques in combination should be installed. Note that bedload transport may vary significantly in spanwise direction, and hence should be measured across the entire channel width (Albayrak et al. 2015).

The suspended sediment transport can be monitored in real-time by numerous techniques, e.g. acoustic or optical instruments (Felix et al. 2016). Among them, turbidimeters are relatively inexpensive, easy to handle and the most suitable for application in high-speed flows as in SBTs. However, they need a site-specific calibration since particle size, shape, composition and color affect the measurements (cf. Chapter 4.3.8). For a representative sensor calibration it is mandatory to include samples of a large discharge range, in particular from flood events, when most of the sediments are transported (Gippel 1995, Teixeira et al. 2016).

The sediment accumulation in the reservoir can be determined either indirectly based on the measurements of sediment inflow and outflow of the reservoir, or directly by means of bathymetric surveys. For the latter the accumulation volume is determined by comparing digital elevation models of the bathymetry obtained from echo sounding (Jakubauskas and deNoyelles 2008).

The abrasion of the tunnel invert can be monitored by means of various methods, among them geodetic leveling and 3D-laser scans are commonly applied in SBTs (Jacobs et al. 2001, Nakajima et al. 2015). Moreover, digital photogrammetry has been successfully used for close range in-situ measurements of bedrock erosion within the sub-millimeter range (Rieke-Zapp et al. 2012). However, its application to large scales is relatively time consuming and requires a powerful illumination in the SBT, which is challenging.

The effect of the SBT operation on the downstream ecomorphology should be monitored in order to optimize the SBT operation also with regard to the fauna and flora. Therefore, various techniques can be used. Line samplings or sieve analysis provide information on the particle size distribution. Changes in the river bed topography can be quantified by means of manually measured cross-sectional profiles or complete topography maps obtained from digital
photogrammetry or airborne laser scanning (Chandler et al. 2000, Facchini et al. 2017). The latter was successfully applied at the Albula, using bathymetric LiDAR surveys with a green laser, which penetrates the water surface and hence measures not only the dry but also the wet perimeter of the river bed (Facchini et al. 2017). The quality of the aquatic habitat is expected to increase by the SBT operation due to the restored sediment continuity (Kondolf et al. 2014).

The effect of the SBT on the ecology of a river can be assessed based on several indicators, such as: sediment respiration, periphyton biomass and macroinvertebrate density and richness (Martin et al. 2015, Auel et al. 2017c, Martin et al. 2017). The effect of SBT operation on the fish can be determined based on the fish population. Electrofishing is a common method used to sample fish populations, but requires a careful and professional implementation to avoid permanently harmed fishes (Tamagni 2013).

The techniques listed above are applicable to monitor the bypass efficiency and to analyze the effect of SBT operation on the downstream river reach. However, it should be noted that this list does not claim for completeness and should be adopted based on the site-specific conditions and requirements.
12 Conclusions and outlook

Reservoir sedimentation can cause various severe problems affecting economy, environment and operational safety. SBTs are an efficient and holistic measure against reservoir sedimentation by diverting sediment-laden flows past the dam. Hence, the natural sediment continuity is mimicked or even restored and the manifold problems linked to reservoir sedimentation are reduced or even solved. However, high flow velocities and sediment transport rates can cause hydroabrasive damage on the invert, requiring expensive refurbishment and hence significantly reduce the cost-effectiveness of SBTs.

The main goal of the present study is to contribute to a sustainable design and operation of SBTs and other hydraulic structures suffering from severe hydroabrasion. To this end, the abrasion resistance and abrasion behavior of various invert materials was investigated at three Swiss SBTs, namely the Solis, Pfaffensprung and Runcahez SBTs, as well as in the laboratory. The hydraulic operating conditions at the studied SBTs were continuously monitored, while the sediment transport rates in the SBT were estimated based on the hydrographs and validated using bathymetric data and/or former studies including field surveys and numerical simulations. At the Solis SBT, the sediment transport, i.e. the suspended sediment and the bedload transport, were additionally continuously monitored by using turbidimeters and a Swiss plate geophone system, respectively. The abrasion depths on the tunnel inverts were determined based on regularly performed laser scans or geodetic leveling. The abrasion resistance of the cementitious invert materials was additionally tested in the laboratory using a Dresdner drum. The obtained results including literature data were used to evaluate the applicability and accuracy of abrasion prediction models and to investigate the upscaling from laboratory to prototype applications. Finally, the cost-effectiveness of a range of invert materials was determined and compared based on different abrasion scenarios. The main results of the present study and an outlook are summarized below.

This thesis is a contribution to the understanding of the hydroabrasion processes in high-speed sediment-laden open-channel flows. Moreover, the findings serve for the calibration and validation of numerical abrasion models and give new insights into the flow and sediment transport characteristics in SBTs and reservoirs in relation with their operation regimes.
12.1 Invert abrasion

The abrasion resistance of various invert materials was investigated in this study by means of prototype tests in three Swiss SBTs and laboratory tests using a Dresdner Drum. Moreover, the scalability of laboratory drum test results to prototype scale was investigated and abrasion models were calibrated.

The prototype experiments showed that hydroabrasion is a self-intensifying process triggered by discontinuities and structural weaknesses. Monolithic materials tend to show undulating abrasion patterns, while modular invert systems, e.g. granite or cast basalt pavements, suffer abrasion concentrations along the joints. Proper, accurate and professional implementation of the invert material minimizes the risk of fast propagating invert abrasion. However, despite this, abrasion concentrations occurred. Strong bedload transport and abrasion concentrations were observed at the inner side of tunnel bends, which are attributed to the Prandtl’s first type of secondary currents. The abrasion patterns in the Pfaffensprung SBT as well as the bedload transport measurements in the Solis SBT indicated that these secondary currents affect the flow field and bedload transport not only in the bend but also in the downstream. Different abrasion patterns were observed in the Runcahez SBT, which represents, to the author’s knowledge, the first long-term in-situ investigation on hydroabrasion with an observation period of 19 years. In this tunnel, incision channels formed along the side walls due to bed shear stress peaks close to the side walls caused by Prandtl’s second type of secondary currents occurring in narrow open channel flows, i.e. for \( b/h < 4 - 5 \). The ratio between the maximum (= 95%-percentile) and spatially averaged abrasion depths was 1.6 to 2.2 irrespective of the SBT or the invert material. Similar abrasion patterns and ratios between maximum and spatially averaged abrasion depths were observed in physical scale model tests (Auel 2014). This indicates that the abrasion mechanics in the SBT were appropriately reproduced in the laboratory model and hence confirm the use of the physical scale model to investigate abrasion processes in SBTs.

One of the important findings from the study in the Pfaffensprung SBT is the about 6 times higher abrasion resistance of the Urner Granite compared to the installed high-strength concrete. The results from the Runcahez SBT showed that the abrasion resistance of invert materials generally increases with increasing compressive and splitting tensile strengths. However, selection of optimum invert materials depends on their cost-effectiveness, which is a function of the material abrasion resistance, as well as of the material cost, abrasion conditions, i.e. hydraulic and sediment transport conditions, and target life time. The cost-effectiveness analysis of the studied invert materials showed that hard rocks and ( ultra)-high performance
Conclusions and outlook

Concrete are more suitable for long-term applications under severe abrasion conditions despite high investment cost, whereas less abrasion-resistant and less expensive materials are more suitable for moderate to low abrasion conditions or short-term applications.

The calibration of existing abrasion models based on the prototype measurements performed herein resulted in abrasion coefficients, which are generally in good agreement with literature data, in particular with the concrete value proposed by Auel et al. (2017b). The model accuracy is significantly enhanced by using material-specific abrasion coefficients, namely for concrete, hard rock and steel, instead of the proposed values from the literature. Despite this, there is still a considerable uncertainty, because the investigated models do not account for the abrasiveness of the transported sediments and insufficiently determine the abrasion resistance of the invert material. Therefore, its abrasion resistance was further investigated by means of laboratory tests for a range of invert materials.

The results obtained from the Dresdner drum using concrete samples from the Pfaffensprung SBT showed that abrasion rates in the laboratory were about 300 times higher compared to the prototype scale values. However, the abrasion mechanics in laboratory drum tests significantly deviate from those in the prototype as well as from those considered by the abrasion models due to deviating hydraulic and sediment transport conditions. Therefore, the abrasion models are not applicable to the results obtained from the Dresdner drum tests, and scaling from laboratory to prototype scale is questionable.

12.2 Geophone measurements

The installation and use of the Swiss plate geophone system (SPGS) for continuous and real-time monitoring of bedload transport in the Solis SBT is a novelty as to the (I) operating conditions with highly supercritical flow and flow depths of more than 1 m, (II) inclination of the geophone plate against the flow direction and (III) unique field calibration. Moreover, together with the SPGS installed by the WSL in the Albula upstream of the reservoir, it allowed for the first time a direct comparison of bedload transport in the inflow and outflow of a reservoir using geophones as an indirect method to measure bedload transport.

The device was calibrated by means of laboratory and in-situ large-scale experiments. The results indicated that the SPGS is a suitable device for continuous bedload transport measurements in SBTs under high flow velocities, discharges and bedload transport rates. Moreover, it was found that the amplitude class method introduced by Wyss (2016) can be
applied to determine not only the transport rate but also the grain size distribution of the bedload transport. The inclination of the geophone plate of 10° (instead of the common installation flush to the channel bottom) seems to eliminate the flow velocity dependency of the calibration. This is a major finding since it implies that SPGS might be calibrated by means of laboratory instead of expensive and challenging field tests.

### 12.3 SBT operation and bypass efficiency

The bypass efficiency and its dependency on the operational regime of the SBT and the reservoir was investigated based on three examples, namely the Solis, Pfaffensprung and Runcahez SBTs. The latter two SBTs are of type (A), whereas the Solis SBT is the only existing SBT of type (B) with an intake within the reservoir instead of at the reservoir head. In contrast to the Pfaffensprung and the Runcahez studies, the data basis for the Solis study includes extensive measurements of suspended sediment and bedload transports into and out of the reservoir.

Despite completely different operation durations of more than 100 days versus approx. 1 day per year, the bypass efficiencies of the Pfaffensprung and Runcahez SBT were high, i.e. 98% and 83%, respectively, which is in agreement with the typical values for type (A) SBTs. The Pfaffensprung SBT directly bypasses the incoming sediments, whereas at the Runcahez SBT inflowing sediments are hindered to enter the reservoir by a large guiding weir and hence settle in front of the SBT. During SBT operations, the sediment deposits are eroded and conveyed past the dam.

In contrast, the Solis SBT exhibited a bypass efficiency of only 31%, which is considerably lower because of the different characteristics of type (B) SBTs. The results indicate temporary sediment deposition negatively affecting the bypass efficiency. Moreover, the bypass efficiency strongly depends on the operating regime of the SBT as well as of the reservoir. The sediment transport rate increases with decreasing reservoir water level and increasing approach flow discharge, and the total amount of bypassed sediment increases with operation duration.
12.4 Outlook

The present study investigates the abrasion resistance and abrasion behavior of various invert materials by means of prototype and laboratory tests. Although the data basis includes prototype results from 1 to 19 years, the observation durations are relatively short compared to common target life times of SBTs. Extended prototype experiments are therefore recommended to obtain representative long-term data. Moreover, systematic prototype and laboratory experiments using a physical scale model are needed to (I) quantify the effect of the invert material properties on their abrasion resistance, (II) determine the effect of sediment hardness and angularity on the abrasion rate, (III) evaluate the scalability of the laboratory results to prototype scale and (IV) provide further material-specific abrasion coefficients applicable for abrasion estimates. An ongoing research project at VAW aims at addressing these issues. In this study the abrasion resistance of foams and mortars with a range of compressive strengths will be investigated under various hydraulic conditions, sediment transport rates and sediment properties by means of physical scale model tests. Moreover, to enhance the representativeness of the results and to analyze the effect of the sediment abrasiveness the prototype experiments presented in this study will be continued, additionally including an analysis of the sediment properties.

The findings of the laboratory and the extended prototype studies, as referred above, are of prime importance to further enhance the prediction accuracy of the abrasion models by adding an abrasiveness term accounting for both the sediment hardness and angularity. Moreover, an integrated analysis of the two studies may help to establish a universal standardized laboratory test procedure to determine site-specific abrasion resistance of invert materials for a range of prototype conditions.

The determination of the abrasion resistance of invert materials based on abrasion models or laboratory tests includes significant uncertainties. Therefore, the choice of invert material has been often based on the experiences gained on site, which resulted in a relative small range of usually applied invert materials, i.e. different concretes, steel, granite, cast basalt. Material technology has been advanced in the last decades and originated many novel materials, such as ultra-high performance concretes, concretes with different admixtures (steel / glass / plastic / carbon fibers, superplasticizer, stabilizers, basaltic or rubber aggregate) and resins made of epoxy, polyurea elastomers or nanoparticles. The performance of these materials under hydroabrasive impact is mostly unexplored and therefore should be the topic of further research,
while considering not only the abrasion resistance but also the environmental impact, in particular of the nanoparticle resins.

In this study, the Swiss plate geophone system (SPGS) installed in the Solis SBT was calibrated by means of laboratory and *in-situ* experiments. The results indicated that the flow velocity-dependency of the calibration might be eliminated by inclining the geophone plate 10° against flow direction. This is a promising result, since it indicates that challenging and expensive field calibrations of the SPGS might be substituted by less expensive and elaborative laboratory calibrations. However, the present finding bases only on a few tests. Moreover, the *in-situ* calibration tests revealed a significant signal saturation due to high sediment transport rates. To enlarge the data basis and investigate the effect of sediment transport rate on the calibration further *in-situ* calibration tests in the Solis SBT are recommended. However, the test conditions in the Solis SBT are not discretionary. Therefore, additional laboratory and field experiments with a systematic variation of the test conditions, are recommended to reliably quantify the effects of the various parameters on the calibration and to assess the applicability of laboratory calibration.
13 Notation

13.1 Symbols

\( a \) Longest particle axis / abrasion depth \([\text{m}] / [-]\)

\( A \) Amplitude of geophone signal / area / integral constant \([\text{V}] / [\text{m}^2] / [-]\)

\( A_r \) Vertical abrasion rate \([\text{m/s}]\)

\( A_{rs} \) Gravimetric abrasion rate \([\text{g/h}]\)

\( A_{rv} \) Volumetric abrasion rate \([\text{m}^3/\text{s}]\)

\( b \) Channel width or intermediate axis of a grain \([\text{m}]\)

\( b' \) Bedload transport effective channel width after Marti (2006) \([\text{m}]\)

\( b_{ws} \) Water surface width \([\text{m}]\)

\( B \) Integral constant \([-]\)

\( BE \) Bypass efficiency \([-]\)

\( BE_{op} \) Operational bypass efficiency \([-]\)

\( BL \) Bed load \([\text{to}]\)

\( BL_{22} \) Bed load of particles larger than 22 mm \([\text{to}]\)

\( c_{1,\ldots,8} \) Coefficients \([-]\)

\( C \) Suspended sediment concentration \([\text{kg/m}^3]\)

\( C_a \) Suspended sediment concentration at reference level \([\text{kg/m}^3]\)

\( CAP \) Reservoir storage capacity \([\text{m}^3]\)

\( CIR = CAP/MAR = \) capacity-inflow ratio \([\text{yr}]\)

\( d \) Particle diameter \([\text{m}]\)

\( d_a \) Particle diameter of the armor layer \([\text{m}]\)

\( d_m \) Mean grain diameter \([\text{m}]\)

\( d_{m,\text{armor}} \) Mean grain size of the armor layer \([\text{m}]\)

\( d_{m,\text{substrate}} \) Mean grain size of the substrate \([\text{m}]\)
<table>
<thead>
<tr>
<th>Notation</th>
<th>Definition</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>( d_{xy} )</td>
<td>Characteristic grain size than which ( xy% ) of the material by weight is finer</td>
<td>[m]</td>
</tr>
<tr>
<td>( D )</td>
<td>( = 4R_h ) = hydraulic diameter for open channels or diameter</td>
<td>[m]</td>
</tr>
<tr>
<td>( D^* )</td>
<td>Dimensionless grain size</td>
<td>[-]</td>
</tr>
<tr>
<td>( DF )</td>
<td>Dilution factor</td>
<td>[-]</td>
</tr>
<tr>
<td>( E_{frac} )</td>
<td>Fracture energy</td>
<td>[J/m(^2)]</td>
</tr>
<tr>
<td>( E_i )</td>
<td>Particle kinetic energy</td>
<td>[J]</td>
</tr>
<tr>
<td>( E_k )</td>
<td>Total deformation work by saltating particles</td>
<td>[J]</td>
</tr>
<tr>
<td>( f_{bst} )</td>
<td>Bending tensile strength (also called flexural tensile strength)</td>
<td>[Pa]</td>
</tr>
<tr>
<td>( f_c )</td>
<td>Compressive strength</td>
<td>[Pa]</td>
</tr>
<tr>
<td>( f_c^* )</td>
<td>Reference value</td>
<td>[Pa]</td>
</tr>
<tr>
<td>( f_{c,cyl} )</td>
<td>Cylindrical compressive strength</td>
<td>[Pa]</td>
</tr>
<tr>
<td>( f_i )</td>
<td>Exponent</td>
<td>[-]</td>
</tr>
<tr>
<td>( f_r )</td>
<td>Macro roughness factor ( (f_r = 1 ) for moderate macro roughness)</td>
<td>[-]</td>
</tr>
<tr>
<td>( f_{st} )</td>
<td>Splitting tensile strength</td>
<td>[Pa]</td>
</tr>
<tr>
<td>( f_t )</td>
<td>Uniaxial tensile strength</td>
<td>[Pa]</td>
</tr>
<tr>
<td>( f_u )</td>
<td>Fracture stress</td>
<td>[Pa]</td>
</tr>
<tr>
<td>( f_y )</td>
<td>Yield stress</td>
<td>[Pa]</td>
</tr>
<tr>
<td>( F )</td>
<td>Suspension parameter</td>
<td>[-]</td>
</tr>
<tr>
<td>( F_i )</td>
<td>Impact force</td>
<td>[J]</td>
</tr>
<tr>
<td>( g )</td>
<td>Gravitational acceleration ((=9.81 \text{ m/s}^2))</td>
<td>[m/s(^2)]</td>
</tr>
<tr>
<td>( h )</td>
<td>Flow depth</td>
<td>[m]</td>
</tr>
<tr>
<td>( h_f )</td>
<td>Head loss</td>
<td>[m]</td>
</tr>
<tr>
<td>( HAI )</td>
<td>Hydroabrasion index</td>
<td>[m\cdot\text{to/yr}]</td>
</tr>
<tr>
<td>( H_p )</td>
<td>Particle hop height</td>
<td>[m]</td>
</tr>
<tr>
<td>( HQ_x )</td>
<td>Discharge of an ( x ) year flood event</td>
<td>[m(^3)/s]</td>
</tr>
<tr>
<td>( J )</td>
<td>Initial investment</td>
<td>[CHF]</td>
</tr>
<tr>
<td>Notation</td>
<td>Description</td>
<td>Units</td>
</tr>
<tr>
<td>----------</td>
<td>----------------------------------------------------------------------------</td>
<td>-------</td>
</tr>
<tr>
<td>$k_{BL22}$</td>
<td>Share of $BL_{22}$</td>
<td>[-]</td>
</tr>
<tr>
<td>$k_E$</td>
<td>Aggregate coefficient</td>
<td>[Pa$^{2/3}$]</td>
</tr>
<tr>
<td>$k_s$</td>
<td>Equivalent bed/sand roughness height</td>
<td>[m$^{1/3}$/s]</td>
</tr>
<tr>
<td>$k_s^*$</td>
<td>$= k_s U_*$</td>
<td>[-]</td>
</tr>
<tr>
<td>$k_{st}$</td>
<td>Strickler’s value for the roughness</td>
<td>[m$^{1/3}$/s]</td>
</tr>
<tr>
<td>$k_{St,S}$</td>
<td>Strickler’s value for the bed roughness</td>
<td>[m$^{1/3}$/s]</td>
</tr>
<tr>
<td>$k_{St,r}$</td>
<td>Strickler’s value for the grain roughness of the bed</td>
<td>[m$^{1/3}$/s]</td>
</tr>
<tr>
<td>$k_v$</td>
<td>Abrasion coefficient</td>
<td>[-]</td>
</tr>
<tr>
<td>$k_Y$</td>
<td>Dimensional coefficient</td>
<td>[1/Pa]</td>
</tr>
<tr>
<td>$K_b$</td>
<td>Bedload coefficient</td>
<td>[-/kg]</td>
</tr>
<tr>
<td>$l_{ch}$</td>
<td>Characteristic length</td>
<td>[N/m]</td>
</tr>
<tr>
<td>$l_w$</td>
<td>Wave length</td>
<td>[m]</td>
</tr>
<tr>
<td>$L$</td>
<td>Length</td>
<td>[m]</td>
</tr>
<tr>
<td>$L_p$</td>
<td>Particle hop length</td>
<td>[m]</td>
</tr>
<tr>
<td>$m$</td>
<td>Mass</td>
<td>[kg]</td>
</tr>
<tr>
<td>MAR</td>
<td>Mean annual runoff</td>
<td>[m$^3$/yr]</td>
</tr>
<tr>
<td>$n$</td>
<td>Number of time steps</td>
<td>[-]</td>
</tr>
<tr>
<td>$n_1$</td>
<td>Auxiliary parameter</td>
<td>[9.81 N/m$^2$]</td>
</tr>
<tr>
<td>$n_i$</td>
<td>Particle number per unit volume</td>
<td>[1/m$^3$]</td>
</tr>
<tr>
<td>$N_i$</td>
<td>$= L/L_p = \text{number of impact}$</td>
<td>[-]</td>
</tr>
<tr>
<td>NPV</td>
<td>Net present value</td>
<td>[CHF]</td>
</tr>
<tr>
<td>$p_i$</td>
<td>Gravimetric portion of a grain size class</td>
<td>[-]</td>
</tr>
<tr>
<td>$P_R$</td>
<td>Rolling probability</td>
<td>[-]</td>
</tr>
<tr>
<td>$P_{Rouse}$</td>
<td>$= V_s/(\kappa U_*)$ Rouse number</td>
<td>[-]</td>
</tr>
<tr>
<td>$P_{su}$</td>
<td>Suspension probability</td>
<td>[-]</td>
</tr>
<tr>
<td>$q$</td>
<td>Specific discharge</td>
<td>[m$^3$/(s·m)]</td>
</tr>
<tr>
<td>$q_s$</td>
<td>Specific gravimetric bedload transport</td>
<td>[kg/(s·m)]</td>
</tr>
<tr>
<td>Notation</td>
<td>Description</td>
<td>Units</td>
</tr>
<tr>
<td>----------</td>
<td>-------------</td>
<td>-------</td>
</tr>
<tr>
<td>$q_s^*$</td>
<td>Specific gravimetric bedload transport capacity</td>
<td>[kg/(s⋅m)]</td>
</tr>
<tr>
<td>$q_{ss}^*$</td>
<td>Specific gravimetric suspended sediment transport capacity</td>
<td>[kg/(s⋅m)]</td>
</tr>
<tr>
<td>$q_v$</td>
<td>Specific volumetric bedload transport</td>
<td>[m³/(s⋅m)]</td>
</tr>
<tr>
<td>$q_v^*$</td>
<td>Specific volumetric bedload transport capacity</td>
<td>[m³/(s⋅m)]</td>
</tr>
<tr>
<td>$q_{vtn}^*$</td>
<td>Non-dimensional volumetric bedload transport capacity</td>
<td>[-]</td>
</tr>
<tr>
<td>$q_{vt}$</td>
<td>Specific volumetric total load</td>
<td>[kg/(s⋅m)]</td>
</tr>
<tr>
<td>$Q$</td>
<td>Water discharge</td>
<td>[m³/s]</td>
</tr>
<tr>
<td>$\bar{Q}$</td>
<td>Temporal averaged discharge</td>
<td>[m³/s]</td>
</tr>
<tr>
<td>$Q_0$</td>
<td>Approach flow discharge</td>
<td>[m³/s]</td>
</tr>
<tr>
<td>$\bar{Q}_0$</td>
<td>Averaged approach flow discharge</td>
<td>[m³/s]</td>
</tr>
<tr>
<td>$Q_{BL22}$</td>
<td>Transport rate of bedload with $d \geq 22$ mm</td>
<td>[kg/s]</td>
</tr>
<tr>
<td>$Q_d$</td>
<td>Design discharge</td>
<td>[m³/s]</td>
</tr>
<tr>
<td>$Q_s$</td>
<td>Gravimetric bedload transport</td>
<td>[kg/s]</td>
</tr>
<tr>
<td>$Q_{SBT}$</td>
<td>Discharge in the SBT</td>
<td>[m³/s]</td>
</tr>
<tr>
<td>$\bar{Q}_{SBT}$</td>
<td>Averaged discharge in the SBT</td>
<td>[m³/s]</td>
</tr>
<tr>
<td>$Q_{SSL_{fine}}$</td>
<td>Transport rate of fine suspended sediment $d \leq 0.5$ mm</td>
<td>[kg/s]</td>
</tr>
<tr>
<td>$Q_{SSL_{coarse}}$</td>
<td>Transport rate of coarse suspended sediment $d = 0.5-22$ mm</td>
<td>[kg/s]</td>
</tr>
<tr>
<td>$Q_{TL}$</td>
<td>Total sediment transport rate</td>
<td>[kg/s]</td>
</tr>
<tr>
<td>$Q_v$</td>
<td>Volumetric bedload transport</td>
<td>[m³/s]</td>
</tr>
<tr>
<td>$r$</td>
<td>Interest rate</td>
<td>[-]</td>
</tr>
<tr>
<td>$R$</td>
<td>Radius</td>
<td>[m]</td>
</tr>
<tr>
<td>$R$</td>
<td>Reynolds number ($R = 4UR_p/\nu$)</td>
<td>[-]</td>
</tr>
<tr>
<td>$R^2$</td>
<td>Correlation coefficient</td>
<td>[-]</td>
</tr>
<tr>
<td>$R_h$</td>
<td>Hydraulic radius</td>
<td>[m]</td>
</tr>
<tr>
<td>$R_p$</td>
<td>Particle Reynolds number ($= U_\ast D/\nu$)</td>
<td>[-]</td>
</tr>
<tr>
<td>$R_S$</td>
<td>Hydraulic radius relative to bed</td>
<td>[m]</td>
</tr>
<tr>
<td>$RWL$</td>
<td>Reservoir water level</td>
<td>[masl]</td>
</tr>
<tr>
<td>Notation</td>
<td>Description</td>
<td></td>
</tr>
<tr>
<td>----------</td>
<td>-------------</td>
<td></td>
</tr>
<tr>
<td>$s$</td>
<td>Ratio of solid to fluid density [-]</td>
<td></td>
</tr>
<tr>
<td>$ssc$</td>
<td>Local suspended sediment concentration [kg/m$^3$]</td>
<td></td>
</tr>
<tr>
<td>$ssc_a$</td>
<td>Reference $ssc$ at the reference level $z = z_a$ [kg/m$^3$]</td>
<td></td>
</tr>
<tr>
<td>$S$</td>
<td>Slope [%]</td>
<td></td>
</tr>
<tr>
<td>$S'$</td>
<td>Reduced slope after Rickenmann (2005) [%]</td>
<td></td>
</tr>
<tr>
<td>$S_e$</td>
<td>Energy slope [%]</td>
<td></td>
</tr>
<tr>
<td>$SSC$</td>
<td>Suspended sediment concentration [kg/m$^3$] = [1000 mg/l]</td>
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</tr>
<tr>
<td>$SSL$</td>
<td>Suspended sediment load [to]</td>
<td></td>
</tr>
<tr>
<td>$SSR$</td>
<td>Transport rate of suspended sediment [kg/s]</td>
<td></td>
</tr>
<tr>
<td>$t$</td>
<td>time [s]</td>
<td></td>
</tr>
<tr>
<td>$T$</td>
<td>Bedload sampling duration [s]</td>
<td></td>
</tr>
<tr>
<td>$T^*$</td>
<td>Transport stage ($= (U/U_{*c})^2 = \theta/\theta_c$) [-]</td>
<td></td>
</tr>
<tr>
<td>$TE$</td>
<td>Trap efficiency [-]</td>
<td></td>
</tr>
<tr>
<td>$TL$</td>
<td>Total load [m$^3$]</td>
<td></td>
</tr>
<tr>
<td>$Tr$</td>
<td>Turbidity [FNU] or [NTU])</td>
<td></td>
</tr>
<tr>
<td>$\Delta t_i$</td>
<td>Impulse duration [s]</td>
<td></td>
</tr>
<tr>
<td>$u$</td>
<td>Streamwise flow velocity [m/s]</td>
<td></td>
</tr>
<tr>
<td>$U$</td>
<td>Flow velocity [m/s]</td>
<td></td>
</tr>
<tr>
<td>$U^+$</td>
<td>$= U/U_*$ [-]</td>
<td></td>
</tr>
<tr>
<td>$\Delta U^+$</td>
<td>roughness shift [-]</td>
<td></td>
</tr>
<tr>
<td>$U_*$</td>
<td>Friction velocity [m/s]</td>
<td></td>
</tr>
<tr>
<td>$U_{*c}$</td>
<td>Critical friction velocity at onset of motion [m/s]</td>
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</tr>
<tr>
<td>$U_p$</td>
<td>Horizontal particle velocity [m/s]</td>
<td></td>
</tr>
<tr>
<td>$V$</td>
<td>Volume [m$^3$]</td>
<td></td>
</tr>
<tr>
<td>$V_p$</td>
<td>Particle velocity [m/s]</td>
<td></td>
</tr>
<tr>
<td>$V_s$</td>
<td>Particle fall velocity in still water [m/s]</td>
<td></td>
</tr>
<tr>
<td>$V_{ts}$</td>
<td>Volume of transported sediment [m$^3$]</td>
<td></td>
</tr>
</tbody>
</table>
Notation 233

\( w/c \) Gravimetric water-cement ratio \([-]\)

\( W_f \) Total friction work \([\text{J}]\)

\( W_{im} \) Mean vertical particle impact velocity \([\text{m/s}]\)

\( W_p \) Vertical particle velocity \([\text{m/s}]\)

\( x \) Streamwise coordinate/ variable \([-] / [\text{unit}]\)

\( \bar{x} \) Expected value /mean of the variable \( x_i \) \([\text{unit}]\)

\( X \) Indirectly determined value \([\text{unit}]\)

\( y \) Spanwise coordinate \([-]\)

\( Y_M \) Elastic Young’s modulus \([\text{Pa}]\)

\( z \) Height above bottom or vertical coordinate \([\text{m}] / [-]\)

\( z^+ = z \cdot U^*/\nu \) \([-]\)

\( z_0 \) Zero velocity level \([\text{m}]\)

\( z_a \) Elevation of border between bedload and suspended transport layer above bed \([\text{m}]\)

\( z_p = \sum \Delta t_i / T \) Impact overlap indicator \([-]\)

13.2 Greek symbols

\( \alpha \) Angle \([-]\)

\( \beta \) Diffusion coefficient \([-]\)

\( \gamma_i \) Particle impact angle \([-]\)

\( \varepsilon \) Strain \([-]\)

\( \theta \) Shields parameter / non-dimensional shear stress \([-]\)

\( \theta_c \) Critical Shields parameter / non-dimensional critical shear stress \([-]\)

\( \theta_c' \) Critical Shields parameter of the armor layer \([-]\)

\( \theta_s \) Slope-corrected Shields parameter \([-]\)

\( \theta_{su} \) Critical Shields parameter for suction removal of sediment between blocks \([-]\)

\( \kappa \) von-Karman constant (= 0.41) \([-]\)
<table>
<thead>
<tr>
<th>Sign</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>λ</td>
<td>Flow resistance coefficient</td>
<td>[-]</td>
</tr>
<tr>
<td>μ</td>
<td>Poisson’s ratio</td>
<td>[-]</td>
</tr>
<tr>
<td>ν</td>
<td>Kinematic viscosity (water at 20°C: $10^{-6}$ m²/s)</td>
<td>[m²/s]</td>
</tr>
<tr>
<td>Π</td>
<td>Wake parameter</td>
<td>[-]</td>
</tr>
<tr>
<td>ρ</td>
<td>Water density</td>
<td>[kg/m³]</td>
</tr>
<tr>
<td>ρᵝ</td>
<td>Concrete / invert material density</td>
<td>[kg/m³]</td>
</tr>
<tr>
<td>ρₛ</td>
<td>Solid particle density (typically 2650 kg/m³)</td>
<td>[kg/m³]</td>
</tr>
<tr>
<td>σ</td>
<td>Particle uniformity coefficient / standard deviation</td>
<td>[-] / [unity]</td>
</tr>
<tr>
<td>σᵣ</td>
<td>Relative standard deviation</td>
<td>[-]</td>
</tr>
<tr>
<td>τ</td>
<td>Shear stress</td>
<td>[Pa]</td>
</tr>
<tr>
<td>τᵦ</td>
<td>Bed shear stress (= $ρgR_rS$)</td>
<td>[Pa]</td>
</tr>
<tr>
<td>φ</td>
<td>Angle of repose</td>
<td>[°]</td>
</tr>
<tr>
<td>χ</td>
<td>Velocity correction factor</td>
<td>[-]</td>
</tr>
</tbody>
</table>

13.3 Subscripts

- **a**  Armor layer, bedload layer
- **b**  Bed
- **BL22**  Bedload transport with $d \geq 22$ mm
- **bt**  Bending tensile
- **ch**  Characteristic
- **c**  Critical, compressive
- **cyl**  Cylindrical
- **d**  Design
- **e**  Energy line
- **est**  Estimated
- **field**  Value obtained from field
\( h \) Hydrualic
\( i \) Running index
\( im \) Impact
\( in \) Inflow
\( lab \) Value obtained from laboratory
\( m \) Mean
\( max \) Maximum
\( meas \) Measured
\( min \) Minimum
\( n \) Non-dimensional
\( op \) Operational
\( out \) Outlet/ outflow
\( s \) Gravimetric
\( SBT \) Sediment bypass tunnel
\( SSL_{coarse} \) Coarse suspended sediment \((d = 0.5 - 22 \text{ mm})\)
\( SSL_{fine} \) Fine suspended sediment \((d = 0 - 0.5 \text{ mm})\)
\( st \) Splitting tensile
\( t \) Uniaxial tensile, total
\( TL \) Total sediment load including suspend sediment and bedload
\( p \) Particle
\( R \) Rolling
\( Reservoir \) Reservoir
\( u \) Uniform
\( v \) Volumetric
\( w \) Wave
\( ws \) Water surface
13.4 Abbreviations

AC Amplitude class (method)
C1, C2 High-strength Concrete test field installed at test site 1 and 2, respectively
FOEN Federal Office for the Environment FOEN
G1, G2 Granite test field installed at test site 1 and 2, respectively
GSD Grain size distribution
HAC High Alumina Cement concrete
HPC High Performance Concrete
HSC High-Strength Concrete
LSC Low Shrinkage Concrete
MDEM Modified discrete element method
NC Normal Concrete
PC Polymer Concrete
RCC Roller Compacted Concrete
SAM Saltation Abrasion Model
SAM Saltation Abrasion Model adapted by Auel
SBT Sediment Bypass Tunnel
SC Silica fume concrete
SF Steel Fiber concrete
SPGS Swiss Plate Geophone System
TAM Total Abrasion Model
TUD Technical University Dresden (Institute of Construction Materials)
UHPC Ultra-High Performance reinforced Concrete
WSL Swiss Federal Institute for Forest, Snow and Landscape Research WSL
A Overview reservoir sedimentation countermeasures

Figure A.1 Overview of reservoir sedimentation countermeasures (adopted from Sumi et al. (2004b) and Boillat et al. (2003))
### B Existing SBTs: Plan view and cross section

<table>
<thead>
<tr>
<th>Location</th>
<th>Map with River Access</th>
<th>Cross-Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asahi</td>
<td><img src="image" alt="Asahi Map" /></td>
<td><img src="image" alt="Asahi Cross-Section" /></td>
</tr>
<tr>
<td>Koshibu</td>
<td><img src="image" alt="Koshibu Map" /></td>
<td><img src="image" alt="Koshibu Cross-Section" /></td>
</tr>
<tr>
<td>Matsukawa</td>
<td><img src="image" alt="Matsukawa Map" /></td>
<td><img src="image" alt="Matsukawa Cross-Section" /></td>
</tr>
<tr>
<td>Miwa</td>
<td><img src="image" alt="Miwa Map" /></td>
<td><img src="image" alt="Miwa Cross-Section" /></td>
</tr>
<tr>
<td>Nunobiki</td>
<td><img src="image" alt="Nunobiki Map" /></td>
<td><img src="image" alt="Nunobiki Cross-Section" /></td>
</tr>
<tr>
<td>Mud Mountain</td>
<td><img src="image" alt="Mud Mountain Map" /></td>
<td><img src="image" alt="Mud Mountain Cross-Section" /></td>
</tr>
<tr>
<td>Nanhua</td>
<td><img src="image" alt="Nanhua Map" /></td>
<td><img src="image" alt="Nanhua Cross-Section" /></td>
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<tr>
<td>Patrind</td>
<td><img src="image" alt="Patrind Map" /></td>
<td><img src="image" alt="Patrind Cross-Section" /></td>
</tr>
</tbody>
</table>


C Uncertainties and sensitivity

All measurements are flawed by stochastic errors. The errors can be declared as error margins or standard deviations $\sigma$ of the variable $x$. The standard deviation determines the mean error of $n$ measurement points from the expected value $\bar{x}$ and is defined as:

$$\sigma = \sqrt{\frac{1}{n} \sum_{i=1}^{n} (x_i - \bar{x})^2} \quad (C.1)$$

Low values indicate that the data points are close to the expected value of the data set. By normalizing with $\bar{x}$ the relative standard error follows:

$$\sigma_r = \frac{\sigma}{\bar{x}} \quad (C.2)$$

The error of an indirectly determined value $X$, which depends on several independent variables $x_i$ with the exponents $f_i$ and coefficients $c_i$ follows (error propagation):

$$\sigma_{r,X} = \sqrt{\sum_{i}^{} (f_i \cdot \sigma_{r,i})^2} \quad \text{for} \quad X = x_1^{f_1} \cdot x_2^{f_2} \cdot \ldots \cdot x_n^{f_n}$$

$$\sigma_X = \sqrt{\sum_{i}^{} (c_i \cdot \sigma_{i})^2} \quad \text{for} \quad X = c_1 \cdot x_1 + c_2 \cdot x_2 + \ldots + c_n \cdot x_n \quad (C.3)$$

In case of more complex relationships with differently nested arithmetic operations, the error margin of the result can also be computed by setting all variables to their possible minimum and maximum, respectively, to compute the minimum and maximum deviation of the expected value. This method was applied for bedload calculations.

Furthermore the sensitivity of a result to the input variables can be investigated by keeping all variables constant but varying the interested input parameter.
D Total load abrasion model of Lamb

Lamb et al. (2008) extended the SAM to include not only the impacts of bedload but also turbulence-driven impacts of suspended load. The total load abrasion model (TAM) follows:

\[ A_1 = A_1 \rho \frac{Y_M}{k_f} \frac{q_{vt}W_{im}^3}{U_h \chi + U_p H_p} \left(1 - \frac{q_v}{q_v^*}\right) [m/s] \]  

(D.1)

where \( A_1 < 1 \) accounts for the lift forces acting on the particles near the bed and \( q_{vt} = \) total volumetric sediment load. \( 0 \leq \chi \leq 1 \) is an integral coefficient, describing the vertical distribution of velocity and sediment concentration.

While the SAM predicts accurate erosion rates for coarse grains, it does not fit for small grains, which are in suspension and do not contribute to abrasion in this model (Figure D.1). In contrast, the TAM matches well with experimental data even for smaller grains if viscous damping effects, reducing the impact energy of small grains, is accounted for. However, with increasing grain size, the contribution of the small particles to the abrasion depth becomes smaller. In the bedload regime its portion is orders of magnitudes less than of larger particles and becomes insignificant. Therefore, both SAM and TAM lead to similar results within this regime. Hence, for practical use with dominating bedload transport and minor suspended sediment, the SAM is appropriate.

Figure D.1  Erosion rate depending on particle size for the total load abrasion model, the saltation abrasion model and for the abrasion mill data gained by Sklar and Dietrich (2001) and Scheingross et al. (2014) (after Lamb et al. (2015))
E Solis SBT

E.1 River topography

![Figure E.1 Overview of the analyzed river stretch (map: courtesy of map.geo.admin.ch)](image)

Table E.1 Cross section parameters of the Albula upstream of the Solis Reservoir

<table>
<thead>
<tr>
<th>Profile</th>
<th>Distance from the reservoir head [m]</th>
<th>Channel width [m]</th>
<th>Mean elevation [m.s.l.]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>14</td>
<td>823.0</td>
</tr>
<tr>
<td>2</td>
<td>140</td>
<td>13</td>
<td>823.8</td>
</tr>
<tr>
<td>3</td>
<td>435</td>
<td>18</td>
<td>825.7</td>
</tr>
<tr>
<td>4</td>
<td>745</td>
<td>20.5</td>
<td>830.2</td>
</tr>
<tr>
<td>5</td>
<td>960</td>
<td>13.5</td>
<td>833.5</td>
</tr>
<tr>
<td>6</td>
<td>1150</td>
<td>11</td>
<td>839.5</td>
</tr>
<tr>
<td>7</td>
<td>1350</td>
<td>15.5</td>
<td>842.3</td>
</tr>
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<td>8</td>
<td>1575</td>
<td>11.5</td>
<td>845.3</td>
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<tr>
<td>9</td>
<td>1810</td>
<td>16.5</td>
<td>849.8</td>
</tr>
<tr>
<td>10</td>
<td>2075</td>
<td>15.5</td>
<td>852.7</td>
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<tr>
<td>11</td>
<td>2335</td>
<td>10.5</td>
<td>856.9</td>
</tr>
<tr>
<td>12</td>
<td>2760</td>
<td>8.5</td>
<td>866.8</td>
</tr>
</tbody>
</table>
E.2 Invert material test samples

Table E.2 Material properties and geometries of the test fields at the Solis

<table>
<thead>
<tr>
<th>Material</th>
<th>Samples</th>
<th>Dimension(s) [cm]</th>
<th>Strengths [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>NC</td>
<td>Cubes</td>
<td>15 × 15 × 15</td>
<td>$f_c = 105 \pm 1.8$</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>-</td>
<td>$f_{bt} = 11.5 \pm 0.5$</td>
</tr>
<tr>
<td>HSC</td>
<td>Cubes</td>
<td>15 × 15 × 15</td>
<td>$f_c = 78.9 \pm 2.6$</td>
</tr>
<tr>
<td></td>
<td>Beams</td>
<td>36 × 12 × 12</td>
<td>$f_{bt} = 12.4 \pm 0.5$</td>
</tr>
<tr>
<td>LSC</td>
<td>Cubes</td>
<td>15 × 15 × 15</td>
<td>$f_c = 84.7 \pm 2.2$</td>
</tr>
<tr>
<td></td>
<td>Beams</td>
<td>36 × 12 × 12</td>
<td>$f_{bt} = 10.8 \pm 0.6$</td>
</tr>
<tr>
<td>HAC</td>
<td>Cubes</td>
<td>15 × 15 × 15</td>
<td>$f_c = 86.3 \pm 3.4$</td>
</tr>
<tr>
<td></td>
<td>Beams</td>
<td>36 × 12 × 12</td>
<td>$f_{bt} = 11.5 \pm 0.8$</td>
</tr>
<tr>
<td>UHPC</td>
<td>Cubes</td>
<td>4 × 4 × 4</td>
<td>$f_c = 187 \pm 11$</td>
</tr>
<tr>
<td></td>
<td>Beams</td>
<td>16 × 4 × 4</td>
<td>$f_{bt} = 20.9 \pm 1.1$</td>
</tr>
</tbody>
</table>

E.3 Effect of SPGS impulse threshold on determined sediment transport

The bedload transport in the Solis SBT was measured by using a SPGS. To filter out background noise, a threshold signal value of 0.1 V is commonly used and also applied in this study (Rickenmann et al. 2013, Wyss 2015, Chiari et al. 2016). The corresponding detection particle size determined in laboratory is 22 mm (Chapter 5.1.1). However, the background noise of the SPGS in the Solis SBT was significantly below this value. A reduction of the threshold signal value decreases the detection particle size and enables to detect a larger particle size range. As a result, the uncertainty of the sediment transport estimation described in Chapters 4.3.5 - 4.3.10 is expected to reduce. Therefore, the effect of the threshold value on the determined bedload and on the total sediment transport was investigated varying the detection threshold between 0.05 V and 0.1 V. For the lower threshold, the detection particle size is reduced to $d = 16$ mm according to Wyss’ (2016) fit shown in Figure 5.11.

Figure E.2a shows the calibration coefficient $K_b$ as a function of particle size for the field calibration tests for both thresholds, i.e. 0.1 V and 0.05 V. Reducing the impulse threshold from 0.1 V to 0.05 V results in a slight increase of $K_b$, indicating that only a small amount of particles with $d = 16$ - 22 mm was detected. According to Abbott and Francis (1977) over 90% of these particles are transported in suspension, rarely hitting the geophone and hence confirm this result.
By applying the method described in Chapter 4.3.7, with the adapted parameters $c_5 = 15000$, $c_6 = 0.051$, $c_7 = 1.80$ and $c_8 = 48$ in Equation (5.3) and respecting the effect of signal saturation by applying Equation (5.4) the bedload transport in the Solis SBT was computed for the operation on August 13, 2014. The corresponding masses of fine and coarse suspended sediment were determined as explained in the Chapters 4.3.9 and 4.3.10. The obtained GSD using both threshold signal values 0.05 V and 0.1 V are shown in Figure E.2b. The GSDs are similar, but the one with the lower impact threshold exhibits an additional data point due to the reduced detection particle size. During the operation on August 13, 2014 the total estimated sediment mass was 1600 to. With the threshold value of 0.05 V, an insignificantly lower mass of 1590 to was obtained. This result indicates that a reduction of the threshold value by 50% has no significant effect on both the GSD and the total sediment mass for the operation on August 13, 2014 despite the significantly reduced detection particle size. Therefore, the commonly used impact threshold of 0.1 V was applied for the sake of comparability with other SPGS measurements.

![Figure E.2](image.png)

Figure E.2  a) Calibration coefficient $K_b$ as a function of particle size using two different impulse threshold values, and b) the resulting GSD of the sediment transport in the Solis SBT during the operation on August 13, 2014 using two different impulse threshold values
F Pfaffensprung SBT

F.1 River topography

Figure F.1 Overview of the analyzed river stretch (map: courtesy of map.geo.admin.ch)
Table F.1  Cross section parameters of the Reuss upstream of the Pfaffensprung SBT

<table>
<thead>
<tr>
<th>Profile</th>
<th>Distance from the reservoir head [m]</th>
<th>Channel width [m]</th>
<th>Mean elevation [masl]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>200</td>
<td>24</td>
<td>812.03</td>
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<tr>
<td>2</td>
<td>325</td>
<td>15</td>
<td>816.67</td>
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<tr>
<td>3</td>
<td>468</td>
<td>20</td>
<td>822.52</td>
</tr>
<tr>
<td>4</td>
<td>708</td>
<td>18.5</td>
<td>833.75</td>
</tr>
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<td>5</td>
<td>890</td>
<td>22</td>
<td>839.46</td>
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<td>6</td>
<td>1010</td>
<td>19</td>
<td>843.54</td>
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<td>7</td>
<td>1180</td>
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<td>850.53</td>
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<td>18.5</td>
<td>861.23</td>
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<td>9</td>
<td>1920</td>
<td>17.5</td>
<td>878.90</td>
</tr>
<tr>
<td>10</td>
<td>2400</td>
<td>17</td>
<td>894.12</td>
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<tr>
<td>11</td>
<td>2480</td>
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<tr>
<td>12</td>
<td>2670</td>
<td>15</td>
<td>907.25</td>
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<tr>
<td>13</td>
<td>2880</td>
<td>14.5</td>
<td>914.59</td>
</tr>
</tbody>
</table>

F.2  GSD analysis using BASEGRAIN

Wolgemuth (2015) investigated the grain size distribution near the intake of the SBT based on top-view photographs taken with an ultra-light camera (GoPro Hero3+ Black Edition) mounted on an unmanned aerial vehicle (Quadrocopter DJI Phantom FC40). In an altitude of 8 m several high-resolution images were taken from the river bed (Figure F.2a). These pictures were matched and combined by means of an overview picture taken at an altitude of 33 m above ground exhibiting a number of feature points (Figure F.2b). It should be noted that all pictures were lens-corrected, georeferenced and orthorectified with scaling error of less than ±2% (Detert and Weitbrecht 2015). The GSD of the bed material was determined based on 12 different high-resolution images using the BASEGRAIN software, an automatic object detection software tool for granulometric analysis of fluvial gravel beds (Detert and Weitbrecht 2013). The GSD was computed following Fehr’s (1987) line-sampling method assuming that 15% of the particles were finer than 40 mm. The obtained GSD is shown in Figure F.3. The error of the GSD estimation is assumed to be less than 20% (Detert 2015).
Figure F.2  a) High-resolution image and b) overview image of the Reuss bed with georeferenced and feature points (modified from Wolgemuth (2015))

<table>
<thead>
<tr>
<th>$d$</th>
<th>[mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>$d_{max}$</td>
<td>1280</td>
</tr>
<tr>
<td>$d_{90}$</td>
<td>606</td>
</tr>
<tr>
<td>$d_{84}$</td>
<td>481</td>
</tr>
<tr>
<td>$d_{m}$</td>
<td>250</td>
</tr>
<tr>
<td>$d_{50}$</td>
<td>174</td>
</tr>
<tr>
<td>$d_{30}$</td>
<td>76</td>
</tr>
<tr>
<td>$d_{16}$</td>
<td>20</td>
</tr>
</tbody>
</table>

Figure F.3  GSD of the Reuss bed material based on line samples and BASEGRAIN analysis; modified from Wolgemuth (2015)
G Runcahez SBT

G.1 River topography

Figure G.1 Overview of the analyzed river stretch (map: courtesy of map.geo.admin.ch)
Table G.1  Cross section parameters of the Rein da Sumvitg upstream of the Runcahez SBT

<table>
<thead>
<tr>
<th>Profile</th>
<th>Distance from the reservoir head [m]</th>
<th>Channel width [m]</th>
<th>Mean elevation [masl]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>75</td>
<td>14.7</td>
<td>1277.35</td>
</tr>
<tr>
<td>2</td>
<td>350</td>
<td>17.3</td>
<td>1287.0</td>
</tr>
<tr>
<td>3</td>
<td>700</td>
<td>16.7</td>
<td>1299.85</td>
</tr>
<tr>
<td>4</td>
<td>905</td>
<td>12.5</td>
<td>1309.85</td>
</tr>
<tr>
<td>5</td>
<td>1130</td>
<td>13.1</td>
<td>1320.35</td>
</tr>
<tr>
<td>6</td>
<td>1440</td>
<td>14.2</td>
<td>1331.65</td>
</tr>
<tr>
<td>7</td>
<td>1720</td>
<td>14.6</td>
<td>1338.45</td>
</tr>
<tr>
<td>8</td>
<td>1980</td>
<td>17.0</td>
<td>1349.35</td>
</tr>
<tr>
<td>9</td>
<td>2380</td>
<td>15.8</td>
<td>1363.7</td>
</tr>
<tr>
<td>10</td>
<td>2620</td>
<td>11.3</td>
<td>1372.95</td>
</tr>
</tbody>
</table>

G.2 Hydroabrasion of the test fields

Figure G.2  Abrasion depths of the RCC test field in 1999 after four years of operation
Figure G.3 Abrasion depths of the HPC test field a) in 1999 after four years of operation and b) in 2014 after 19 years of operation

Figure G.4 Abrasion depths of the SF test field a) in 1999 after four years of operation and b) in 2014 after 19 years of operation
Figure G.5  Abrasion depths of the PC test field a) in 1999 after four years of operation and b) in 2014 after 19 years of operation
H Abrasion model calibration

H.1 Pfaffensprung SBT

Table H.1  Operating conditions, estimated bypassed sediment masses and spatially averaged abrasion depths of the test fields in the Pfaffensprung SBT

<table>
<thead>
<tr>
<th>Year</th>
<th>$t$ [d]</th>
<th>$BL$ [$10^3$ to]</th>
<th>$Q_m$ [m$^3$/s]</th>
<th>$U_m$ [m/s]</th>
<th>$a_m$ [mm/yr]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>C1</td>
</tr>
<tr>
<td>2012</td>
<td>91</td>
<td>460</td>
<td>35.5</td>
<td>9.8</td>
<td>15.4</td>
</tr>
<tr>
<td>2013</td>
<td>61</td>
<td>370</td>
<td>32.6</td>
<td>9.8</td>
<td>9.3</td>
</tr>
<tr>
<td>2014</td>
<td>55</td>
<td>140</td>
<td>25.9</td>
<td>9.7</td>
<td>1.3</td>
</tr>
<tr>
<td>2015</td>
<td>96</td>
<td>430</td>
<td>34.6</td>
<td>9.8</td>
<td>5.0</td>
</tr>
</tbody>
</table>

Table H.2  Abrasion coefficient $k_v$ determined based on the Pfaffensprung SBT data

| Year | $k_v$ [$10^6$] | C1 | C2 | G1 | G2 | C1 | C2 | G1 | G2 | C1 | C2 | G1 | G2 |
|------|----------------|----|----|----|----|----|----|----|----|----|----|----|----|----|
| 2012 | 1.14           |    | 7.82 | - |    | 0.88 | - | 9.23 | - | 0.14 | - | 1.45 | - |
| 2013 | 1.55 1.65      | 32.3 | 13.7 |    |    | 1.20 | 1.14 | 38.2 | 16.1 | 0.18 | 0.17 | 5.75 | 2.43 |
| 2014 | 4.93 4.43      | 17.9 | 15.1 |    |    | 3.80 | 3.07 | 21.1 | 17.9 | 0.51 | 0.41 | 2.84 | 2.40 |
| 2015 | 3.37           |    | 16.2 | - |    | 2.60 | - | 19.1 | - | 0.40 | - | 2.95 | - |
| Weighted average | 1.81 | 2.10 | 13.7 | 14.4 | 1.40 | 1.45 | 16.2 | 17.0 | 0.21 | 0.21 | 2.42 | 2.42 |
| Weighted average | 1.89 | 13.9 | 1.41 | 16.4 | 0.21 | 2.43 |
| Min / Max | 1.14 / 4.93 | 7.82 / 32.3 | 0.88 / 3.80 | 9.23 / 38.2 | 0.14 / 0.51 | 1.45 / 5.75 |
| Error $\sigma_r$ [%] | 111 | 107 |

Table H.3  Material parameter $c_1$ determined based on the Pfaffensprung SBT data

<table>
<thead>
<tr>
<th>Year</th>
<th>$c_1$ [$10^{-7}$ m$^2$/kg]</th>
<th>C1</th>
<th>C1</th>
<th>G1</th>
<th>G2</th>
</tr>
</thead>
<tbody>
<tr>
<td>2012</td>
<td>0.46</td>
<td>-</td>
<td>0.100</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>2013</td>
<td>0.36</td>
<td>0.33</td>
<td>0.026</td>
<td>0.061</td>
<td></td>
</tr>
<tr>
<td>2014</td>
<td>0.14</td>
<td>0.15</td>
<td>0.058</td>
<td>0.069</td>
<td></td>
</tr>
<tr>
<td>2015</td>
<td>0.16</td>
<td>-</td>
<td>0.049</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Weighted average</td>
<td>0.31</td>
<td>0.29</td>
<td>0.062</td>
<td>0.064</td>
<td></td>
</tr>
<tr>
<td>Weighted average</td>
<td>0.31</td>
<td>0.063</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Min / Max</td>
<td>0.14 / 0.46</td>
<td>0.026 / 0.100</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Error $\sigma_r$ [%]</td>
<td>146</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### H.2 Runcahez SBT

Table H.4 Operating conditions, estimated bypassed sediment masses and spatially averaged abrasion depths of the test fields in the Runcahez SBT

<table>
<thead>
<tr>
<th></th>
<th>t [d]</th>
<th>BL [10^4 to]</th>
<th>Qm [m^3/s]</th>
<th>U_m [m/s]</th>
<th>(a_m) [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>SC</td>
</tr>
<tr>
<td>1996-1999</td>
<td>6.5</td>
<td>40.4</td>
<td>56.4</td>
<td>7.4</td>
<td>6.5</td>
</tr>
<tr>
<td>1999-2014</td>
<td>21.9</td>
<td>160.1</td>
<td>55.7</td>
<td>7.4</td>
<td>10.3</td>
</tr>
</tbody>
</table>

Table H.5 Abrasion coefficient \(k_v\) determined based on the Runcahez SBT data

<table>
<thead>
<tr>
<th></th>
<th>SAM</th>
<th>SAM*</th>
<th>SAMA</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SC</td>
<td>RCC</td>
<td>HPC</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1996-1999</td>
<td>0.51</td>
<td>1.16</td>
<td>0.79</td>
</tr>
<tr>
<td>1999-2014</td>
<td>1.24</td>
<td>-</td>
<td>1.31</td>
</tr>
<tr>
<td>Weighted average</td>
<td>0.96</td>
<td>1.16+</td>
<td>1.15</td>
</tr>
<tr>
<td>Weighted average</td>
<td>0.97</td>
<td></td>
<td>1.02</td>
</tr>
<tr>
<td>Min / Max</td>
<td>0.51/ 1.31</td>
<td></td>
<td>0.55/ 1.38</td>
</tr>
<tr>
<td>Error (\sigma_r) [%]</td>
<td></td>
<td>111</td>
<td></td>
</tr>
</tbody>
</table>

+ determined based on 4 years data

Table H.6 Material parameter \(c_I\) determined based on the Runcahez SBT data

<table>
<thead>
<tr>
<th></th>
<th>SC</th>
<th>RCC</th>
<th>HPC</th>
<th>SF</th>
<th>PC</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1996-1999</td>
<td>2.46</td>
<td>2.03</td>
<td>2.24</td>
<td>1.45</td>
<td>0.34</td>
</tr>
<tr>
<td>1999-2014</td>
<td>0.98</td>
<td>-</td>
<td>1.33</td>
<td>1.70</td>
<td>1.48</td>
</tr>
<tr>
<td>Weighted average</td>
<td>1.28</td>
<td>2.03</td>
<td>1.51</td>
<td>1.65</td>
<td>1.25</td>
</tr>
<tr>
<td>Weighted average</td>
<td>1.45</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Min / Max</td>
<td></td>
<td>0.34/ 2.46</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Error (\sigma_r) [%]</td>
<td></td>
<td>146</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
H.3 Other SBTs

Table H.7  Operating conditions, estimated bypassed sediment masses and spatially averaged abrasion depths of different SBTs,

<table>
<thead>
<tr>
<th></th>
<th>$t$ [d/yr]</th>
<th>$BL$ [10$^3$ to/yr]</th>
<th>$Q_m$ [m$^3$/s]</th>
<th>$U_m$ [m/s]</th>
<th>$a_m$ [mm/yr]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Egschi</td>
<td>10</td>
<td>146*</td>
<td>25**</td>
<td>8.8</td>
<td>5</td>
</tr>
<tr>
<td>Palagnedra</td>
<td>5</td>
<td>212*</td>
<td>110**</td>
<td>8.5</td>
<td>1.75</td>
</tr>
<tr>
<td>Rempen</td>
<td>3</td>
<td>11</td>
<td>40**</td>
<td>12.5</td>
<td>1</td>
</tr>
<tr>
<td>Val d’Ambra</td>
<td>2.5</td>
<td>21</td>
<td>42.5**</td>
<td>8.3</td>
<td>3</td>
</tr>
<tr>
<td>Mud Mountain (SBT excl. inlet)</td>
<td>83</td>
<td>50</td>
<td>68</td>
<td>11.3</td>
<td>0.59</td>
</tr>
<tr>
<td>Mud Mountain (inlet bend)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.51</td>
</tr>
<tr>
<td>Mud Mountain (straight section)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.61</td>
</tr>
<tr>
<td>Mud Mountain (outlet bend)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.21</td>
</tr>
</tbody>
</table>

*assuming a bypass efficiency of 80%, **assumed to be half of the design discharge

Table H.8  Abrasion coefficient $k_v$ determined for different SBTs; considered data marked in gray

<table>
<thead>
<tr>
<th></th>
<th>$k_v$ [10$^6$]</th>
<th>SAM</th>
<th>SAM*</th>
<th>SAMA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Egschi (Granite)</td>
<td>1.91</td>
<td>1.91</td>
<td>0.69</td>
<td></td>
</tr>
<tr>
<td>Palagnedra (Concrete)</td>
<td>25.7</td>
<td>13.2</td>
<td>5.80</td>
<td></td>
</tr>
<tr>
<td>Rempen (Cast basalt)</td>
<td>0.032</td>
<td>0.072</td>
<td>0.042</td>
<td></td>
</tr>
<tr>
<td>Val d’Ambra (Concrete)</td>
<td>1.33</td>
<td>0.75</td>
<td>0.30</td>
<td></td>
</tr>
<tr>
<td>Mud Mountain (Steel, inlet bend)*</td>
<td>0.0072</td>
<td>0.030</td>
<td>0.0094</td>
<td></td>
</tr>
<tr>
<td>Mud Mountain (Steel, straight section)</td>
<td>0.0061</td>
<td>0.026</td>
<td>0.0079</td>
<td></td>
</tr>
<tr>
<td>Mud Mountain (Steel, outlet bend)</td>
<td>0.018</td>
<td>0.075</td>
<td>0.023</td>
<td></td>
</tr>
<tr>
<td>Error $\sigma_r$ [%]</td>
<td>111</td>
<td>107</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* abrasion partially broke through the steel lining

Table H.9  Material parameters $c_1$ and $c_2$ determined for different SBTs

<table>
<thead>
<tr>
<th></th>
<th>$c_1$ [10$^{-7}$ m$^2$/kg]</th>
<th>$c_2$ [10$^{-10}$ m$^2$/kg]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Egschi (Granite)</td>
<td>0.19</td>
<td></td>
</tr>
<tr>
<td>Palagnedra (Concrete)</td>
<td>0.08</td>
<td></td>
</tr>
<tr>
<td>Rempen (Cast basalt)</td>
<td>0.75</td>
<td></td>
</tr>
<tr>
<td>Val d’Ambra (Concrete)</td>
<td>1.79</td>
<td>7.3</td>
</tr>
<tr>
<td>Mud Mountain (Steel, inlet bend)*</td>
<td>7.3</td>
<td>5.3</td>
</tr>
<tr>
<td>Mud Mountain (Steel, straight section)</td>
<td>8.6</td>
<td>6.2</td>
</tr>
<tr>
<td>Mud Mountain (Steel, outlet bend)</td>
<td>3.0</td>
<td>2.9</td>
</tr>
<tr>
<td>Error $\sigma_r$ [%]</td>
<td>146</td>
<td>146</td>
</tr>
</tbody>
</table>

* abrasion partially broke through the steel lining


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