HYDRAULICS OF SPATIAL DIKE BREACHES

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PIERRE-JACQUES RAUL FRANK

Dipl.-Ing., Karlsruhe Institute of Technology

born on 11.08.1982

citizen of Luxembourg

accepted on the recommendation of

Prof. Dr. Robert M. Boes
Prof. Dr. Sandra Soares-Frazão

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Abstract

Dikes and dams are an essential part of modern infrastructure. Dikes along rivers protect the surrounding low lands, infrastructure and the population from floods, while earthen embankment dams impound reservoirs, e.g. for hydropower, irrigation, artificial snowing or flood retention. The large number of these infrastructures, coupled with increasing flood discharges in the last years amplifies the risk of a breach and potentially large damage. Recent dike breaches highlight the need for a better understanding of the dike breach process to quantify the resulting risks, conduct emergency planning and alert the population in case of impending danger. Although recent advances have been made in dike breach research, the hydraulics of dike breaching are still poorly understood.

The present research project investigates the spatial dike breach process due to overtopping for dikes of homogeneous non-cohesive sediment without a surface or core sealing. 80 plane and 35 spatial preliminary dike breach tests were conducted to optimize the test setup and the novel stereo-photogrammetric measurement system, which was applied to capture the 3D dike breach topography in the submerged breach. 45 systematic 3D dike breach tests were then conducted to test the modelling limitations, namely the (1) influence of seepage, (2) test repeatability, (3) symmetry of half-models, (4) Froude scalability, and to test the effect of the parameters (1) discharge, (2) sediment grain size, (3) pilot channel width, (4) dike cross-section, (5) mobile bed, (6) water surface topography, (7) reservoir volume and shape. The dike breach topography and the hydrographs were determined for the tests and combined for the subsequent data analysis. For tests with constant inflow discharge, the main governing parameters were identified as the critical flow depth, the dike shape and the reservoir water surface area, and to a lesser degree the sediment grain size and the dike height. The generalized main governing parameters were identified as the maximum headwater level, the reservoir water surface area, the dike cross-section and the inflow discharge. The final results include dimensionless relations to describe the breach process.

This research project essentially introduced four novelties to spatial dike breach research, namely (1) accurate submerged 3D breach topography mapping, (2) pump regulation to add a simulated volume to the physical reservoir volume, (3) proof of Froude similitude and (4) equation to predict peak breach discharge developed from laboratory model tests.
Kurzfassung


1 Introduction

1.1 Background and motivation

Dikes are heaped up embankments along a river or a seacoast to protect low-lying land against flooding. They are typically non-permanently impounded, homogeneous earthen structures, as opposed to dams, which are mostly permanently impounded and structured with impermeable elements as e.g. clay cores or surface sealings. Dike failures frequently occur due to a lack of maintenance or upgrading or when the design criteria are exceeded, as, e.g. when extreme floods lead to too high water levels and unplanned dike overtopping, resulting in dike breaches and potentially large damage to infrastructure. Recent floods resulting in dike failures include the River Elbe Floods (2002 and 2013), the New Orleans Flood (2005), the Mississippi Flood (2008), the breach of the Canal del Dique in Colombia (2010) or the controlled breach of a strategic dike in Punjab, Pakistan (2014).

In Switzerland, dikes and earthen embankment dam structures are a common part of modern infrastructure, as e.g. river dikes for flood protection, earth dams of reservoirs for hydropower, irrigation, or artificial snowing, road and railway embankments, but can also be created by natural disasters as e.g. landslide dams or moraine dams. With increasing density of these infrastructures, coupled with a growing population and extreme weather conditions attributed to climate change, the associated risks rise and need to be quantified and managed. In Switzerland, therefore, the OWARNA Project (Optimierung von Warnung und Alarmierung vor Naturgefahren) was launched by the Federal authorities for the optimization of alerting in the event of natural hazards. Furthermore, relevant research has been conducted in recent years to mitigate these risks.

Due to its considerable infrastructural damage potential, there is a need to understand the dike breach process in detail. This research project is a follow-up project of the previous laboratory research at VAW on plane dike breaches due to overtopping by Schmocker (2011), which is extended to spatial dike breaches in this research. Technical restrictions to study the 3D dike breach topography and limitations in hydraulic laboratory experimentation in previous research projects were solved to study the breach process and extract data essential e.g. for computational modeling of spatial dike breaches. Because most dike failures are caused by overtopping, this failure mode is investigated in detail in the current research project for homogeneous model embankments of non-cohesive sediment.
1.2 Goals

The overarching goal of the present study is to investigate the spatial dike breach process due to overtopping using hydraulic model tests. The main focus is on the following topics:

- Design a test setup to study dike breaches for a pure overtopping failure mode
- Improve the novel photogrammetric system to eliminate air/water refraction effects for 3D topography measurements in submerged breaches
- Combine breach topography and hydrograph data for breach process analysis
- Apply novel pump regulation to simulate large reservoirs in laboratory channel
- Validate the symmetry of half-breach models and Froude scalability
- Investigate the effect of specific parameters on the dike breach process
- Develop practical equations and guidelines for the engineering community

To achieve these goals, systematic test runs were conducted for various inflow discharges, reservoir sizes and shapes, dike cross-sections, dike heights, sediment grain sizes and pilot channel widths. All data were systematically analyzed and normalized including scale effects, dike breach profiles, breach discharges, headwater levels, eroded sediment volumes, transverse breach profiles, breach shapes and submerged breach widths. Geotechnical aspects were of inferior interest and importance since only non-cohesive sediment was employed, as described in the literature review. Seepage outflow at the downstream dike toe was avoided to focus on the pure overtopping erosion process.

1.3 Thesis outline

The present study is organized as follows. In Chapter 2, the literature review of Schmocker (2011) is extended, the state-of-the-art is updated and the focus is shifted to the essential findings in literature on the effect of different parameters on the breach process. The focus is on 2D and 3D dike breach modelling due to overtopping. The identified research gaps are summarized. Chapter 3 presents the test setup, the test procedure, the measuring devices with a focus on the photogrammetric system accuracy, and the test program. In Chapter 4 the experimental results concerning seepage, test repeatability, proof of half-model symmetry, and Froude scalability are presented, while in Chapter 5 the effects of the varied parameters on the breach topography, flow hydrographs and eroded sediment volume are studied. The results are presented in dimensionless form in Chapter 6 and a dimensional equation is deduced for peak breach discharge. The main results are summarized in Chapter 7.
2 Literature review

2.1 Introduction

This chapter summarizes the scientific research on laboratory dike breach model tests relevant for the present study. This research project is a follow-up project of the previous laboratory research at VAW on plane dike breaches due to overtopping presented in 2.2 and by Schmocker (2011), which was extended to spatial dike breaches in this research. Therefore, the literature review in this chapter is an extension of Schmocker (2011), with a focus on the spatial dike breach process due to overtopping for non-cohesive granular dikes mainly since 2012.

After a short historical background on dike construction and a list of publications dealing with the historical analyses of dike breaches, guidelines to dike construction and the numerical simulation of dike breaching, Schmocker (2011) discusses in the literature review: (1) Hydraulic model tests in the past; (2) Dike failure modes; (3) Overtopping erosion; (4) Breach process; (5) Breach outflow; (6) Measuring devices; (7) Scale effects, before (8) summarizing the research gaps and the purpose of his study.

The main topics discussed in the present study are: (2.2) Dike breaches in the Swiss context; (2.3) Common dike and dam structures with breach potential; (2.4) Hydraulic model tests since 2011; (2.5) General dike breach observations; (2.6) Hydraulic boundary conditions; (2.7) 3D dike breach topography measurement; (2.8) Relevant breach parameters; (2.9) Model scalability; (2.10) Research gaps and purpose of the present study.

Surveys on laboratory tests were provided by Morris (2009b), Schmocker (2011), Wu et al. (2011) and Al-Riffai (2014). From 2004 to 2009, the integrated FLOODsite project was launched with the objective to better assess and manage flood risks in Europe and was concluded with a state-of-the-art review on breach modelling (Morris 2009b). An international team from France, USA and UK/Ireland, supported by the Netherlands and Germany, compiled The International Levee Handbook (CIRIA 2013) which offers comprehensive guidance on the design, construction, maintenance and improvement of levees. Zhang et al. (2016) published updated databases with 1,443 reconstructed dam failures, 1,044 landslide dam failures, and 1,004 dike failures, along with statistical analyses, failure mechanisms, process modelling, determination of breach parameters and risk eval-
Wu et al. (2011) provide updated lists of ICOLD (1995) of breach models developed using different approaches: Laboratory breach tests, parametric embankment models, simplified physically-based models, and multidimensional physically-based models. Numerical studies were conducted e.g. by VAW (2011), Wu et al. (2012), Volz (2013), Zhang et al. (2013), van Damme (2014), Guan et al. (2014), van Emelen (2014), van Emelen et al. (2014, 2016), Liu et al. (2015), and Volz et al. (2016). Empirical and semi-theoretical numerical models have been developed or improved e.g. by Nakagawa et al. (2011, 2016), Castro-Orgaz and Hager (2013), Hakimzadeh et al. (2013), Mitzutani et al. (2013), Wu (2013), Chiganne et al. (2014), De Lorenzo and Macchione (2014), Michelazzo (2014), and Froehlich (2016a,b). Comparisons between models were conducted e.g. by Pierce et al. (2010), Peeters et al. (2011), Morris et al. (2013), van Emelen (2014), Wahl (2014) and Khodashenas (2016).

2.2 Dike breaches in the Swiss context

Dikes and earthen dam structures are essential structures on which the modern society heavily depends, but pose a potential risk to infrastructure and population in the case of a failure. Dike breaches can also be triggered by natural events such as landslides or the retreat of glaciers. In Switzerland, one major dam breach occurred in 1888 in Sonzier, Montreux, with 7 deaths reported (Attinger 1902). In 2005, severe floods almost resulted in overtopping of the existing levees at the Hagneck Canal (Schmocker et al. 2013) and at the Linth Canal. Large and small dike breaches have regularly occurred in Switzerland and have been described by Vischer (2003) and Minor and Hager (2004).

With increasing density of these infrastructures, coupled with a growing population and extreme weather conditions attributed to climate change, the associated risks rise and need to be quantified and managed. In Switzerland, therefore, the OWARNA project was launched by the federal authorities for the optimization of warning and alerting in the event of natural hazards. Furthermore, relevant research to determine the risks associated with different scenarios is funded by different state levels and research funds, as e.g. current work by VAW (2011), Schmocker et al. (2013), Volz (2013), Peter et al. (2014), Boes et al. (2015), Vonwiller et al. (2015) or Volz et al. (2016). Furthermore, diverse 2D dike breach tests were conducted at the VAW Laboratory: Lüthi (2005) conducted 35 tests with different inflow discharges $Q_0$, sediment grain sizes $d$, dike heights $w$, dike slopes $S$ and one Froude scale test series, and presented dimensionless relations, e.g. a dimensionless time $T_{2D}$; the results were presented in part by Hager and Unger (2006).
Schmocker (2011) conducted 68 tests to determine model limitations, test repeatability, Froude scalability, and the effect of $Q_o$, $d$, $w$ and $S$ on the breach process, and presented dimensionless relations, e.g. $T_{2D}$. Winz (2012) conducted 33 tests to determine the effect of grain size distribution, water content and compaction, and surface layers or cores on the breach process. Frank (2013) conducted 80 tests to determine test repeatability, the effect of the drainage position, drainage discharge capacity, inflow discharge, reservoir volume, sediment grain size, and grain size distribution, resulting in optimized test setups and test procedures; the results were partially presented by Schmocker et al. (2014). Müller et al. (2016) conducted 42 tests to determine test repeatability and the effect of $Q_o$, $S$, crest length and constant headwater levels on the breach process.

2.3 **Common dike and dam structures with breach potential**

Dike and earthen dam structures pose a potential risk to infrastructure and population in the case of a breach. Common dike structures with breach potential include:

- River and channel dikes for flood protection
- Earth dams of artificial reservoirs, e.g. for hydropower or pumped-storage plants
- Road and railway embankments
- Temporary dams in rivers during construction works
- Dams of ponds or lakes for irrigation, artificial snowing or aquaculture
- Tailing and slurry dams used by the mining industry
- Landslide dams
- Moraine dams and ice dams created when glaciers retreat

A breach example for a provisional dam is shown in Figure 2.1 at the construction site of a driftwood retention rake at Sihl River upstream of Zürich. The images were captured by the Office of Waste, Water, Energy and Air (AWEL) of the Canton of Zürich using a webcam showing the dike breach evolution on August 5, 2016. At 10:20, two gullies are visible where previously an excavator had passed, initiating the breach. At 11:10, the flood level has further risen, increasing the flow through the breach and eroding the downstream face of the dike. At 11:40, the breach has grown vertically, before widening horizontally at 12:10 and 13:10. At 15:10, the flood has receded, breach discharge has dropped and large blocks remain at the bottom of the breach channel. The damage was small in this case, as no construction equipment was destroyed during the flood.
Figure 2.1  Webcam images of temporal evolution of provisional construction dam breach in Sihl River on August 5, 2016. (→) flow direction in Sihl River, (→) flow direction in breach channel. (Courtesy of AWEL, Canton of Zürich)

Figure 2.2  Formation of moraine-dammed lake with (a) advancing glacier pushing sediment, (b) retreat of glacier due in times of warmer climate leaving rocky debris called moraine forming a dam, (c) rain and meltwater form a lake, (d) retreat of glacier and increase of lake size, (e) overflow over moraine-dam (Awal et al. 2010)
An overview of different types of obstructive natural dams and the breach thereof is given by Costa and Schuster (1988). Moraine-dam breaches are one example for dike breaches in the natural environment. Due to climate change and warmer temperatures on a global scale, glaciers are retreating, often leaving behind an end moraine, which consists of the material transported by the glacier front over large periods of time (Figure 2.2). These end moraines typically consist of non-uniform sediment depending on the landscape and can reach several tens of meters in height, as e.g. in Figure 2.3, showing a moraine-dam after the breach in Khumbu Himal, Nepal, with a breach 200 m wide and 60 m high (Vuichard and Zimmermann 1987). The breaching of the moraine was triggered by wave action following an ice avalanche of 150,000 m³ into the lake and resulted in an estimated peak discharge of 1,600 m³/s about 30 minutes after breach start. Further research on moraine dam breaches has been conducted e.g. by Awal et al. (2010).

![Figure 2.3](image)

Figure 2.3  V-shaped trench in Langmoche terminal moraine (height: 60 m; width: 200 m) on October 21, 1985 (Vuichard and Zimmermann 1987)

Landslides occur in regions with steep slopes and are frequently triggered by severe rains and high groundwater levels. In Switzerland, landslides often lead to a temporary blockage of roads or railways. When a landslide occurs in a river valley and impounds the river, the breach of this natural dam may lead to damage in the downstream reach, as described by Bezzola et al. (1996). As an example, a massive landslide in Utah in 1983 led to a river impoundment until the dam created by the toe of the landslide breached and destroyed the downstream village Thistle. The breach of landslide dams has been studied by Zhang et al. (2010), Capart (2013), Gordon et al. (2015), Xu et al. (2015), Chen et al. (2016), Shi et al. (2016) and Zhang et al. (2016). Dams of upper reservoirs of pumped-storage hydropower plants may breach when overtopped, as has occurred at the Taum Sauk pumped storage power plant in Reynold County, USA (Rogers and Watkins, 2008).
So-called erodible dikes, also termed fuse plugs, i.e. dikes that intentionally erode down to a certain level when overtopped, can be one of several appropriate measures to divert parts of a flood discharge into retention basins. Bühlmann and Boes (2014) describe the following measures for reducing flood discharge using lateral structures:

- Fixed crest dams without gates (unregulated)
- Tilting fuse plug elements (unregulated)
- Fuse plug embankments (unregulated)
- Fuse gate systems (unregulated)
- Gated structures (regulated)

Unregulated lateral inlet structures can be used for the retention of the peak flow in small and medium sized catchment areas, as well as emergency spillways. According to Bühlmann and Boes (2014), the advantages of erodible fuse plug embankments are: + less maintenance work, + later overtopped than fixed crest, + no recognizable artificial structure in landscape, while the disadvantage consist in the restoration work after a flood.

Such a lateral erodible dike 300 m long and 1.2 m high has been implemented on top of a fixed weir sill at the artificial Hagneck Canal to avoid uncontrolled downstream overtopping during extreme floods. The layout of the zoned erodible dike is shown in Figure 2.4 resulting from hydraulic model tests at VAW (Schmocker et al. 2013).

![Figure 2.4 Cross-section of fuse plug embankment at Hagneck Canal with fat clay (CH), Poorly-graded Sand (SP), Well-graded Gravel (GW) and Poorly-graded Gravel (GP) (Schmocker et al. 2013)](image)

2.4 Hydraulic model tests since 2011

Table 2.1a-e gives an overview on past hydraulic dike breach modelling due to overtopping, whereas model investigations dealing solely with e.g. cohesive materials, piping failure, seepage or surface protection measures are not considered herein. The parameters
are described for full-model tests as \( w = \text{dike height}, L_K = \text{crest length}, b = \text{channel width}, \)
\( S_o \text{ and } S_d = \text{upstream and downstream dike slope, respectively, } I_S = \text{channel bed slope}, d = \text{sediment size}, b_p = \text{pilot channel width at top}, Q_o = \text{inflow discharge}, Q_b = \text{breach discharge}, Q_M = \text{peak breach discharge}, Q_{b,d} = \text{design breach discharge}, h_o = \text{headwater level}, h_M = \text{maximum headwater level}, V_R = \text{reservoir volume}, A_R = \text{reservoir water surface area} \) and \( \lambda = \text{scale ratio}. \) The design breach discharge \( Q_{b,d} = Q_o - Q_{d,r,d} \) is used for test comparison with \( Q_{d,r,d} = \text{design drainage discharge}, \) which is determined by fully impounding typical test dikes prior to the breach tests and which is needed to define \( Q_o \) for each breach test. The listed investigations differ particularly regarding the erosion process, i.e. 2D (breach initiated across entire channel width along horizontal dike crest) and 3D (local breach initiation in pilot channel followed by breach widening through side erosion), constant inflow discharge and constant or falling reservoir levels and the presence or absence of a surface or core layer. Past tests at VAW are summarized in 2.2.

Schmocker (2011) summarized past hydraulic dike-breach modelling and systematically analyzed plane (2D) dike breach tests with constant inflow discharge as hydraulic boundary condition. He conducted 39 tests to determine model limitations and scale effects, and further 29 breach process tests to investigate the effects of dike slope, inflow discharge, sediment diameter, cohesion, and dike height. The normalized results were then determined for the peak breach discharge, maximum reservoir level, maximum dike height and the dike volume. Preliminary spatial (3D) dike breach tests were conducted and a photogrammetric system was applied to determine the breach topography, which resulted in qualitative data. Schmocker and Hager (2012a) presented selected 2D dike breach tests.

Pickert et al. (2011) conducted 3D dike breach tests using non-cohesive sediment of different grain sizes with a focus on apparent cohesion and pore-water pressure. Two main breach slope failure types were identified, namely shearing and tension. Nakagawa et al. (2011, 2016) conducted 2D dike breach experiments with dikes 0.15 to 0.4 m high built of sediment of grain sizes \( d = 0.064 \) to 0.334 mm to study the effect of resisting shear strength due to matric suction on the erosion rate for unsaturated, non-cohesive sediment and compared it with a numerical model. Orendorff et al. (2011) introduced Particle Tracking Velocimetry (PTV) to determine velocities in a spatial dike breach. Assuming the breach crest cross-section and measuring the flow depth at the breach crest, the breach discharge was determined. Orendorff et al. (2013) conducted four spatial dike breach tests to study the effect of different breach initiation methods, namely V-notch, flat-crest, and blast-induced crater, on breach morphology and outflow characteristics.
Islam (2012) studied the effect of different channel bed heights with an approach flow parallel to the dike axis on the 3D dike breach process. Parameters varied were sediment grain size and inflow discharge. Höck et al. (2012) and Schmocker et al. (2013) described fuse plug model tests conducted at VAW. 2D scale model tests of fuse plugs with a clay core were tested, and a 3D test on prototype scale was conducted. The findings were implemented at the Hagneck Canal design. Hakimzadeh et al. (2013) conducted 40 spatial dike breach tests using dikes of different height \( w = 0.3 \) to 0.4 m, reservoir water surface areas \( A_R \approx 10-20 \text{ m}^2 \), sediment grain sizes \( d_{50} = 0.1 \) to 2 mm and different internal friction angles and cohesive strengths. The dikes were installed at the end of a flume without drainage or impermeable core and the sensitivity of different parameters on the breaching process was determined conducting genetic programming. Yu et al. (2013) conducted fluvial 3D dike breach tests in a rectangular flume with a 180° bend to investigate the effect of river discharge, river water level, sediment size and specific sediment weight.

Van Emelen (2014) studied the effect of sediment transport formulations on dike breach modelling and compared the results with 2D dike breach tests by Schmocker and Hager (2009) and to 3D dike breach tests by Spinewine et al. (2004), but also to new tests conducted by Ferbus and Spitaels (2013) using dikes 0.2 m high of non-cohesive sediment with grain sizes \( d = 0.61 \) and 1.85 mm. The effect of seepage on the breaching mode and the influence of a downstream sand layer on the breach profile evolution were determined. Al-Riffai (2014) conducted 31 successful or partially successful spatial dike breach tests with a focus on sediment compaction effort, initial soil-water state, and tilted (distorted) and quasi-exact scale tests (the sediment was size not scaled). Saturated and unsaturated properties of sediment and the effect of the drainage on the phreatic line are described. Further, 8 successful 2D dike breach tests were conducted to determine the effect of initial soil-water states on the dike erosion properties. For this purpose, the headwater level was impounded above the dike crest using a release gate and pore water pressures and sediment erosion were measured.

Wallner (2014) conducted 3D dike breach tests to study the effect of different reservoir sizes and shapes for dikes of height 0.31 m and falling reservoir water levels. Half-model and full-model tests delivered the same results and validated the assumption of symmetry for half-model tests. Wallner determined the breach discharge, the eroded sediment volume and qualitative breach topographies. Rüdisser (2016) used the same test setup to study the effect of different sediment grain sizes on the 3D dike breach. Amaral et al. (2014) conducted two spatial dike breach tests using widely graded material of median grain size \( d_{50} = 0.3 \text{ mm} \) (silty sand) to determine the effect of compaction and the pilot
channel geometry on the breach discharge hydrographs. Michelazzo (2014) conducted 12 fluvial 3D dike breach tests with parallel approach flow and introduced a “Froude graph”, to determine the inflow to outflow ratio as a function of the boundary conditions. Prior to these tests, Michelazzo conducted 26 tests with different fixed boundary conditions, e.g. side weir length or outflow condition. Battarai et al. (2015) conducted 3D dike breach tests with parallel approach flow and constant inflow discharge using different sediment sizes without drainage or impermeable layer.

Cestero et al. (2015) conducted eight 3D dike breach tests under constant headwater level conditions using sediment of grain sizes $d_{50} = 0.159$ to 0.628 mm. The effect of different clay percentages in the sediment mixture on the breach process, hydrographs and breach topography was determined. A novel test method using floating foam tracers was validated. Arita et al. (2015) studied the effect of different dike materials with sediment grain sizes $d = 0.022$ to 1.43 mm on the breach widening for 3D dike breach tests. Ma et al. (2015) conducted 3D dike breach tests with falling reservoir level to determine the effects of dike height, sediment grain size, surface slope and reservoir volume. They determined the breach discharge hydrographs, the breach profiles and the erosion rate. Müller et al. (2016) conducted two 2D dike breach test series for (1) constant inflow discharge conditions, and (2) constant headwater conditions for dikes of different crest lengths and surfaces slopes. Rüdisser (2016) conducted 3D dike breach tests with dikes of height $w = 0.31$ m and falling reservoir level to determine the effect of different sediment grain sizes $d \approx 1$ to 4 mm on sediment erosion and breach discharge. Rüdisser found that sediment erosion and peak breach discharge $Q_M$ is large for large $d$, and that $Q_M$ occurs at an earlier time $t_M$ for large $d$. Further studies with dikes built of non-cohesive sediment and breached by overflow were conducted by Yang et al. (2015) and Rifai et al. (2016a).

Kakinuma and Shimitzu (2014) conducted four large-scale spatial dike breach field tests with parallel approach flow on a river channel. Parameters varied were the sediment grain size $d = 0.2$ to 5.4 mm and inflow discharge $Q_o = 35$ to 70 m$^3$/s. Peeters et al. (2015) presented large-scale dike breach tests conducted in Lillo, Belgium, confirming the five breach stages of Visser (1998). Larese et al. (2015) conducted 2D laboratory tests on rockfill dams subjected to seepage and overflow and compared the results with a numerical approach. Hiller et al. (2016) conducted large-scale field tests of rockfill dams 3 m high with and without riprap protection and determined the hydrographs and the dam stability as a function of overtopping discharge.
Table 2.1a  Hydraulic model tests in dike breaching due to overtopping, values for full-model tests

<table>
<thead>
<tr>
<th>Author</th>
<th>Breach process</th>
<th>Dike dimension</th>
<th>Dike material</th>
<th>Seepage protection</th>
<th>Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frank (2016)</td>
<td>3D, constant inflow discharge, constant headwater level, different reservoir water surface areas, rectangular and triangular reservoir shapes, triangular and trapezoidal pilot channels (PC)</td>
<td>Constant Q$<em>o$ tests: $Q</em>{o,f}$=1.3-83.6 l/s, $w$ = 0.15-0.6 m, $L_K$ = 0-0.4 m, $b$ = 0.5-2 m, $S_v$=$S_t$f=1.5-2, $A_R$ = 0.3-6.9 m$^2$, $b_p$ = 0.04-1.2 m, $\lambda$ = 0.25-1 Reservoir tests: $Q_{o,f}$=0, $w$ = 0.3 m, $L_K$ = 0.1 m, $b$ = 2 m, $S_v$=$S_t$f = 2, $A_R$ = 6.9-206.9 m$^2$</td>
<td>Uniform sediment, Constant Q$_o$ tests: $d$ = 0.43-3.78 mm Reservoir tests: $d$ = 1.75 mm</td>
<td>Bottom drainage below middle of downstream dike slope, drainage length ≈ 0.05-0.2 m (generally 0.1 m)</td>
<td>Breach process, Froude scale models, repeatability, symmetry, hydrographs ($Q_o$, $Q_n$, $Q_{o,f}$, $h_o$), $Q_M$, sediment surface topography, longitudinal and transverse breach profiles, erosion rates, lateral erosion, hydraulic properties for breach profiles ($A$, $v$, $F$, $H$), water surface topography, simulated additional reservoir volume in laboratory channel</td>
</tr>
<tr>
<td>Müller et al. (2016)</td>
<td>2D, constant inflow discharge, constant headwater level</td>
<td>$w$ = 0.2 m, $b$ = 0.2 Constant Q$<em>o$ tests: $Q</em>{o,f}$=4-16 l/s, $L_K$ = 0-0.4 m, $S_v$=$S_t$f=2-3 Constant $h_o$ tests: $h_M$ = 0.226-0.245 m, $L_K$ = 0.1 m, $b$ = 2 m, $S_v$=$S_t$f = 2</td>
<td>Uniform sediment</td>
<td>Bottom drainage below upstream dike slope, limited drainage discharge capacity</td>
<td>Breach process, breach profiles, $Q_b$, $Q_M$</td>
</tr>
<tr>
<td>Walder et al. (2015)</td>
<td>3D, falling reservoir level, triangular PC (inserted after filling)</td>
<td>$w$ = 0.57-1.02 m, $L_K$ = 0.33-1.78 m, $b$ = 2 m, $S_v$=$S_t$f=1.73, $A_R$ = 18.1-23.7 m$^2$</td>
<td>$d_{so}$ = 0.21 mm Uniform sediment</td>
<td>Drain tube on the ground below the middle of the downstream dike slope surface</td>
<td>Breach process, timing of key events, hydrographs, $Q_b$, $v$, inflow crest shape, Froude number at crest ($F_{crest}$), $Q_b$, central streamwise breach profiles, dimensionless relations</td>
</tr>
<tr>
<td>Cestero et al. (2015)</td>
<td>3D, constant headwater level, triangular PC</td>
<td>$w$ = 0.25 m, $L_K$ = 0.1 m, $b$ = 0.5 m, $S_v$=$S_t$f=3, $A_R$ = 5.4 m$^2$, $b_p$ = 0.1 m</td>
<td>$d_{so}$ = 0.159 to 0.628 mm, non-uniform sediment, varied % of clay</td>
<td>None</td>
<td>Breach process, breach shape using novel method (floating foam tracers)</td>
</tr>
</tbody>
</table>
Table 2.1b  Hydraulic model tests in dike breaching due to overtopping, values for full-model tests

<table>
<thead>
<tr>
<th>Author</th>
<th>Breach process</th>
<th>Dike dimension</th>
<th>Dike material</th>
<th>Seepage protection</th>
<th>Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>van Emelen (2013, 2014)</td>
<td>2D, constant inflow discharge</td>
<td>$Q_o = 2.5-5 \text{l/s}$, $w = 0.2 \text{ m}$, $L_K = 0.1 \text{ m}$, $b = 0.2 \text{ m}$, $S_o=S_d=2$</td>
<td>$d = 0.61$ and $1.85 \text{ mm}$</td>
<td>Uniform sediment</td>
<td>Sediment surface profiles, pivot point, seepage effects, comparison to numerical simulations</td>
</tr>
<tr>
<td>van Emelen et al. (2015)</td>
<td>3D, falling reservoir level, different reservoir sizes and shapes, triangular PC</td>
<td>$w = 0.31 \text{ m}$, $L_K = 0.01 \text{ m}$, $b = 2 \text{ m}$, $S_o=S_d=2$, $V_R = 1-4 \text{ m}^3$, $A_K \approx 3.3-25 \text{ m}^2$, $b_p = 0.04 \text{ m}$</td>
<td>$d_{so} = 1.1 \text{ mm}$</td>
<td>Uniform sediment</td>
<td>Bottom drainage below downstream dike face slope</td>
</tr>
<tr>
<td>Wallner (2014)</td>
<td>3D, falling reservoir level, triangular PC</td>
<td>$w = 0.25-0.6 \text{ m}$, $L_K = 0.08-0.2 \text{ m}$, $b = 0.38-1.5 \text{ m}$, $S_o=S_d=2$, $\rho_d = 1496-1598 \text{ kg/m}^3$, $b_p = 0.04 \text{ m}$, $\lambda = 0.33-1$</td>
<td>$d_{so} = 0.17-0.245 \text{ mm}$</td>
<td>Non-uniform sediment</td>
<td>Tests with and without undertow drain ($d_{so} = 1.14-4.5 \text{ mm}$)</td>
</tr>
<tr>
<td>Al-Riffai (2014)</td>
<td>3D, fluvial dike breach, constant inflow discharge, downstream channel regulation by sluice gate, triangular PC</td>
<td>$I_S \approx 1/1000$, $w = 0.25 \text{ m}$, $L_K = 0.1 \text{ m}$, $S_o=S_d=2$, $Q_o = 7.2$ and $70.3 \text{ l/s}$, $b_p = 0.02 \text{ m}$</td>
<td>$d_{so} = 0.84 \text{ mm}$</td>
<td>Uniform sediment</td>
<td>Toe drain</td>
</tr>
<tr>
<td>Michelazzo (2014)</td>
<td>3D, fluvial dike breach, constant inflow discharge, downstream channel regulation by dam-up facility, trapezoidal PC</td>
<td>$I_S \approx 1/500$, $w = 3 \text{ m}$, $L_K = 3-6 \text{ m}$, $S_o=S_d=2$, $Q_o = 35$ and $70 \text{ m}^3/\text{s}$</td>
<td>$d = 0.13$ and $1.00 \text{ mm}$</td>
<td>Non-uniform sediment</td>
<td>None</td>
</tr>
<tr>
<td>Kakinuma and Shimizu (2014)</td>
<td>3D, fluvial dike breach, constant inflow discharge, downstream channel regulation by dam-up facility, trapezoidal PC</td>
<td>$I_S \approx 1/500$, $w = 3 \text{ m}$, $L_K = 3-6 \text{ m}$, $S_o=S_d=2$, $Q_o = 35$ and $70 \text{ m}^3/\text{s}$</td>
<td>$d = 0.13$ and $1.00 \text{ mm}$</td>
<td>Non-uniform sediment</td>
<td>None</td>
</tr>
<tr>
<td>Kakinuma and Shimizu (2014)</td>
<td>3D, fluvial dike breach, constant inflow discharge, downstream channel regulation by dam-up facility, trapezoidal PC</td>
<td>$I_S \approx 1/500$, $w = 3 \text{ m}$, $L_K = 3-6 \text{ m}$, $S_o=S_d=2$, $Q_o = 35$ and $70 \text{ m}^3/\text{s}$</td>
<td>$d = 0.13$ and $1.00 \text{ mm}$</td>
<td>Non-uniform sediment</td>
<td>None</td>
</tr>
</tbody>
</table>

Seepage protection
- 2 tests none, 2 tests impermeable plank
- 1 test sand core, 4 tests 55 mm downstream sand layer

Bottom drainage
- Below downstream dike face slope

Tests with and without undertow drain
- ($d_{so} = 1.14-4.5 \text{ mm}$)

Seepage protection
- $h_o$, reservoir volume, erosion rate, breach shape after each test, qualitative surface topography

Results
- Repeatability, symmetry, $Q_b$, $Q_M$, $h_o$, reservoir volume, erosion rate, breach shape after each test, qualitative surface topography
- Breach process, distorted and quasi-exact scale models, repeatability, symmetry, $Q_b$, $Q_M$, erosion rate, lateral erosion, inflow crest shape, effect of soil suction
- Breach process, breach width, bed topography, hydrographs, flow velocities, “Froude graph” (ratio inflow / outflow as a function of boundary conditions)

Wallner (2014)
- Repeatability, symmetry, $Q_b$, $Q_M$, $h_o$, reservoir volume, erosion rate, breach shape after each test, qualitative surface topography
## Table 2.1c  Hydraulic model tests in dike breaching due to overtopping, values for full-model tests

<table>
<thead>
<tr>
<th>Author</th>
<th>Breach process</th>
<th>Dike dimension</th>
<th>Dike material</th>
<th>Seepage protection</th>
<th>Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hakimzadeh <em>et al.</em> (2013)</td>
<td>3D, falling reservoir level, rectangular PC</td>
<td>$w = 0.3-0.4$ m, $L_K = 0$ m, $b = 1$ m, $S_v=2.5-3.5$, $S_d=2.5$, $b_p = 0.02$ m, $A_R \approx 10-20$ m$^2$</td>
<td>$d = 0.1-2$ mm, $\varphi = 30-45^\circ$, sand and sand-clay mixtures</td>
<td>None</td>
<td>Symmetry, hydrographs, final breach width and depth, qualitative sensitivity of parameters</td>
</tr>
<tr>
<td>Yu <em>et al.</em> (2013)</td>
<td>3D, fluvial boundary condition, constant inflow discharge, downstream channel regulation by V-notch weir, 180° flume bend, rectangular PC</td>
<td>$I_S = 1/1000$, $w = 0.15-0.18$ m, $L_K = 0.1-0.15$ m, $S_v=1-1.6$, $S_d =1-2.8$, $Q_o = 5.64-15.62$ l/s, $h_o = 0.0135-0.0165$ m</td>
<td>$d = 0.33-0.62$ mm, Uniform sediment $\rho_S = 1.42-2.65$ kg/m$^3$</td>
<td>None</td>
<td>Breach process, breach width, breach profile, hydrographs, dimensionless relationship of $Q_b$ and $h_o$</td>
</tr>
<tr>
<td>Peñuela (2013)</td>
<td>3D, constant water level, centrifugal model, increased gravity 33-g</td>
<td>$w = 0.15$ m, $L_K = 0.12$ m, $b = 0.5$ m, $S_v=S_d=2-3$</td>
<td>$d = 0.24$ mm, Non-uniform sediment</td>
<td>None, toe filter, cut-off wall, combined toe filter and cut-off wall</td>
<td>Breach process differences, pore water pressures, breach surface profiles</td>
</tr>
<tr>
<td>Islam (2012)</td>
<td>3D, fluvial dike breach, constant inflow discharge, downstream channel regulation by V-notch weir, rectangular PC</td>
<td>$I_S = 1/500-1/1000$, $w = 0.15$ m, $L_K = 0.1$ m, $S_v=S_d=2$, $b_p = 0.1$ m, $Q_o = 5-9.5$ l/s</td>
<td>$d = 0.13$ and $1.00$ mm, Uniform sediment</td>
<td>Bottom drainage below downstream dike slope, length = 0.05 m</td>
<td>Breach process, longitudinal breach widening, breach topography, surface flow velocities, comparison with numerical simulation</td>
</tr>
<tr>
<td>Schmocker (2011)</td>
<td>2D, constant inflow discharge</td>
<td>$w = 0.1-0.4$ m, $L_K = 0.05-0.1$ m, $b = 0.1-0.4$ m, $S_v=S_d=2-3$</td>
<td>$d = 0.86-7.9$ mm, Uniform sediment</td>
<td>Bottom drainage below upstream dike slope, limited discharge capacity</td>
<td>Breach process, breach profiles, $Q_b$, study on scale effects, dimensionless relations</td>
</tr>
<tr>
<td>Author</td>
<td>Breach process</td>
<td>Dike dimension</td>
<td>Dike material</td>
<td>Seepage protection</td>
<td>Results</td>
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</tr>
<tr>
<td>Pickert et al. (2011)</td>
<td>3D, rectangular pilot channel, constant reservoir level</td>
<td>$w = 0.3 \text{ m}, L_K = 0.1 \text{ m}, b = 2 \text{ m}, S_o=S_d=3$</td>
<td>$d = 0.185-0.64 \text{ mm}$ Uniform sediment</td>
<td>Bottom drainage at downstream toe</td>
<td>Breach process, breach profiles, $Q_b$, soil suction, dimensionless breach hydrographs</td>
</tr>
<tr>
<td>Gregoretti et al. (2010)</td>
<td>3D, constant inflow discharge, sloping bed 0-10%</td>
<td>$w = 0.2-0.4 \text{ m}, L_K = 0-0.4 \text{ m}, b = 0.5 \text{ m}, various S_o and S_d$</td>
<td>$d = 3.9-10.3 \text{ mm}$ Uniform sediment</td>
<td>None</td>
<td>Breach process, headcutting failure</td>
</tr>
<tr>
<td>Jandora and Jaromir (2008)</td>
<td>3D, falling reservoir level, rectangular PC</td>
<td>$w = 0.86 \text{ m}, L_K = 0.2 \text{ m}, b = 3.71 \text{ m}, S_o=S_d=2$</td>
<td>$d = 1.7 \text{ mm}$ Non-uniform sediment</td>
<td>None</td>
<td>Breach process, $Q_b$</td>
</tr>
<tr>
<td>Ribi et al. (2008)</td>
<td>2D, 3D, constant reservoir level</td>
<td>$w = 0.4 \text{ m}, L_K = 0.2 \text{ m}, b = 0.15-5 \text{ m}, S_o=S_d=1.5$</td>
<td>$d = 0.4-3 \text{ mm}$ Uniform sediment</td>
<td>None</td>
<td>Breach process, lateral erosion, scaling issues</td>
</tr>
<tr>
<td>Dupont et al. (2007)</td>
<td>2D, constant inflow discharge</td>
<td>$w = 0.25-0.35 \text{ m}, L_K = 0.2-0.28 \text{ m}, S_o=1.75-2, S_d=2$</td>
<td>$d = 2-4 \text{ mm}$ and 2-7 mm, non-uniform sediment</td>
<td>Clay surface layer</td>
<td>Similarity rules, breach process, breach profiles, pivot point, $Q_b$</td>
</tr>
<tr>
<td>Morris et al. (2007)</td>
<td>3D, constant reservoir level, trapezoidal PC</td>
<td>$w = 0.5-0.6 \text{ m}, L_K = 0.2-0.3 \text{ m}, b = 4 \text{ m}, S_o=S_d=1.7-2$</td>
<td>$d = 0.0045-0.7 \text{ mm}$ non-uniform sediment</td>
<td>None</td>
<td>Breach formation, comparison with Norwegian field tests (Høegg 2004)</td>
</tr>
<tr>
<td>Visser et al. (2006) Visser (1998)</td>
<td>3D: rectangular pilot channel, constant reservoir level; 3D field test: constant headwater level (tide-dependent) trapezoidal PC</td>
<td>$3D: w = 0.15 \text{ m}, L_K = 0.2 \text{ m}, b \approx 10 \text{ m}, S_o=S_d=2, S_o=S_d=4, b_p = 0.2 \text{ m}; 3D field test: w = 2.6 \text{ m}, L_K = 8 \text{ m}, b = 0.7 \text{ m}, S_o=1.6, S_d=3, b_p = 3.6 \text{ m}$</td>
<td>$3D: d = 0.088 \text{ mm}$ Uniform sediment</td>
<td>None, mobile bed</td>
<td>Breach process, breach stages, breach types, breach width evolution, breach profiles, hydrographs</td>
</tr>
</tbody>
</table>
### Table 2.1e  Hydraulic model tests in dike breaching due to overtopping, values for full-model tests (adapted from Schmocker 2011)

<table>
<thead>
<tr>
<th>Author</th>
<th>Breach process</th>
<th>Dike dimension</th>
<th>Dike material</th>
<th>Seepage protection</th>
<th>Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lüthi (2005)</td>
<td>2D, constant inflow discharge</td>
<td>$Q_o = 1-10$ l/s, $w = 0.05-0.3$ m, $L_K = 0.1$ m, $b = 0.1$ m, $S_o=S_f=2-5$</td>
<td>$d = 1.15$-5 mm Uniform sediment</td>
<td>None</td>
<td>Breach process, breach profiles, $Q_b$, dimensionless relations</td>
</tr>
<tr>
<td>Hager and Unger (2006)</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Chinnarasri et al. (2004)</td>
<td>3D, constant inflow discharge, rectangular PC</td>
<td>$w = 0.6$ m, $L_K = 0.3$ m, $b = 4$ m, $S_o=3$, $S_f=2-3$</td>
<td>$d = 0.34$-0.6 mm Uniform sediment</td>
<td>3 tests with clay surface layer</td>
<td>Breach process, breach geometry, $Q_b$</td>
</tr>
<tr>
<td></td>
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<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Spinewine et al. (2004)</td>
<td>3D, falling reservoir level, trapezoidal PC</td>
<td>$w = 0.47$ m, $L_K = 0.2$ m, $b = 2.4$ m, $S_o=2$, $S_f=3$</td>
<td>$d = 1$-2 mm Uniform sediment</td>
<td>None</td>
<td>$Q_b$, surface velocities, water and bed level profiles,</td>
</tr>
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<tr>
<td>Chinnarasri et al. (2003)</td>
<td>2D, constant inflow discharge</td>
<td>$w = 0.8$ m, $L_K = 0.3$ m, $b = 1$ m, $S_o=3$, $S_f=2-5$</td>
<td>$d = 0.36$-0.86 mm Uniform sediment</td>
<td>Bentonite surface layer</td>
<td>Breach process, breach patterns, degradation rate of dike crest</td>
</tr>
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<tr>
<td>Rozov (2003)</td>
<td>3D, constant reservoir level, rectangular PC</td>
<td>$w = 0.2$ m, $L_K = 0.2$ m, $b = 1.25$ m, $S_o=S_f=3$</td>
<td>$d = 0.34$ mm Uniform sediment</td>
<td>None</td>
<td>Breach process, breach profiles, $Q_b$</td>
</tr>
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<tr>
<td>Coleman et al. (2002)</td>
<td>3D, constant reservoir level, triangular PC</td>
<td>$w = 0.3$ m, $L_K = 0.065$ m, $b = 4.8$ m, $S_o=S_f=2.7$</td>
<td>$d = 0.5$-2.4 mm Uniform sediment</td>
<td>Toe drain (downstream)</td>
<td>Breach process, breach profiles, erosion rates, $Q_b$</td>
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<tr>
<td>Pugh (1985)</td>
<td>3D, constant reservoir level, triangular PC</td>
<td>$w = 0.15$-0.38 m, $L_K = 0.12$-0.24 m, $b_{max} = 5.4$ m, $S_o=S_f=2$</td>
<td>$d = 0.6$-1 mm Uniform sediment</td>
<td>Clay core and sand filter</td>
<td>Breach process, lateral erosion rate,</td>
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<tr>
<td>Sametz (1981)</td>
<td>2D, initial target headwater level $h_0$ using board on dike crest</td>
<td>$w = 0.15$-0.6 m, $L_K = 0$, $b = 0.76$ m, $V_R = 0.5 - 4.5$ m³, $S_o=S_f=1.5-2$, $S_f=1.3-2$</td>
<td>$d = 0.5$-15 mm Non-uniform sediment</td>
<td>Sand core or sand surface layer mixed w. wallpaper paste</td>
<td>Breach process, breach profiles, hydrographs, erosion rates</td>
</tr>
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<td></td>
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<td></td>
</tr>
<tr>
<td>Tinney and Hsu (1961)</td>
<td>3D, constant reservoir level, clay and sand core, rectangular PC</td>
<td>$w = 0.2$-0.4 m, $L_K = 0.15$ m, $S_o=1.25$, $S_f=1.5$</td>
<td>$d = 4$-10 mm Uniform sediment</td>
<td>Clay core with sand filter</td>
<td>Breach process, washout rates, scaling issues</td>
</tr>
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</table>
Dike breach tests using geotechnical centrifuge facilities to increase gravity and correctly simulate the stresses in the soil mass were conducted by Kamalzare et al. (2012) and Peñuela (2013). Recent research focusing on headcut erosion for dike breaches using cohesive dikes was e.g. conducted by Zhao et al. (2014), who describes three types of erosions, namely surface erosion, headcut erosion and lateral/helicoidal erosion.

The majority of the presented research focused on one or few specific dike breach parameters for different hydraulic boundary conditions. The different test conditions and observations for the specific parameters are described below.

2.5 General dike breach observations

2.5.1 Failure modes

Earth dams are built of homogeneous material or are structured depending on several factors: Function, hydraulic and geotechnical conditions, safety requirements, available materials, maintenance. Foster et al. (2000) and CIRIA (2013) give an overview of the different dam zoning categories and differentiate between causes of failure and the modes of failure. The failure mode categories are divided into:

- Flood overtopping
- Piping (through foundation or embankment)
- Slope instability (up- or downstream slides)
- Earthquake.

According to Singh (1998) and Foster et al. (2000), overtopping failures, as considered herein, are the most common failure modes, in general agreement with further national statistics.

2.5.2 Flow regimes

For 2D dike breaches, Powledge et al. (1989) found that the flow regime changes from sub- to supercritical above the dike crest. For 3D dike breach tests, Coleman et al. (2002, 2004) stated that a non-cohesive breach boundary will deform to approach flow conditions of minimum energy and maximum discharge per unit width for the available specific energy, similar to Minimum Energy Loss (MEL) weirs (Chanson 2004a). In a simple
MEL inlet design, the flow is assumed critical from the upstream lip to the throat. Chanson (2004b) concludes by observing images by Andrews (1998) for non-cohesive embankment breaches, that flow conditions are near critical ($0.5 < F < 1.8$) and that the movable boundary flow tends to an equilibrium associated with minimum energy conditions and maximum discharge per unit width for the available specific energy. Walder et al. (2015) conducted 3D dike breach tests using falling reservoir water level conditions and also found a breach channel shape similar to a MEL inlet and Froude numbers of $F = 0.75\pm0.14$ at the inflow crest. Their findings were confirmed e.g. by Al-Riffai (2014) with $F = 0.75$-0.85 for flow over a weir of trapezoidal profile and narrow crest, and by Kirkgoz et al. (2008) with $F = 0.78$ for flow over a triangular-profiled weir. Zhao et al. (2015) studied the flow properties for different fixed breach setups.

### 2.5.3 Breach stages

Visser (1998) describes five Breach Stages for non-cohesive 3D dike breaches:

- **Stage I:** Steepening of inner slope in initial breach
- **Stage II:** Decrease of pilot channel length
- **Stage III:** Acceleration of breach growth
- **Stage IV:** Continuation of breach growth in lateral direction. A dependency of the breach evolution on the dike foundation becomes obvious, divided in three Breach Types:
  - A. 1. Solid clay-layer prevents further vertical erosion
     2. Toe construction protects outer slope against further vertical erosion
  - B. High foreland reduces increase of breach inflow
  - C. Unobstructed breach growth
- **Stage V:** Deceleration of breach growth

Visser (1998) observes that the breach channel develops in Stages I and II, with almost no increase in breach discharge. In Stage III, the breach growth accelerates vertically and horizontally, and consequently also the breach discharge. After wash-out of the breach channel, the breach growths mainly laterally in Stage IV. In Stage V, a reduced head decelerates the breach discharge and the breach widening slows down. The breach erosion processes in Stages IV and V depend on the erodibility of the base material upstream, below and downstream of the dike.
For dike breaches due to overtopping, progressive surface erosion (sediment transport in dispersed particles driven by shear forces) is usually observed for non-cohesive dikes, while headcut erosion (formation and migration of a vertical or almost vertical drop in the breach channel) is observed for cohesive embankments. Still, non-cohesive and heavily compacted dikes may also erode by a headcut. The transition between surface and headcut erosion modes is not well defined yet (Wu et al. 2011). For clay-dikes, Zhu (2006), similarly to Visser (1998), describes the breach erosion process in five stages. Other stage or phase distinctions were described e.g. by Morris et al. (2009a,b).

2.6 Hydraulic boundary conditions for dike breach models

Different hydraulic boundary conditions exist for prototype dikes. When conducting systematic dike breach tests in a laboratory channel, most researchers conduct the tests with frontal approach flow due to the flume setup, and have to decide on the hydraulic boundary condition for their test series:

Constant inflow discharge \( Q_0 \) was applied as hydraulic boundary condition by Schmocker (2011), Schmocker et al. (2014) and Müller et al. (2016) for 2D dikes. For these tests, the reservoir water surface area is relatively small compared with the inflow discharge, and the water level rises continuously even after dike overflow is initiated, reaching a maximum headwater level before falling fast with the eroding dike crest, which leads to the maximum breach discharge. Eventually, breach discharge will reach inflow discharge. This setup has the advantage of a well-controlled boundary condition and is easily applicable in relatively small hydraulic channels, so that the general breach properties can be studied for 2D and 3D dike breaches. For the 3D tests however, the pilot channel dimensions need to be relatively large for the constant inflow discharge to pass without overtopping the dike. Depending on the channel dimensions and the dike position, the reservoir water surface area is relatively large in 3D dike breach tests.

Constant reservoir level \( h_0 \) generally proved to be challenging concerning discharge control and pumping capacity. Coleman et al. (2002) conducted tests with constant reservoir level conditions without specifying the regulation method or presenting the headwater level evolution, although repeatability of the breach discharge hydrograph was satisfactory. Cestero et al. (2015) regulated the constant headwater level in the first stages of the experiment by adapting the inflow stepwise using two pumps up to the maximum pump capacity. Müller et al. (2016) conducted tests for 2D dike breaches by applying a standard
pump regulation with a proportional-integral-derivative (PID) controller and one input
signal (inflow discharge) commonly used in industrial control systems. This method led
to satisfactory results even though the headwater level fluctuations were large at breach
start.

Reservoir water surface area tests with $Q_o = 0$ (falling headwater level) with different
reservoir volumes and shapes generally occupy large laboratory space when the volumes
are physically modelled. Furthermore, the breach start conditions can lead to difficulties
when comparing different tests. Tests with different reservoir water surface areas and
reservoir shapes were conducted by Sametz (1981), Hakimzadeh et al. (2013) and Wall-
ner (2014), although others such as Walder et al. (2015) also varied the reservoir water
surface area without it being one of the main investigated parameters. Duverney (2016)
implemented a linear-quadratic-Gaussian (LQG) controller with reference and disturb-
ance feedforward, which is a model-based reference tracking controller with several input
signals (inflow discharge and headwater level), to simulate different reservoir water sur-
face areas and shapes in addition to the reservoir water surface area physically available
in the laboratory channel by regulating the pump rotational speed. His method allowed
for a satisfactory 3D dike breach procedure and accuracy.

Reservoir water surface area tests with $Q_o = \text{constant}$ and frontal approach flow represent
a standard case of prototype reservoirs, whose capacity is exhausted due to extreme floods
or a spillway blockage resulting in overflow of the dike crest. For these scenarios, tests
are mostly conducted as scale models for a specific dam with a specific flood discharge
(e.g. Shuibo and Loukola 1993), as they occupy a relatively large laboratory space. A
systematic variation in a physical model is therefore difficult. Besides numerical simul-
ations, the pump regulation with the LQG controller presented in 3.5 is a possible ap-
proach, since a combination of inflow discharge, reservoir water surface area and reser-
voir shape can be simulated, so that the laboratory expenditure is considerably reduced.

Spatial dike breaches with parallel approach flow (fluvial tests) have been conducted to
simulate the complex interactions between river discharge and water levels and the asym-
metric breach development. These tests are relatively complex concerning the river out-
flow condition and the breach topography measurement using standard measurement
tools, since a non-symmetrical breach develops and no profiles can be extracted e.g.
through a glass wall. Tests using the setup with parallel approach flow have been con-
ducted by Islam (2012), Yu et al. (2013), Michelazzo (2014), and currently Rifai et al.
(2016a), the difference being mainly the river outflow condition and the system applied
to determine the breach characteristics. As an example, Rifai et al. (2016a) use perforated boards to obtain a certain water level dependent outflow discharge. Kakinuma and Shimizu (2014) conducted large-scale 3D dike breach tests on an actual river channel and used a dam-up facility to control the river outflow condition.

Breaches induced by wave overtopping are another important breach scenario. Waves are created by strong winds or strong currents and can overtop a dike structure in a non-continuous way. A first attempt to simulate these processes has been conducted as part of the IMPACT Program by developing models simulating breach initiation from wave action (Morris 2009b). Further research was conducted e.g. by Xu et al. (2015) and Evers et al. (2017).

Note that the term ‘overtopping’ describes herein a continuous water overflow rather than overtopping by waves.

2.7 Methods to determine 3D dike breach topography

Plane dike breaches have been studied on a laboratory scale by recording the breach progress with a camera through the glass side wall (e.g. Sametz 1981, Schmocker and Hager 2009). When applying this method to half-model 3D dike breach tests though, only the dike surface profile directly at the glass wall is recorded and the remaining topography is unknown. Since dike breaches are unsteady flow processes, the breach shape is difficult to assess with non-intrusive experimentation. Different methods recently applied to determine the 3D dike breach topography are therefore presented in the following.

Coleman et al. (2002) captured the breach topography of spatial dike breaches by repeating each test several times, stopping at a certain instant to furnish the instantaneous breach topography. The spatial breach profiles were recorded using digital potentiometers. This approach requires a big laboratory effort, and may therefore be applied only to a limited test number. Rozov (2003) conducted spatial breach tests using washout indicators. Spinewine et al. (2004) used a laser line sheet system for capturing spatial dike breaches, allowing for an unprecedented degree of details. The sweeping cycle takes about 5 s. For the submerged breach portion, the difference in the refractive indices of air and water leads to an error in the sediment surface elevation of the order of 30 mm. A similar system is applied by Rifai et al. (2016a) for fluvial dike breach tests with parallel approach flow delivering promising results. The accuracy of this system was determined by Rifai et al. (2016b) for dry and submerged dikes without water flow. Pickert et al. (2011) employed
a fringe projection system with a side camera, allowing for continuous and non-intrusive measurements of the dike breach topography across the glass wall. Inaccuracies due to different refraction at the glass/air interface above, and the glass/water interface below the water surface occur, so that significant manual correcting was applied. This system appears to be interesting if erosion is mainly in the vertical direction. Peñuela (2013) captured low resolution spatial dike breach features in geotechnical centrifuge facility tests by mounting reference markers on the dike surface and applying a photogrammetric system. Acceleration sensors were used to determine the breach evolution in field tests as a complementary method e.g. by Kakinuma and Shimitzu (2014) or Hiller et al. (2016).

Al-Riffai (2014) captured the basic topographical dike breach features (inflow crest, local breach width) by applying a photogrammetric system and a georeferenced grid painted on the dike surface. However, refraction effects were not quantified. Wallner (2014) analyzed the influence of reservoir shape and size on the discharge hydrograph for spatial dike breaches; he captured dike breach surfaces with an optical system based on the commercially available Microsoft Kinect® sensor, resulting in dike sediment surface data; the accuracy is relatively poor due to refraction effects. Cestero et al. (2015) embedded colored foam tracers in four sediment layers with a grid spacing of 80 to 100 mm during the model dike construction. The tracers’ appearance on the flowing water surface marked the moment of their detachment. This novel method is interesting for flows where the sediment surface is not visible due to water turbidity or sediment overhangs. The duration between the tracer dislodgement and observation was assumed negligible. Since the tracers do not dislodge simultaneously, the breach topography is determined by interpolation over time and space, as discussed by Cestero et al. (2015a,b) and Frank and Hager (2016a). Walder et al. (2015) applied a novel photogrammetric method, including an underwater camera upstream of the dike, to determine the inflow crest shape and the streamwise breach profiles by analyzing selected stereo pairs of still-camera images.

A 3D-photogrammetric measuring system was developed for the German Federal Waterways Engineering and Research Institute (BAW) by AICON 3D Systems Ltd., Germany (Godding et al. 2003). BAW and Henning et al. (2009) used this system for riverbed measurements below quasi-horizontal water surfaces. Preliminary tests on its applicability to dike breach experiments were conducted by Schmocker and Hager (2011). The first results including a successful elimination of refraction effects were presented by Frank (2014) and Frank and Hager (2014), who determined the measuring accuracy and limitations of this novel technique. Further results including the topography and hydraulic characteristics for 3D dike breach tests were presented by Frank and Hager (2015, 2016c). In
summary, the photogrammetric system functioned well after software and hardware optimizations (3.8) and some 35 preliminary 3D dike breach tests, which were also used to optimize the test setup (3.2 to 3.7) and followed by 80 preliminary 2D dike breach tests by Frank (2013) not presented in this study. Evers and Hager (2015) studied the propagation of spatial impulse waves applying this videometric system on white-colored water and determined the temporal evolution of the water surface topography. A green laser system is currently employed at VAW to determine river bed profiles during the test and results in high resolution topographic data of satisfactory data quality by eliminating refraction effects (Friedl et al. 2016). The green laser system is currently too slow to capture the fast process of spatial dike breaches, however.

2.8 Effect of relevant breach parameters

2.8.1 Discharge and flow velocity

Schmocker (2011) and van Emelen (2014) found that the 2D dike breach process is accelerated for large inflow discharge. The peak outflow to inflow ratios are small, which suggests that the outflow is dominated by the inflow conditions as opposed to a falling head corresponding to a large reservoir volume, as also noted by Al-Riffai (2014).

Yu et al. (2013) found for fluvial 3D dike breach tests that a large water level difference between up- and downstream of the dike leads to a faster breach expansion and slows down when the head decreases. For different constant inflow discharges, the breach develops similarly during the first 50 s because the breach process is mostly dominated by the available head. The breach widening increases only when the breach has developed under considerable breach discharge. Michelazzo (2014) also conducted fluvial tests with parallel approach flow and found a linear relationship between inflow discharge $Q_o$ and the breach length.

Bishop et al. (2016) found that the pickup flux of sand at high flow velocities of 2 to 6 m/s is influenced by sediment porosity and can be explained by the theory of dilatancy-reduced pickup, as confirmed by van Rhee (2010). Van Rhee derived an erosion formula that includes the effect of hindered erosion based on the effect of soil dilatancy and permeability. Bishop et al. (2016) applied the simplified van Rhee model including hindered erosion to a large-scale dike breach test, the so-called Zwin’94 experiment (Visser 1998).
with satisfactory agreement of erosion rates for large flow velocities as compared with the approach of van Rijn (1984).

Sametz (1981) compiled data on the critical discharge $Q_c$ and the critical headwater level $h_c$ for the inception of sediment transport as a function of the downstream dike slope $S_u$, sediment grain size $d$, and sediment compaction. For large $S_u$, $d$, or compaction, $Q_c$ and $h_c$ become large as compared to small $S_u$, $d$, or compaction.

### 2.8.2 Reservoir properties

The reservoir water surface area $A_R$ and its shape play a major role in the dike breach process, as is described by most parametric formulations, e.g. Walder and O'Connor (1997) or Froehlich (2016a,b). Nevertheless, few experimental test series have been conducted with a systematic variation of $A_R$, due to the large laboratory surface needed and the resulting test complications. Sametz (1981) studied the reservoir effect on 2D dike breach model tests and found that for large reservoir volumes $V_R$, the sediment erosion rate $V_E$ and peak breach discharge $Q_M$ are large as compared with small $V_R$, and $Q_M$ occurs at later times $t_M$ (time-to-peak).

Hakimzadeh et al. (2013) found for 3D dike breach tests that the reservoir water volume $V_R$ and the headwater level $h_o$ play a much more important role as compared to the length of the dam $L_D$ or the soil characteristics. Wallner (2014) also conducted 3D dike breach tests and found that for reservoirs containing the same water volume, peak breach discharge was always largest for reservoirs of triangular shape as compared with those of rectangular shape. Furthermore, doubling the reservoir volume led to an increase in peak breach discharge by 70% for rectangular and 50% to 76% for triangular reservoirs. Two distinct dimensionless equations were proposed for the two reservoir shape series.

The main interest in the reservoir shape has been to determine the temporal storage capacity for floods and hydropower generation. For Switzerland, therefore, Kühne (1978) describes the characteristic shapes of 40 artificial and 6 natural reservoirs. The reservoir shape was described graphically either with the capacity curve (reservoir water volume $V_R$ versus the headwater level $h_o$ above the reservoir base) or the curve of the water surface area (reservoir water surface area $A_R$ versus $h_o$). $A_R(h_o)$ is usually determined from topographic data, from which the capacity curve $V_R(h_o)$ can be deduced by integration as

$$V_R(h_o) = \int_0^{h_o} A_R(h_o) dh_o$$  \hspace{1cm} (2.1)
The reservoir capacity curve \( V_R(h_o) \) [\( \text{m}^3 \)] is described with \( a \) [\( \text{m}^3\cdot\text{c} \)] and \( c \) [-] (reservoir shape factor) as reservoir-specific constants as

\[
V_R = ah_o^c
\]  

(2.2)

Accordingly, from Eq. (2.1) and Eq. (2.2), the water surface curve \( A_R(h_o) \) [\( \text{m}^2 \)] is

\[
A_R = ah_o^{c-1}
\]  

(2.3)

For artificial reservoirs, Kühne (1978) determined values between \( 2\cdot10^0 \) and \( 1.96\cdot10^5 \) m\(^3\cdot\text{c} \) for \( a \), and between 1.38 and 3.56 for \( c \), while for natural lakes, \( a \) ranged between \( 4.21\cdot10^4 \) and \( 2.04\cdot10^6 \) m\(^3\cdot\text{c} \) and \( c \) between 1.42 and 1.99. Large values for \( a \) are characteristic for large reservoir water surface areas, while small values for \( c \) are typical for gently inclined reservoir beds, as found in natural lakes. Note that rectangular reservoirs with vertical walls would feature \( c = 1 \), while a triangular cross-section combined with a rectangular longitudinal section (or vice versa) results in \( c = 2 \). A reservoir with a constant slope of the walls and the floor results in \( c = 3 \), as illustrated in Figure 2.5 and summarized in Table 2.2. The same method was described by Walder and O’Connor (1997).

As an example, the artificial Sihl Lake upstream of Zürich with a maximum reservoir capacity of \( 96.5\cdot10^6 \) m\(^3 \) is described with \( a = 1.76\cdot10^5 \) m\(^0.79 \) and \( c = 2.21 \). While the capacity curves of most reservoirs are well described with this formulation, a description using two different sets \((a,c)\) for the upper and lower reservoir reaches is suggested by Kühne in particular cases.

For prototype dike breaches at large reservoirs, further effects need to be considered though. Frank (1951) highlights the effects of reservoir size, shape and of down-surge on

\[\text{Figure 2.5} \quad \text{Longitudinal reservoir sections (I-IV) and reservoir cross-sections (1-4) (adapted from Kühne 1978)}\]
Table 2.2  Dependency of the reservoir shape factor $c$ on the reservoir shape from Figure 2.5. Reservoir shapes marked with * are approximations, those marked with in grey are the most common reservoir shapes (adapted from Kühne 1978)

<table>
<thead>
<tr>
<th>Cross-section</th>
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<th>IV</th>
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<td>1</td>
<td>1.5</td>
<td>2</td>
<td>2.5</td>
</tr>
<tr>
<td>2</td>
<td>1.5</td>
<td>2*</td>
<td>2.5</td>
<td>3*</td>
</tr>
<tr>
<td>3</td>
<td>2</td>
<td>2.5</td>
<td>3</td>
<td>3.5</td>
</tr>
<tr>
<td>4</td>
<td>2.5</td>
<td>3*</td>
<td>3.5</td>
<td>4*</td>
</tr>
</tbody>
</table>

the outflow discharge during a dam breach. Goodell and Wahlin (2009) found differences of up to 20% in peak discharge when applying dynamic flood routing compared to the level-pool flood routing as numerical approach, as also highlighted by Froehlich (2016a).

2.8.3  Sediment grain size

Depending on the sediment grain size $d$, a sediment has either non-cohesive or cohesive properties when exposed to water. To avoid cohesive effects, the mean diameter has to be $d > 0.2$ mm according to Zanke (1982), while $d > 0.8$ mm is necessary according to Hager and Oliveto (2002). The experimental results for dike breach tests using different sediment grain sizes are presented below, divided into 2D and 3D dike breaches tests.

2D dike breach tests

Sametz (1981) conducted 22 2D dike breach model tests for $V_R = 0.5$ to 4.5 m$^3$ and $d_{50} = 0.5$-15 mm, and used a mixture of wallpaper paste and fine sediment as impermeable core. He found for non-cohesive sediment $d \geq 1$ mm that the breach process and erosion rate are independent of the sediment material during a certain duration after breach start. The duration depends on the sediment properties and the channel bed slope, after which the erosion process slows down considerably. The remaining sediment volume at the breach end increases for coarser and angular sediment. A reduced peak breach discharge was observed for small $d < 1$ mm with cohesive effects, however.

Lüthi (2005) found a smaller erosion resistance for small $d$ and therefore a faster erosion process compared to large $d$. Nakagawa et al. (2011, 2016) conducted 2D dike breach
experiments with $d = 0.064$ to $0.334$ mm and found that for large $d$ the erosion rate increases. Their study attributes this effect on the erosion rate to resisting shear strength due to matric suction; they were able to reproduce these results in a numerical model by considering suction. The difference in erosion rates is not quantified though, and appears to be relatively small in the diagrams. The dissimilarities in the dike breach profile in terms of a more continuous surface erosion for large $d$ and headcut erosion for small $d$ are clearly visible.

Schmocker (2011) conducted 2D dike breach tests with $d = 0.31$ to $8$ mm and found that the breach process is accelerated with increasing $d$ due to surface slips and the increased dike permeability for coarse sediment. For large times, the erosion process is decelerated with increasing $d$, however, as this stage is mainly controlled by sediment transport and therefore by the incipient motion criteria. It was noted that for $d < 1$ mm the eroded sediment does not deposit below the breach to form a stabilizing body but is transported further downstream, in contrast to dike breaches with $d > 1$ mm. While sediment of grain size $d = 1$ mm is considered non-cohesive, cohesive characteristics were observed and attributed to apparent cohesion. Schmocker et al. (2014) found that the sediment grain size distribution has no effect on the breach process for grain sizes $d_{50} = 1.8$ to $5.9$ mm. The results indicate that in the 2D dike breach process, breach discharge and erosion rate are independent of the sediment grain size during a certain duration after breach start for non-cohesive sediment during breach tests, but only if dike failure is not accelerated due to seepage. For larger tests times and smaller downstream breach surface slopes, the process is mainly controlled by sediment transport and therefore by the entrainment criteria, so that the remaining dike body will be larger for large $d$. For small $d < 1$ mm, apparent cohesion or even real cohesion will influence the 2D dike breach process and lead e.g. to smaller peak breach discharges (Sametz 1981).

3D dike breach tests

Tinney and Hsu (1961) found for 3D dike breaches and $d = 4$-10 mm that the washout rate is slightly larger for small sediment as compared to large sediment. In contrast, Chinnarasri et al. (2004) found for 3D dike breach tests and $d = 0.34$-$0.6$ mm, that the peak outflow increases and breach deformation time decreases as $d$ increases.

Xu and Zhang (2009) conducted a multivariable regression on prototype earth and rockfill dam breaches and found that the dam erodibility is the most important factor, influencing
all other breach parameters. Higher dike erodibility allows for a more rapid erosion development, and hence leads to larger breach depths and widths, a higher peak outflow rate, and a shorter failure time.

Pickert et al. (2011) conducted 3D dike breach tests using sediment of grain size $d = 0.185$ to $0.64$ mm. They found that coarse material tends to have steady erosion due to regular side slope failures, while fine material leads to unsteady erosion rates due to large mass failures of breach side slopes. Two main breach slope failure types were identified, namely shearing and tension. Slope failure due to tension was observed mainly for fine sediments, while shearing failure was observed mainly for coarse sediment. The breach side slopes were found to be stable due to the negative pore-water pressure and the corresponding apparent cohesion for non-cohesive sediment. They concluded that the effect of apparent cohesion promotes a tensile strength approach. Extending their results with test data of Coleman and Andrews (2000) and Morris et al. (2007), the authors differentiated between tests with $d > 1$ mm (shear stress controlled) and $d \leq 1$ mm (apparent cohesion controlled). A dimensionless time for apparent cohesion controlled tests was determined.

For fluvial dike breach tests, Islam (2012) noticed a faster breach widening for large grain sizes for 3D dike breach tests, whereas Yu et al. (2013) found for $d = 0.4$ and $0.62$ mm a slightly faster widening at breach start for the large grain size as compared with the smaller, but a slower expansion after 50 s for the larger gain size.

Hakimzadeh et al. (2013) conducted 3D half- and full-model tests with $d = 0.1-2$ mm and found that larger sediment sizes (and therefore reduced cohesive strength) and small friction angles lead to faster breach processes, and the peak outflow, final breach width and depth were increased. No drainage or impermeable layer were employed though, which might lead to a breach influenced by seepage (van Emelen 2014). Using generic programming and their tests as calibration, they found that the sediment size and cohesion effects were not predominant, because the reservoir water volume and the water depth play a much more important role in the dike breach.

The diagrams of the fluvial large-scale experiments by Kakinuma and Shimitzu (2014) indicate that apparent cohesion becomes evident for small $d \approx 0.2$ mm and reduces the erosion rate in the breach, while the erosion rate for coarser sediment $d > 0.2$ mm is independent of $d$. Battarai et al. (2015) conducted 3D dike breach tests with parallel approach flow and constant inflow discharge using different sediment grain sizes without drainage or impermeable layer. They found that for $d = 0.089$ to $0.314$ mm, the finer materials are
more vulnerable to scour downstream of the breach. Furthermore, it was found that the dike breach process accelerated for coarser sediment due to seepage.

Cestero et al. (2015) conducted 3D dike breach tests for constant headwater level conditions using sediment of size $d_{50} = 0.159$ to $0.628$ mm with different proportions of sand, silt and clay. The increase of the percentage of finer sediment in the mixture and a higher compressive strength resulted in a slower breach process, reduced peak breach discharge $Q_M$ and a smaller final breach width. Critical shear stress was found to be relatively insensitive to the soil composition. The breach process was therefore found to be strongly dependent on soil properties (mainly unconfined compressive strength but also dry unit weight and moisture content). Rüdisser (2016) conducted 3D dike breach tests with sediment of $d \approx 1$ to $4$ mm and falling reservoir level finding that sediment erosion and peak breach discharge $Q_M$ increase for large $d$, while time-to-peak $t_M$ decrease for large $d$. Walder et al. (2015) compared breach discharges of 3D dike breach tests using sediment of $d = 0.21$ mm with the results of Coleman et al. (2002) for sediment of $d_{50} = 0.5$-2.4 mm, stating that larger $d$ lead to a slight, almost negligible, increase in $Q_b$. Furthermore, they found that the retreating headcut maintained a slope near the angle of repose.

In summary, the results indicate that the 3D dike breach process for dikes built of non-cohesive sediment is slightly faster for larger sediment sizes, and that breach discharge, breach widening and erosion rate increase for larger sediment sizes. The inclusion of a larger percentage of fine (cohesive) sediment opposes this trend. This effect can be attributed to apparent cohesion, and promotes a tensile strength approach described by Pickert et al. (2011). The breach process is therefore closely related to the soil-moisture retention curve of Kovacs (1981), as has been indicated by e.g. Al-Riffai (2014) or Volz et al. (2016). As observed e.g. by Hakimzadeh et al. (2013), however, the effects of sediment size and cohesion on the breach process are not predominant for non-cohesive sediment, as opposed to reservoir water volume and water depth.

### 2.8.4 Sediment compaction

Sametz (1981) and Simmler and Sametz (1982) found for 2D dike breach model tests with $d_{50} = 0.5$-15 mm that compaction plays a negligible role. Winz (2012) conducted 2D dike breach tests and found that compaction of non-uniform sediment of $d = 2.5$ mm slightly delays dike erosion at breach start for small inflow discharge as compared to non-compact ed dikes. For larger inflow discharges and / or longer breach times, the difference
is negligible. For the same setup, 2D dike breach tests of Müller (2015) indicated that sediment of \( d = 1.75 \) mm not subjected to any compaction and almost no formwork during dike setup will lead to a faster initial erosion as compared with dikes with slight compaction and formwork.

Al-Riffai (2014) found for 3D dike breach tests with \( d_{50} = 0.17 \) to 0.245 mm that the peak breach discharge \( Q_M \) was reduced by 16% for the highest compaction effort as compared to the lowest compaction effort, while the corresponding decrease in time-to-peak \( t_M \) was 28%. Furthermore, undercutting and side-slope failure frequency in the breach channel were reduced for dikes built with a high compaction effort. Al-Riffai also conducted 2D tests with different initial soil-water states by impounding the headwater level above the dike crest using a Plexiglas release gate and found that the initial soil-water state has an effect on non-cohesive sediment erosion. Amaral et al. (2014) conducted two spatial dike breach tests using widely graded material of median grain size \( d_{50} = 0.3 \) mm (silty sand) finding that a higher degree of compaction influences breach discharge and time.

Dikes built of cohesive sediment involve different breach characteristics. For dike breach tests using cohesive material, Zhu (2006) found that headcut erosion dominates the breach process and that soil compaction plays a major role in the breach erosion rate, even more important than the proportion of clay in the soil material. Hanson and Hunt (2007) found that soil texture and plasticity, together with the compaction water content and compaction effort play a major role for sediment erodibility.

In summary, these results indicate that the effect of compaction on the breach process for dikes built of non-cohesive sediment is negligible for larger grain sizes and only becomes important for sediment grain sizes \( d \approx 0.2 \) mm and smaller. The breach process for small sediment grain sizes and cohesive sediment is dominated by the geotechnical properties of compaction water content and compaction effort, which is not the focus of this study.

2.8.5 Dike geometry and channel bed

Sametz (1981) conducted 2D dike breach tests with constant headwater level and found a similar absolute erosion rate for different dike surface slopes, although the erosive development was found to depend on the stabilizing sediment body downstream of the impermeable layer. The breach process was found to be faster and peak breach discharge higher for steeper dike surface slopes. Lüthi (2005) found no effect of the dike height \( w \), while the breach process was found to accelerate for steep downstream dike surface slopes.
Schmocker (2011) found that the 2D dike breach process is accelerated for small $w$. Müller et al. (2016) found for 2D dike breach tests that the breach process is slow for large dimensionless dike shape parameters $\mu$, which describe the dike cross-section when multiplied with $(7/3)w^2$. According to the experimental observations of 2D dike breaches by Schmocker and Hager (2012a) and Müller et al. (2016), a pivot point of the dike surface profiles is located in the reach of the dike toe above the channel floor. This behavior was also noted by Dupont et al. (2007), Schmocker and Hager (2009), Schmocker et al. (2014) and van Emelen et al. (2015). The latter additionally studied the effect of a sand layer below the dike for 2D dike breach tests. The breach profiles indicate that the pivot point is higher and further upstream and that the maximum dike height is higher for larger test times as compared to tests without a downstream sand layer.

Visser (1998) presents a general overview on the effect of different channel beds and protective measures on the continuation of breach growth in Stage IV. After the complete wash-out of the breach channel in Stage III, the breach continues to grow laterally in Stage IV and, depending on the dike subsoil, also in vertical direction, leading to a scour hole. Chinnarasri et al. (2004) found for 3D dike breach tests that the peak outflow increases for steep downstream embankment slopes. Islam (2012) conducted fluvial 3D dike breach tests with different river bed heights as compared to the bed level downstream of the dike and found increased breach widening and sediment erosion for higher river bed heights.

Different pilot channel geometries and breach initiation methods (V-notch, flat-crest, blast-induced crater) were tested by Orendorff et al. (2013) for 3D dike breach tests. No effect on the peak breach discharge $Q_M$ or the breach widening was found, only the time needed for breach initiation was affected. Amaral et al. (2014) found no effect of the pilot channel geometry on the breach discharge hydrographs.

In summary, the dike cross-section has a significant effect on the breach process, while the downstream boundary condition (sediment layer, channel bed slope) influences peak breach discharge and the pivot point of the 2D breach profiles. For fluvial dike breach tests, the river bed height influences the breach process, while the influence of the initial pilot channel shape is only noticed during breach initiation, but not in the general breach process.
2.8.6 Dimensionless time

Depending on the studied parameters, different dimensionless times were introduced by various authors to describe the breach process. Lüthi (2005) introduced a dimensionless time for 2D dike breach tests \( T_{2D} = t g^{1/2} h_c^{-1/2} \), with \( t \) as time, \( g \) as gravity acceleration and \( h_c \) as critical water depth for a defined inflow discharge \( Q_o \). Pickert et al. (2011) conducted a dimensional data analysis and introduced a dimensionless time \( t^* = t g^{1/2} h_{cap}^{-1/2} \) to describe the dimensionless evolution of the breach discharge for constant headwater conditions and apparent cohesion controlled unsaturated sediment \((d \leq 1 \text{ mm})\), with \( h_{cap} \) as capillary rise (Fredlund and Rahardjo 1993). Schmocker (2011) introduced the dimensionless time \( T_{2D} = t g^{1/2} h_c d^{-1/2} W^{-1} \) to describe the 2D dike breach process, with \( g' \) as submerged specific gravity, as also described in Chapter 6. The breach process is accelerated for larger inflow discharge and smaller sediment size and dike heights. Al-Riffai (2014) scaled the time-to-peak of a tilted (distorted) geometric and a quasi-exact scale test series of 3D dike breaches using Froude similitude with satisfactory results (sediment size not scaled).

In summary, the temporal evolution of the breach process was described using distorted Froude scale models or dimensionless times developed from one or few studied parameters. According to Pickert et al. (2011), the breach process is decelerated for small sediment sizes due to the large capillary height, while according to the dimensionless time of Schmocker (2011), the breach process is accelerated for small sediment sizes. As a conclusion, no generalized dimensionless time including the main driving and retaining forces has been developed so far from laboratory models.

2.9 Sediment entrainment and scalability of dike breach models

Schmocker (2011) describes the flow hydraulics, incipient motion of a single sediment grain, the Shields criterion, the effect of bed slope on sediment entrainment, and the bedload sediment transport formulae. These will therefore not be repeated here and only the relevant equations are introduced as a reference.

While erodibility in cohesive soils is dominated by cohesive forces, the Shields parameter is one of the best known and most commonly used stability criteria for non-cohesive particles under uniform flow conditions. Based on the diagram of Shields (1936), van Rijn (1984) plotted Figure 2.6 with the critical mobility parameter \( \theta_{cr} \) [-] as a function of the particle parameter \( D^* \) [-], the former defined as
Here \( u_{cr} \) is critical bed-shear velocity according to Shields (1936), \( g' = g(\rho_s - \rho) / \rho \) the submerged specific gravity [m/s²] with \( g \) = acceleration of gravity [m/s²], \( \rho_s \) = solid particle density [kg/m³], \( \rho \) = fluid density [kg/m³], and \( d_{50} \) [m] = median sediment grain size. The non-dimensional particle parameter is defined as

\[
D_* = d_{50} \left( \frac{g'}{\nu^2} \right)^{\frac{1}{3}} \approx \frac{d_{50}}{d_R}
\]  

with \( \nu \) [m²/s] as kinematic viscosity (here for \( T \approx 16^\circ \)) and \( d_R = 1/22,600 = 4.4 \times 10^{-5} \) m is a reference diameter. While \( D_* \) was introduced by van Rijn (1984) and Shields (1936) to describe sediment entrainment (motion is initiated faster for small \( D_* \)) for uniform sediment and parallel-streamlined flow on a near horizontal bed, it describes in this study the retaining force of apparent cohesion, which is larger for small \( D_* \). This linear relation is valid if the capillary force, characterized by the capillary height, is linearly proportional to surface tension \( \sigma \) and inversely proportional to the horizontal pore size (Kovacs 1981), which is proportional to \( d_{50} \).

It is often assumed that model dike breach tests can be scaled to prototype dike breaches by applying Froude similitude, see e.g. Chinnarasri et al. (2004) or Hakimzadeh et al. (2013). The issue of scaling is therefore treated hereafter.

**Figure 2.6** Sediment entrainment according to Shields (1936) (adapted from van Rijn 1984)
As described by Schmocker (2011), the important parameters for hydraulic modeling are dimensionless ratios formed with respect to inertia. An open-channel model is normally operated according to the ratio of inertia and gravity forces, represented by the Froude number $F$ as

$$\text{F} = \frac{v}{\sqrt{gL}}$$

(2.6)

Here $v = \text{flown velocity}$, $g = \text{gravity acceleration}$, and $L = \text{length scale}$. The geometric scale ratio is defined as $\lambda = L_m/L_p$ where subscripts $m$ and $p$ refer to model and prototype, respectively. To respect Froude similitude, $\lambda_F = 1$ is required, with $\lambda_F$ as the scale factor for the Froude number. The Froude scale relations are shown in Table 2.3.

<table>
<thead>
<tr>
<th>Characteristics</th>
<th>Scale factor $\lambda = L_m/L_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length [m]</td>
<td>$\lambda$</td>
</tr>
<tr>
<td>Area [m²]</td>
<td>$\lambda^2$</td>
</tr>
<tr>
<td>Volume [m³]</td>
<td>$\lambda^3$</td>
</tr>
<tr>
<td>Time [s]</td>
<td>$\lambda^{1/2}$</td>
</tr>
<tr>
<td>Velocity [m/s]</td>
<td>$\lambda^{1/2}$</td>
</tr>
<tr>
<td>Acceleration [m/s²]</td>
<td>1</td>
</tr>
<tr>
<td>Force [N]</td>
<td>$\lambda^3$</td>
</tr>
<tr>
<td>Discharge [m³/s]</td>
<td>$\lambda^{5/2}$</td>
</tr>
</tbody>
</table>

Ranga Raju et al. (1990) studied the effects of viscosity and surface tension on embankment weir flow. They identified the characteristic parameter $\Phi = R^{0.2}W^{0.6}$ in which $R = (gh)^{1/2}v^{-1}$ is the Reynolds number; $W = \rho g h^2 \sigma^{-1}$ is the Weber number; $v$ is the fluid viscosity; $\rho$ is fluid density; and $\sigma$ is surface tension. For $\Phi > 10^3$, scale effects attributed to surface tension and viscosity are absent, almost independently of weir face angle, weir height, and discharge.

Figure 2.7 shows the settling velocity $v_s$ of sand and silt in still water as a function of its grain size $d$ (Pugh 1985, 2008). The diagram highlights that small particles $d \leq 1$ mm settle at slower velocities than according to Froude scaling as the particles become smaller. For particle diameters $d > 1$ mm, $v_s$ varies with $d^{1/2}$, which is in accordance with Froude scaling for velocity $v \sim L^{1/2}$. The settling velocity of small sediment can be compensated to the proper value for Froude scaling by increasing the size of the model sediment, as elaborated by Pugh (1985, 2008).
Tinney and Hsu (1961) state that tests using fine sediment for modelling dike breaches yield only indicative results for scaling, whereas they refer to large-scale models with large particle sizes by Straub (1953), where model scaling was quite accurate. Tinney and Hsu therefore conducted 3D dike breach model tests using dikes of height $w = 0.2$ and $0.4$ m with a clay core and using the same particle size as in the prototype. They found for constant headwater conditions that the washout rate is slightly larger for small sediment as compared to large. They also state that for a scale ratio of 1:2 between two models built of the same material, the larger will have a $2^{1/3}$ times faster side erosion.

Dunglas and Fayoux (1971) conducted laboratory scale tests for a prototype of a dike 12 m high using models of height $w = 0.3$ and 4 m and sediment of size $d_{60} = 0.09$ to 0.18 mm. They were able to describe the dike breach process and found a satisfactory scalability for model scales of 1:10 to 1:5. Zarn (1992) found that for scaling the critical bed-shear velocity in river engineering models according to Froude similitude, the sediment grain sizes $d = 0.22$ to 4 mm need to be coarsened. Also, grain sizes $d < 0.22$ mm should be removed from the sediment mixture since these obey to different scaling laws.

Schmocker and Hager (2009) found for three 2D dike breach scale family tests that no major scale effects are present if sliding failure is avoided. The reference dike $\lambda = 1$ of height $w = 0.4$ m was influenced by seepage at breach start due to the drainage position and its discharge capacity, but showed satisfactory agreement with the smaller test dikes $\lambda = 0.25$-0.5 at larger test times. They concluded that sediment size should be limited to 1 mm to avoid cohesive effects and apply Froude similitude. Minor side wall effects were

**Figure 2.7** Particle settling velocity $v_s$ versus particle size $d$ (adapted from Pugh 1985, 2008)
found for channel widths of 0.1 m, while neither a small model dike height \( w = 0.1 \) m nor small inflow discharge 1 l/s had a significant effect on the dike erosion profiles. Froude numbers were found to be smaller for \( \lambda = 0.25 \) as compared with \( \lambda = 1 \). As noted by Al-Riffai (2014), the scale family for the largest grain size range (e.g. 2, 4, 8 mm versus the two other scale families using 1, 2, 4 mm) yielded the largest variability in breach profiles due to the governance of sliding failures observed in the downstream slope for larger sediments due to the immediate saturation. Schmocker (2011) therefore reduced the seepage effect by limiting the sediment grain size to \( d < 5.5 \) mm and inflow discharge \( Q_o > 2 \) l/s. Variations in the dike profiles decreased for larger inflows and larger times.

Al-Riffai (2014) found that scaling model tests using dikes of height \( w = 0.3 \) m and tilted (distorted) geometric scales (dike surface slopes \( S = 1:1, 1:2, 1:3 \) resulted in good agreement for the peak breach discharge \( Q_M \) and the corresponding time \( t_M \) (sediment size not scaled). Tilted geometric scales are used to improve the Reynolds number similarity and water depth measurement accuracy (Chanson 2004a). The disadvantage of this method is that lateral and vertical time scales in the breach channel evolution become irrelevant (Julien 2002). For the quasi-exact geometric scale series, Al-Riffai (2014) found that \( t_M \) was also well-scaled, while \( Q_M \) was not. Al-Riffai recommends to conduct scale effect tests using larger scale ratios including scaled sediment grain sizes.

Studies with scaled clay core thicknesses were conducted by Tinney and Hsu (1961), Pugh (1985) and Schmocker et al. (2013). Peñuela (2013) conducted spatial dike breach tests in a geotechnical centrifuge facility to increase gravity to 33-g and simulate the stresses in the soil mass correctly for a scale model of 1:33. The centrifugal modelling of slope failures, seepage and surface flow was described by Goodings (1979, 1982).

In summary, effects due to settling issues can be expected when scaling sediment of grain size \( d < 1 \) mm according to Froude. While there are indications that dike breach models can be scaled according to Froude, the results are partially inconsistent and not systematically described. Contradictions in the results are partially attributed to issues with the test setup such as seepage. For 2D dike breaches Froude similitude can be assumed, although Schmocker (2011) found that the breaches were influenced by seepage. The most reliable tests indicating Froude scalability for 3D dike breach tests were distorted Froude scale model tests by Al-Riffai (2014), for which only specific properties are properly scaled though. As a conclusion, a general verification of Froude similitude for 3D dike breach models has not been conducted so far.
2.10 Research gaps and purpose of this study

This literature review reveals that much research has been conducted to describe the spatial dike breach by different approaches. Although several essential features during dike breaching have been assessed separately, no model combining all parameters tested in the laboratories has been developed so far. This research contributes to the most important gaps in laboratory dike breach testing, namely the

- weighting of parameters involved in the breach process,
- limit of cohesive and non-cohesive sediment, and
- model scaling.

For this purpose, systematic spatial dike breach model tests were conducted for dikes built of non-cohesive sediment. The dikes were progressively eroded by overtopping for two hydraulic boundary conditions: (1) constant inflow discharge, and (2) different reservoir sizes but no inflow discharge (falling water level). The new dike breach channel setup allowed for successful dike breach tests. Using advanced data acquisition methods (i.e. a novel photogrammetric system) and innovative test procedure and evaluation approaches (e.g. increased reservoir size by simulation using a linear-quadratic-Gaussian (LQG) controller for pump regulation, see also 3.5), the dike breach process is described by combining the topographic data, the hydrographs and the hydraulic properties. Empirical spatial dike breach formulations are developed from these laboratory tests, which are a useful tool to predict dike breach properties for emergency situations or risk assessments, to complement and crosscheck empirical or parametric embankment models, or to design erodible dikes for flood protection.
3 Experimental setup and procedure

3.1 Introduction

The Dike Breach Channel was used at two different locations because the VAW laboratory moved to its new site in 2013. The setup at the former location was employed to determine the accuracy of the photogrammetric system (Frank and Hager 2014). In addition to 80 preliminary 2D dike breach tests (2.2), 35 preliminary 3D tests improved the planning of the channel setup presented by Frank and Hager (2015, 2016c) and are described in detail hereafter, including the measuring devices, the test and data analysis procedures and the test program. 45 tests conducted with the improved setup are detailed below.

3.2 Laboratory channel

The dike breach experiments were conducted in a channel 6.5 m long, 1 m wide and 1 m deep, with an inlet length of 2.4 m and a total inner flume length of 11.9 m (Figure 3.1, Figure 3.2). Its front sidewall consists of a 40 mm thick glass wall and its remaining parts are made of steel. A raised PVC floor was inserted both to add a bottom drainage at a distance of 4.44 m from the inlet wall and to facilitate experimentation. The usable channel dimensions were hereby reduced to 5.1 m in length, 1 m in width and 0.6 m in height.

Figure 3.1 Dike Breach Channel setup with (a) side view and (b) top view with, ① pump unit, ② inlet, ③ raised PVC floor, ④ dike, ⑤ drainage, ⑥ sediment trap, ⑦ outlet, ⑧ movable traverse, ⑨ Inductive Discharge Measurement (IDM)
The channel is used as a recirculating system, pumping the water from the outlet to the inlet. The limited water volume in the downstream reservoir was compensated with additional water volume for the tests using large dikes of 0.6 m height to avoid air entering the inlet pipe, as shown in 4.3. Given the smooth channel surface, flows were assumed to be in the turbulent smooth regime.

Note the light-impermeable enclosure over the entire test facility in Figure 3.2, which is useful for dust protection and to improve the contrast for the photogrammetric measurement system detailed below. To allow for simple sediment management including the dike set-up and the drying of sediment after a test, the tent is easily opened from the side. Furthermore, test quality was noticeably improved through the automation of the pump set and the installation of a redesigned raised PVC floor, a new drainage system of sufficient discharge capacity coupled with discharge measurement, a redesigned inlet and a flexible lighting system. The test performance was significantly enhanced by a redesigned work platform and an improved crane positioning, harmonizing well with the movable traverse for the photogrammetric system.

Figure 3.2 Photos of Dike Breach Channel with (a) ① pump unit, ② inlet, ③ raised PVC floor, ④ drainage and measurement basin, ⑤ outlet, ⑥ dike, ⑦ grid projector and cameras on movable traverse, ⑧ crane, ⑨ supporting structure, ⑩ computer, (b) ① light-impermeable enclosure, ② sediment trap
3.3 Inflow and breach discharge

The channel pump set is automated using the computer and a frequency converter, pumping the water in a closed system from the outlet to the inlet with a maximum discharge capacity of slightly less than 100 l/s. The supply pipe of 250 mm diameter was locally reduced to 150 mm to achieve a ±0.6% accuracy of the inflow discharge measurement using the IDM. For small inflow discharge \( Q_o < 6 \, \text{l/s} \), a mobile metering pump of maximum discharge capacity of 7 l/s was used, increasing the discharge accuracy.

The temporal breach discharge \( Q_b(t) \) was determined applying the reservoir storage equation as used by Coleman et al. (2002), Cestero et al. (2014) or Frank and Hager (2015) as

\[
Q_b(t) = Q_o(t) + Q_R(t) - Q_{dr}(t) = Q_o(t) - \frac{\Delta h_o(t)}{\Delta t} A_R(t) - Q_{dr}(t) \tag{3.1}
\]

Herein the inflow discharge is \( Q_o(t) \), the discharge from water stored in the reservoir \( Q_R(t) \), the drainage discharge \( Q_{dr}(t) \) as presented in 3.4, the headwater level \( h_o(t) \) and the instantaneous reservoir water surface area \( A_R(t) \) (Figure 3.3).

![Figure 3.3 Streamwise section of Dike Breach Channel with parameters necessary to determine breach discharge \( Q_b \) applying the reservoir storage equation (Eq. 3.1)](image)

The values of \( h_o \) and the water level in the drainage measurement basin were determined using ultrasonic probes, which measured the distance from the probe to the water surface level \( h_{US} \) at 10 Hz with a measuring accuracy <0.5 mm and a range of 100 to 700 mm. The quality of the hydrographs was improved as compared to Frank and Hager (2015) by applying a Savitzky-Golay (1964) filter to the measured data and cross-checking the filtered and raw data as shown in Figure 3.4. The applied method proves to be repeatable, reliable, sensitive to breach process changes, and has a high temporal resolution compared to other methods previously applied (Figure 3.5), as also described by Frank and Hager (2016b). In combination with visual observations and topographic measurements, it is possible to adequately interpret and cross-check the acquired data and thus avoid errors.
Figure 3.4  Qualitative comparison of data (−) as measured with a frequency of 10 Hz and (−−−−−−) filtered using the Savitzky and Golay (1964) method for (a) ultrasonic probe data of headwater level $h_{US}(t)$, (b) reservoir discharge $Q_R(t)$ using the software MatLab.

Figure 3.5  Breach discharge $Q_b(t)$ for plane dike breach tests. Results of Schmocker and Hager (2012b) (——) with and (−−−−−−−−−) without manual correction, (−−−−−−−−−−−−) (−−−−) and (−−−−) applying Eq. (3.1) (Frank and Hager 2016b).

For the planning of the test program in 3.9, the drainage design discharge $Q_{dr,d}$ resulting from seepage was estimated during pre-tests by impounding dikes of initial crest heights $150 \text{ mm} \leq w \leq 600 \text{ mm}$ and sediment of grain sizes $0.43 \text{ mm} \leq d \leq 3.78 \text{ mm}$. Expected values of $Q_{dr,d}/Q_o$ were in the range of 4 to 29%, which highlights the importance of the choice of the relevant discharge to compare results. In accordance with Coleman et al. (2002), the breach discharge $Q_b(t)$ was therefore selected as the relevant discharge for comparison of results, given by Eq. (3.1). Accordingly, a design breach discharge $Q_{b,d}$ was determined for each test, taking $Q_{dr,d}$ into account.
3.4 Bottom drainage

The goal of this research project is to study dike breaches due to overtopping and subjected to sediment erosion. To prevent uncontrolled dike failure by seepage, the seepage line is lowered at the downstream dike toe by installing a 0.21 m long drainage across the entire channel, below the center of the downstream dike face. The drainage is equipped with a fine wire net of mesh size 0.2 mm to avoid sediment washout. The wire net was held by two perforated sheets and fixed to a T-shaped PVC container placed on the channel floor (Figure 3.6). This drainage was connected through tubes to a drainage measuring basin of 1 m³ volume with a base size of 1.82×0.92 m² and a height of 0.6 m. The water level was monitored using an ultrasonic probe to determine the drainage discharge $Q_{dr}$.

The maximum drainage discharge capacity was around 10 l/s for large dikes. When the drainage basin is filled, excess water escapes through an overflow so that the drainage measurement is no longer possible. The necessary streamwise length of the drainage depends on the dike dimensions and sediment grain size. For dikes of height $w = 0.6$ m, the drainage length $L_{dr}$ was 0.21 m; for $w = 0.3$ m, $L_{dr} = 0.1$ m; for $w = 0.15$ m, $L_{dr} = 0.05$ m. For Tests 1 to 3 with $w = 0.2$ m (see 3.9), the drainage length was larger than necessary with $L_{dr} = 0.21$ m. With this setup, it was necessary to assure aeration in the T-shaped PVC container to avoid fluctuations of the drainage and seepage flows. Figure 3.7 shows a preliminary test for $w = 0.6$ m, for which the drainage was inadequately positioned, so that the downstream dike surface slope became unstable due to seepage outflow. The optimizations are presented in 4.2 leading to the optimum positioning at $x_{dr} = S_{ow} + L_K + 0.5S_{dw}$, below the center of the downstream dike face slope.

![Figure 3.6](image_url)

**Figure 3.6** Drainage details: (a) front view of T-shaped PVC container before raised floor was installed, (b) (---) seepage line after dike breach test, with inflow discharge $Q_o$ from left to right and drainage discharge $Q_d$ through perforated sheet visible on bottom
3.5 Hydraulic boundary conditions

The scale test and breach process series of Table 3.2 and Table 3.3 were conducted using as hydraulic boundary condition constant inflow discharge $Q_o$, while the additional test series of Table 3.4 were conducted using different reservoir water surface areas $A_R$ and $Q_o = 0$. The tests were conducted as follows:

For the constant inflow discharge mode with $Q_o = \text{constant}$, a pump regulation with a proportional-integral-derivative (PID) controller and one input signal (inflow discharge) commonly used in industrial control systems was used to regulate the pump’s rotational speed and to reach a defined inflow discharge. The design drainage discharge $Q_{dr,d}$ was predicted in preliminary tests and added to the desired design breach discharge $Q_{b,d}$ for the inflow discharge ($Q_o = Q_{b,d} + Q_{dr,d}$). The constant inflow discharge is reached few seconds after test start and the headwater level rises until after the dike breach is initiated, dropping again thereafter. Breach discharge augments fast after breach start, reaching peak breach discharge, and then drops to $Q_{b,d}$. The hydraulic boundary condition of constant inflow discharge typically represents a situation when e.g. a landslide-induced dam suddenly blocks a river, leading to an upstream impoundment, until water overflows the lowest dike elevation initiating the breach, followed by a rapid reservoir drawdown (see also Test case in 6.8.1).

The horizontal reservoir water surface area $A_R$ has an important effect on the spatial dike breach process. The laboratory channel is physically limited in size and applies only for small $A_R$ with a rapid reservoir drawdown. Many research projects consider instead a constant upstream reservoir elevation, corresponding to large water bodies $A_R \rightarrow \infty$. These conditions are achieved by different techniques of pump regulation. For intermediate conditions though, the water level will drop with a velocity depending on $A_R$. These conditions have been modelled physically by different authors for relatively small sizes of
AR < 25 m², due to the considerable laboratory space needed for these tests. In the Dike Breach Channel, these intermediate AR conditions were achieved using a linear-quadratic-Gaussian (LQG) controller with reference and disturbance feedforward, which is a model-based reference tracking controller with several input signals (inflow discharge and headwater level), to add a simulated water surface area AR, simulated to the physical water surface area AR, physical in the channel by regulating the pump rotational speed, as shown in Figure 3.8. It is then theoretically possible to simulate reservoirs from AR = 0 to AR → ∞, while compensating the drainage discharge and thereby improving the accuracy. Furthermore, it is possible to simulate reservoirs of different shapes as described by Kühne (1978), e.g. a triangular longitudinal shape as shown in Figure 3.8b (see also 2.8.2). Another important advantage of this reservoir simulation method compared to physical reservoirs is the high level of control over the breach initiation conditions, since the velocity of headwater level rise at breach start is crucial for test comparison. Therefore, when simulating AR, deviations at breach start are reduced, so that the specific effects investigated can be better distinguished. In physical reservoirs, breach initiation conditions are prone to variations and depend on AR, leading to large deviations in the temporal evolution of the breach. In this research project, the headwater level was raised by 1 mm/s to 20 mm above the initial pilot channel used for breach initiation. The timing proved to be excellent and led to satisfactory results (Chapter 5). The channel width was a limiting factor for the breach widening, so that only half-model tests with AR < 103.44 m² (full-model AR < 206.88 m²) satisfied conditions for the determination of the breach characteristics. For larger AR, a larger channel width and a higher discharge capacity need to be considered.

**Figure 3.8** Physical reservoir (light blue) increased in size by simulation using controller (dark blue) for (a) rectangular, (b) triangular longitudinal reservoir shape
3.6 Sediment

The dikes were built using uniform sediment of grain size $d_{50} \text{[mm]} = 0.43, 0.86, 1.25, 1.75, 3.78$ with a geometric standard deviation of grain size distribution $\sigma_g = (d_{84}/d_{16})^{1/2} < 1.2$. The sediment grain size distribution, the tilt angle and the angle of repose were determined for the different grain sizes (Figure 3.9). The characteristic sediment characteristics are presented in Table 3.1. The sediment grains are estimated to have a sphericity of 0.7 and a roundness of 0.3 according to Krumbein and Sloss (1963), which is in the range of most manufactured crushed sands according to Cho et al. (2006). While it was assumed that cohesive and viscous effects result from sediment with $d < 0.86 \text{ mm}$ for 2D dike breach tests (Schmocker 2011), the results of Chapters 4, 5 and 6 indicate that the sediment transport process does not markedly change for $0.43 \text{ mm} \leq d \leq 3.78 \text{ mm}$ for spatial dike breaches. Furthermore, the good agreement of the tests by Walder et al. (2015), who used sediment with $d = 0.21 \text{ mm}$, with the equations developed in Chapter 6 indicates that no major effects are to be expected for $0.21 \text{ mm} \leq d \leq 3.78 \text{ mm}$, while the lower and upper limit to be determined.

![Figure 3.9](image)

**Figure 3.9** Test facilities for characteristic sediment properties with (a) shaker, (b) tilting table, (c,d) glass cylinder and rough surface for repose angle test

**Table 3.1** Characteristic sediment properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>sediment diameter $d_{50}$ [mm]</td>
<td>0.43-3.78</td>
</tr>
<tr>
<td>geometric standard deviation of grain size distribution $\sigma_g = (d_{84}/d_{16})^{1/2}$</td>
<td>$&lt;1.2$</td>
</tr>
<tr>
<td>density $\rho_s$ [g/dm³] (mean value)</td>
<td>2600</td>
</tr>
<tr>
<td>bulk density $\rho_b$ loose [g/dm³]</td>
<td>1395</td>
</tr>
<tr>
<td>bulk density $\rho_b$ compacted [g/dm³]</td>
<td>1495</td>
</tr>
<tr>
<td>porosity $\varepsilon$ loose [-]</td>
<td>0.47</td>
</tr>
<tr>
<td>porosity $\varepsilon$ compacted [-]</td>
<td>0.41</td>
</tr>
<tr>
<td>repose angle $\Psi$ [-] (mean value)</td>
<td>39.5</td>
</tr>
<tr>
<td>tilt angle $\Phi$ loose [°] (mean value)</td>
<td>38.0</td>
</tr>
<tr>
<td>tilt angle $\Phi$ compacted [°] (mean value)</td>
<td>41.0</td>
</tr>
</tbody>
</table>
3.7 Test dike

For the spatial dike breach tests, a trapezoidal dike was inserted into the 1 m wide Dike Breach Channel at distance $x_D$ from the channel intake to the upstream dike toe. The origin of the coordinate system $(x,y,z)$ is located at the orographic right glass wall of the upstream dike toe. The setup of a spatial dike breach test is shown in Figure 3.10. First, 5 mm thick guiding templates of dike shape are fixed on each channel side. Sediment was then placed at the dike center. After preliminary dike construction, the excess sediment was screened using the guiding templates. After their removal, missing sediment was added so that the final dike model is geometrically accurate to roughly 1 to 2$d$ or 2 mm. The shape of the pilot channel was prepared by removing sediment to tentatively create the pilot channel shape, which was then brought to its final shape by repeatedly moving a template along the $x$-axis and carefully removing the excess sediment.

The method described for setting up the model dike resulted in reliable accuracy. Note that only slight compaction and a completely dry sediment was used to allow for the final dike shape. The sediment characteristics are therefore estimated to be close to the values of slightly compacted/loose sediment. The temporal effort for dike set-up was 2 h for the 0.20 m high dike, whereas it amounted to 6 h for $w = 0.60$ m. The test duration for either of these two was less than 0.5 h from start to stop of water flow, so that this duration is much less than that for model dike preparation. Note that even more time was employed for channel adaptations (e.g. two weeks for breach tests with dikes 600 mm high), or sediment preparation and drying, so that a typical test including all work steps easily lasted from 0.5 to 2 days. Note also that the data processing and data analysis last significantly longer than the laboratory work, as is typical in modern hydraulic experimentation.

![Figure 3.10](image_url)  
*Figure 3.10* Steps for preparation of spatial dike breach test (a) installation of guiding templates and filling with sediment, (b) screeding of excess sediment, (c) final pilot channel shape with template, (d) finished dike with projected grid pattern
Experimental setup and procedure

Figure 3.11  Modified Dike Breach Channel for scaled test dikes of height (a) $w = 0.3$ m. Tests with $w = (b,c) 0.6$ m, (d,e) 0.3 m, with corresponding scales $\lambda = 1, 0.5$

For the scale effect tests presented in Chapter 4, the channel width was reduced using a mobile PVC wall structure as shown in Figure 3.11a. Since the previously described drainage was positioned at a distance of 4.44 m from the inlet wall, the upstream reservoir water surface area $A_R$ was downscaled by installing a wooden overflow weir at distance $x_D$ from the upstream dike toe, followed by a filter mat to reduce turbulences. The downstream platform length $L_f$ was not scaled, since the raised PVC floor was a permanent installation and could not be adapted. $L_f$ was assumed to be long enough in the scaled breach tests for a comparable stabilizing body to form downstream of the breach, so that its effect on the breach process was assumed negligible.

The schematic dike including the main dike characteristics for a half-model test is shown for the initial setup in Figure 3.12a,b and after breach start in Figure 3.12c,d,e. The main features of the initial dike consist of the dike height $w$ and length $L_D$, the crest length $L_K$, the pilot channel notch height $w_p$ and top width $b_p$, its sediment height at the glass wall $w_{p,0}$, the channel width $b$, and the origin of the coordinate system at the upstream dike toe at the orographic right glass wall. After breach start, a breach channel forms, leading to the typical hourglass shape in Figure 3.12c-e. The main breach characteristics are described with the headwater level $h_0$, the maximum dike height along the central breach axis at the glass wall $z_M$, the water level at each profile $h$, the breach profile height $z_b$, the water depth $h_b$ and the water surface width $b_b$. The hourglass shape is described with the smallest water surface width $y_{\text{min}}$, the largest water surface width $y_{\text{max}}$, and the difference between the latter two $y_{\text{hg,cl}} = y_{\text{max}} - y_{\text{min}}$, while the hourglass shape is defined as $y_{\text{hg}} = b_b - y_{\text{min}}$. 
Figure 3.12 Definition sketch of spatial dike breach test with (a) longitudinal, (b) transverse sections of initial setup, and (c) longitudinal, (d) transverse sections and (e) plan view after breach start
3.8 Photogrammetric system

The breach surface topography was recorded with the stereoscopic-videometric commercial measurement system of AICON 3D Systems Ltd., Braunschweig, Germany, which was developed for the German Federal Waterways Engineering and Research Institute (BAW) (Godding et al. 2003). BAW and Henning et al. (2007) used this system for riverbed measurements below quasi-horizontal water surfaces. Preliminary tests with this system for spatial dike breaches involving curved water surfaces were conducted by Schmocker and Hager (2011). The system was then complemented for the spatial dike breach tests at VAW with a fourth camera to record the flow depth along the channel side wall (Figure 3.13, Figure 3.14). Modifications had to be implemented in the AICON software to accurately evaluate the data in the submerged breach areas, resulting in the first entirely successful and accurate 2D and 3D dike breach measurement evaluations by Frank and Hager (2014, 2015) using the AICON software, consisting of:

- AICON MoveInspect® (version 5.18.24, April 2013) for data acquisition with the four cameras, and
- AICON ProSurf® (version 8.53.1, August 2013) for system calibration, data evaluation, and visualization.

Figure 3.13  Setup of photogrammetric system for spatial dike breach tests
The software was continuously improved up to the current versions (MoveInspect® version 6.15.16), ProSurf® (version 8.60.1) to allow for exporting the water surface profiles, which was essential to evaluate Test 10, or the possibility to limit the area of interest for data evaluation, which was helpful for Tests 33 to 35 (see Table 3.2 and Table 3.3). In summary, the AICON system was fully functional after implementing software and hardware improvements and some 35 preliminary tests. Further photogrammetric hardware improvements were implemented by the author to improve grid visibility, e.g. reduction of reflection of glass wall, which led to the results presented in Chapters 4 and 5.

The setup of the photogrammetric system is shown in Figure 3.13 consisting of:

- High-power Grid Projector, mounted 1.6 m above the PVC floor placed in the channel and projecting a distorted rectangular grid of about 25 mm spacing onto the sediment surface. The mounting height was slightly increased for large dikes and decreased for small dikes in the scale tests;
- Three synchronized CCD cameras, located 2.1 m above the PVC floor and separated by 2 m in streamwise and 0.9 m in transverse direction, of about $2 \times 10^6$ pixels resolution and a recording frequency of up to 30 Hz. These cameras continuously record the projected grid and transfer the digital images to the computer;
- Side camera records the water level during a dike breach through the channel glass wall positioned 1.2 m from the glass wall and 1.5 m above the channel floor. It was assumed that the refraction effect is constant across the $y$-axis and depends only on the manually determined water surface profile along the glass wall (Figure 3.14). This assumption of a horizontal water level across the entire channel width was used to eliminate refraction effects, as shown in Figure 3.15 and discussed in 3.8.1.

![Figure 3.14](image)

**Figure 3.14** Streamwise section of measurement plane with water level determined using AICON side camera
Figure 3.15 Comparison of photogrammetric measurement results at impounded upstream dike face at left of image when refraction effects are (●) considered (■) not considered, with (- - -) and ( - - -) respective upstream dike face slopes and ( — ) water surface profile at glass wall during impoundment before breach start.

- Coded measuring marks on the channel top frame with accurately defined 3D-coordinates. These form the superior coordinate system allowed for a Helmert-transformation of the measured grid points into the channel coordinate system;
- Calibration panel with coded measuring marks to determine the relative and the internal camera orientations.

The accuracy of the photogrammetric method was assessed for different hydraulic test cases by Frank and Hager (2014) and is presented in 3.8.1.

### 3.8.1 Accuracy of photogrammetric system

To determine the accuracy of the applied photogrammetric system, the following three independent setups were tested (Frank and Hager 2014):

- Submerged cube test
- Plane dike breach test
- Fixed spatial dike breach test

**Submerged cube test**

A cube of 0.20 m sidewall length was placed onto a horizontal PVC platform and its surface topography assessed using the photogrammetric system for dry and submerged conditions. For the submerged conditions, the water depth was 0.28 m. The AICON data were compared with point gage data of ±0.2 mm accuracy (Figure 3.16).
Figure 3.16  Measurement of cube topography placed into channel with (–––) original cube geometry, AICON System data for (−−) dry, (⋯) submerged conditions.

The results indicate maximum absolute deviations of the two systems of some 2 mm for plane surfaces under both test conditions. For the submerged condition, the deviation is slightly larger as compared with dry test condition. Possible influencing factors are the refraction coefficients of air to glass and air to water. However, the cube corners are poorly represented because of the 25 mm grid spacing, by which cube corners are ill-mapped. If the grid lines are located close to the cube corners, they are not recognized by the AICON System. This effect is improved by reducing the grid spacing. Accordingly, dike breaches involving abrupt changes of the sediment surface have a reduced accuracy in terms of sediment surface mapping. This effect is of concern near the transition lines between the original dike setting and the eroded portions, as detailed below.

**Plane dike breach tests**

To test the effect of a curved and sloping water surface on the accuracy of the AICON System, a plane dike breach was analyzed with a constant approach flow discharge of $Q_o = 40$ l/s. The dike was as in Test 1 (Table 3.2) without any pilot channel. An additional CCD side camera was installed to measure both the dike sediment and water surface profiles during a breach test at $y = 0$ mm. Figure 3.17 shows dike surface measurements $z_b(x)$ at $y = 250$ mm and $500$ mm. Note the excellent fit of the two data sets, demonstrating the reliability of the AICON System for plane dike breaches.
Experimental setup and procedure

Figure 3.17 Dimensionless dike surface profiles $z_b(x)$ for plane dike breach test at planes $y = (\cdots) 250$ mm, $(\cdot\cdot\cdot) 500$ mm, and times from breach initiation $t = (a) 0$, $(b) 4$, $(c) 10$, $(d) 40$ s. (----) Reference sediment surface from side camera at $y = 0$ mm

Fixed spatial dike breach tests

Flow contractions and expansions during a spatial dike breach result in capillary surface waves. To test their effect on the measurement accuracy, the flow structure over a fixed spatial dike breach was investigated. A breach was created on a mobile dike similar to Test 1 by setting a constant inflow discharge $Q_o = 4$ l/s during 32 s after breach start. The channel was then drained, and the resulting dike breach was fixed with cement milk. The resulting ‘fixed dike’ was used after a day for dike overflow tests (Figure 3.18a).

Figure 3.18 Fixed dike breach test (a) photo with projected grid for $Q_o = 16$ l/s, view across channel glass wall, (b) (●) reference dike based on point gage data of fixed dike breach topography with triangulated mesh using TecPlot® 360™
The dike topography $z_b(x,y)$ was measured using a point gage with profiles both along the streamwise and the transverse directions, with a concentration of data along the steep zones of the dike surface. These data were interpolated with TecPlot® 360™ to create a continuous dike topography, forming the so-called reference dike (Figure 3.18b). Tests were then conducted using the fixed dike with $Q_0 = 0, 4, 8, 12$ and $16 \text{ l/s}$, thereby comparing the absolute deviations of the AICON System from the reference dike data. For the maximum discharge tested, the flow surface was exactly at the original dike crest elevation, so that all discharge passed only through the pilot channel (Figure 3.18a). This test highlights the effect of flow contraction previously addressed by a much higher flow depth along the glass wall as compared with that along the dike surface, due to centrifugal forces and visualized in 5.7. In addition, the negative effect of wave formation due to flow contraction is observed by the distortion of the grid on the dike surface.

Figure 3.19 compares various dike surface data sets for $x = 400 \text{ mm}$ and $Q_0 = 8 \text{ l/s}$. The ProSurf software determines the coordinates of grid points recognized by at least two cameras. If only one camera recognizes a certain grid point, the program estimates its possible position based on the neighboring data. Note that the software extrapolates data points even if no grid point is recognized, so that a data analysis is required to remove these data.

The effect of four evaluation methods on the measurement accuracy was compared:

1. Raw AICON data resulting from 0 to 3 cameras,
2. AICON data resulting from 1 to 3 cameras,
3. AICON data resulting from 2 to 3 cameras, and
4. As item 3, but points with a standard deviation of $\sigma_d > 2 \text{ mm}$ are eliminated; $\sigma_d$ results from the bundle adjustment method (e.g. Luhmann et al. 2013).

![Figure 3.19](image-url) Dike breach profiles $z_b(y)$ for $x = 400 \text{ mm}$ and $Q_0 = 8 \text{ l/s}$ using different evaluation methods: (——) reference dike profile, methods (— —) 1, ( - - -) 2, (···) 3
Experimental setup and procedure

The results are shown in Figure 3.19. Overall, the surface profile is similar for all three methods with maximum deviations of 10 to 20 mm. The maximum deviations are observed mainly along the steep dike surface portion at $y \cong 280$ mm, and at the steep water surface zone along the glass wall at $y \cong 100$ mm. Thus, zones of steep surface gradients either of the water or the sediment surfaces need a special care and should be considered in the final data processing.

Since methods 1 and 2 extrapolate the measured data along the dike top and the breach bottom, they tend to create steeper dike slopes as compared with the reference profile. In contrast, method 3 usually leads to less steep slopes as compared with the reference profile. At the bottom of the breach profile, the data deviate noticeably from the point gage profile in method 1. The agreement between the point gage and AICON data is best when using method 3, as shown in Figure 3.19. For method 4, the results are similar to method 3, except that singular data with too large standard deviations are eliminated, resulting in locally better data. Thus, the following evaluations refer solely to method 4.

For the accuracy tests, to avoid large deviations of the AICON data set from the reference data, no interpolation process was allowed for steep dike zones in which more than two grid points were not recognized. For less steep dike zones, no interpolation was allowed for if more than three consecutive grid points were not recognized.

Figure 3.20 shows the grid points of the fixed dike breach for four different discharges, including the recognized points by the AICON System. The white zones at the right side of these plots highlight portions where no points were recognized, due to capillary wave presence, as described above. Note that for $Q_o = 0$, this effect is absent, and that the zone not recognized increases in size with discharge. Note that an observer partially recognizes the grid, as in Figure 3.20c, d, yet the AICON System appears to be unable to do so.

![Figure 3.20](image) **Figure 3.20** AICON camera 3 images for fixed dike breach test with $Q_o = 0$ to 16 l/s, (→) flow direction. (©) Recognized grid points by AICON System
Figure 3.21  AICON camera images for fixed dike breach test, $Q_o = 16$ l/s, (→) flow direction

Figure 3.21 adds to Figure 3.20 by showing images recorded from the four cameras for $Q_o = 16$ l/s. The side camera again highlights the formation of the capillary wave pattern preventing a clear look onto the sediment surface. The images of cameras 1 to 3 showing plan views of the dike breach must be complemented by the AICON ProSurf software to facilitate the recognition of the grid points. Note that camera 1 results in a slightly better data imaging than cameras 2 and 3, from which the disturbances due to capillary wave formation again inhibit a 'clear view' on the entire grid.

Figure 3.22 shows the deviations between method 4 and the reference data for the fixed dike breach, $\Delta z_b(x,y) = z_b_{AICON}(x,y) - z_b_{gage}(x,y)$. For dry conditions ($Q_o = 0$ l/s), large absolute deviations of $|\Delta z_b| > 10$ mm occur only along the steep dike breach zones, since no grid points are recognized, as observed in Figure 3.22a. The missing data have been interpolated by triangulation. Medium deviations of $2$ mm $< |\Delta z_b| < 5$ mm occur at zones where point gage data were not taken densely enough, resulting in a poor reference dike surface. For most of the less steep dike surface zones, the AICON measurement provides a good accuracy of $|\Delta z_b| < 2$ mm.

For $Q_o = 4$ l/s (Figure 3.22b), the accuracy of the AICON System at the relatively flat breach surfaces is reduced to $-5$ mm $< \Delta z_b < 2$ mm in the regions with flow contraction and surface waves. Note the large regions at $x \approx 225$ mm for $y > 250$ mm, and $x \approx 850$ mm for $y > 150$ mm, where deviations are $|\Delta z_b| > 2$ mm. As previously explained, this effect is due to the assumption that the refraction effect is constant across the $y$-axis, depending only on the free surface profile along the glass wall. No further significant differences of $|\Delta z_b| > 2$ mm are observed.

For $Q_o = 8$ l/s (Figure 3.22c), the previous effects increase. Higher surface waves inhibit a recognition of most grid points within the breach zone for the lower right-hand zone $450$ mm $< x < 950$ mm. These comments are also valid for $Q_o = 16$ l/s except that the zone affected is even larger (Figure 3.22d).
Figure 3.22 Absolute deviations $\Delta z_b = z_b^{\text{AICON}} - z_b^{\text{gage}}$ for $Q_o = (a) 0, (b) 4, (c) 8, (d) 16 \text{ l/s}$. (e) Dike elevations $z_b(x,y)$ from reference data taken by gage measurement with (---) initial dike crest and pilot channel, (—) limit between initial dike and breach, (○) recognized grid points by AICON System, (⊙) reference measurements.

Figure 3.22e shows a plan view of the fixed dike with the elevation lines from the reference data, including in dashed lines also the initial dike topography. Note the S-shaped separation line between the initial dike set-up and the eroded zone.

The profiles $z_b(x,y)$ of the fixed spatial dike breach data of Figure 3.22 are shown in Figure 3.23 at (a) $y = 500$, (b) $y = 100$, (c) $x = 300$, (d) $x = 600 \text{ mm}$. For $z_b(x,y = 500 \text{ mm})$ (Figure 3.23a) absolute deviations of $|\Delta z_b| > 2 \text{ mm}$ occur in expanding flow regions for higher $Q_o$ at $x \approx 225 \text{ mm}$ and $x \approx 850 \text{ mm}$. As described in Figure 3.22 and previously explained, this is due to the assumption that the refraction effect is constant across the $y$-axis, depending only on the free surface profile along the glass wall. This effect has no significant influence e.g. for $Q_o = 8 \text{ l/s}$ between $0 < x < 150 \text{ mm}$ and $320 \text{ mm} < x < 750 \text{ mm}$.

Figure 3.23b shows the streamwise profile $z_b(x,y = 100 \text{ mm})$ in the breach region. Note the excellent fit for dry conditions and $Q_o = 4 \text{ l/s}$ if $x < 650 \text{ mm}$. For $Q_o = 8, 12, \text{ and } 16 \text{ l/s}$, the accuracy is good for $x < 340 \text{ mm}$. Note the local deviation at $x \approx 390 \text{ mm}$, which is...
attributed to surface wave effects. For values between $x \simeq 400$ and 800 mm, surface waves inhibit the recognition of most grid points within the breach zone. At $x \simeq 900$ mm, $-5 \text{ mm} < \Delta z_b < +1 \text{ mm}$ for $Q_o = 8 \text{ l/s}$, increasing to $-10 \text{ mm} < \Delta z_b < -2 \text{ mm}$ for $Q_o = 12 \text{ l/s}$, reaching $-10 \text{ mm} < \Delta z_b < -5 \text{ mm}$ for $Q_o = 16 \text{ l/s}$. As previously stated, the deviations increase with discharge $Q_o$ at locations with the formation of capillary waves.

The transverse profile $z_b(x = 300 \text{ mm}, y)$ is shown in Figure 3.23c. Note again the absolute deviation of $|\Delta z_b| > 2 \text{ mm}$ at expanding flow regions for higher $Q_o$, except for $Q_o = 0, 4 \text{ l/s}$ or $y < 275 \text{ mm}$. This is due to the assumption that the refraction effect is constant across the $y$-axis. Note the excellent fit with the reference data for $y < 275 \text{ mm}$. For $Q_o = 12 \text{ l/s}$, a local deviation at $y = 50 \text{ mm}$ is attributed to a measurement error by the AICON System.

The breach profile $z_b(x = 600 \text{ mm}, y)$ (Figure 3.23d) for $y > 225 \text{ mm}$ shows an excellent data fit for all $Q_o$ since the water levels are below $z_b = 150 \text{ mm}$ at the glass wall and no refraction effect occurs, in contrast to Figure 3.23c. For dry conditions ($Q_o = 0 \text{ l/s}$), deviations of $-10 \text{ mm} < \Delta z_b < -2 \text{ mm}$ are noted at $x \simeq 200 \text{ mm}$, due to missing data points because of the steep breach slope and partially due to the automatic triangulation from Tecplot® 360™. The remaining data for dry conditions fit the reference measurement well. For $Q_o = 4 \text{ l/s}$ the data fit the reference data well again, with $-2 \text{ mm} < \Delta z_b < 1 \text{ mm}$.
A local deviation at \( y = 50 \) mm is attributed to surface wave effects. For \( Q_0 = 8, 12, 16 \) l/s, no grid point values were detected due to surface waves at \( y < 200 \) mm, depending on the local flow depth.

**Summary**

The accuracy of the AICON System was experimentally assessed with three test setups:

*Cube tests* indicate that the AICON System represents the cube corners poorly because of the 25 mm grid spacing. For plane surfaces though, it has excellent measurement accuracy under dry conditions and satisfactory accuracy \(<1\) mm for submerged conditions compared with the point gage measurement. This is a good result considering refraction effects of \( 0.28 \) m submergence and the \( 0.04 \) m thick glass wall when determining the water surface at \( y = 0 \), particularly when compared to deviations of some \( 7 \) mm in previous ProSurf software versions, in which adaptations were not yet implemented.

*Plane dike breach tests* indicate that the data of the AICON System represent the dike breach surface well, rendering it a reliable measurement tool for 2D dike breaches.

The *fixed spatial dike breach tests* highlight that:

- Data quality is improved by filtering the raw data of the AICON System, though this leads to less available data points;
- Grid points are poorly recognized on steep breach surfaces either because of the relative position of the projector involving grid distortion and shades, or because of the camera position by which the dike surface is hidden;
- For discharges up to \( 4 \) l/s, the AICON measurement accuracy is considered good;
- For higher discharges, resulting in higher water levels and stronger surface waves, the grid point recognition becomes challenging, so that the accuracy is reduced;
- Flow contraction and expansion result in surface waves having a major influence on grid point recognition by the ProSurf software, because the streamwise grid-lines are hardly recognized in the images, especially in the downstream reach of the breach channel;
- Transverse water surface slopes have a negative effect on the data accuracy both because of refraction and water surface curvature effects (See also 5.7);
- Turbidity lowers the contrast of the grid system especially for large water depths, having a negative effect on the grid point recognition (Figure 3.18a);
- Light reflection inhibits grid point detection if more than one camera is affected;
• Assuming a constant refraction effect across the $y$-axis, the above measurement accuracy applies only for the breach region of similar water levels as the free surface profile along the glass wall. For other regions, the sediment surface is measured with satisfactory accuracy if the water level of these regions is defined.

The limitations of assuming a 2D water surface profile to eliminate refraction effects could be eliminated by determining the 3D water surface topography, as presented in 5.7, in which white-colored water was used for the breach experiment. However, the gain of such a method is expected to be small, notably because the standard test for capturing the sediment surface topography has to be repeated using white-colored water to obtain the water surface topography, and that further development effort would have to be applied to the photogrammetric system. Furthermore, the reaches which would benefit most from this improvement are at the same time strongly influenced by surface waves, so that no grid is visible and no data can be captured. The gain of having a 3D water surface for topography evaluation appears therefore small when compared to the additional effort.

As a conclusion, the AICON System applied to submerged breaches has an accuracy of 2 mm in non-problematic flow zones, whereas deviations of the present set-up increase to typically 10 mm in reaches with high transverse surface slopes. Although grid points were not recognized for high discharges, the system performance is better for tests with a movable granular bed due to a reduced effect of surface waves. Provided that the AICON System recognizes data points in a dike breach zone, the measurement accuracy of the spatial dike breach topography is satisfactory after data filtering, so that the dike breach topography can be adequately described and visualized as in Figure 3.24.

![Figure 3.24](image) Final triangulated dike surface from photogrammetrically computed coordinates
3.8.2 Data evaluation for spatial dike breach tests

The data evaluation for the spatial dike breach tests was conducted largely as described above for the accuracy tests, while improvements were implemented for the main spatial dike breach tests. As noted for Test 1 from the images from camera 3 at different times in Figure 3.25, the visible grid quality varies strongly over the reach captured due to the light reflection on the water surface and wave generation mainly due to the formation of the hourglass shape. Furthermore, the wetted or submerged dike surface appears darker than dry surfaces. Therefore, images often had to be re-considered using different parameters in the AICON ProSurf Software, resulting in two data sets for the two dike reaches. Consider for instance Figure 3.25d, which is composed of a central light portion, a darker portion to the right and the bottom of the image, and several dark portions separated by light reflections at the left. The latter portion is challenging in terms of data acquisition, so that information of other cameras is required to obtain a full surface data set. This lack of data is also noted in Figure 3.26a, in which information in the portions described above is missing. Data processing and surface visualization were realized using the software MATLAB®. The data were filtered according to method 4 previously described, using only AICON data points for which coordinates are determined from 2 to 3 cameras satisfying a standard deviation of $\sigma_d < 1.5$ mm. If two data point sets are required for proper dike surface evaluation, these were combined by adding only additional points from one of the filtered data sets to the main data set. Remaining outliers which were physically impossible and confirmed during a visual inspection were also removed and, if necessary, break lines inserted by adding interpolated points (Figure 3.26b), before applying the Delaunay triangulation method.

Figure 3.25 Spatial dike breach images from camera 3 for Test 1 at times $t [s] = (a) -13$, (b) 0, (c) 10, (d) 20, (e) 40, (f) 60, (g) 100, (h) 200, with $(-)$ contours of initial dike surface. Flow direction from top to bottom.
The resulting triangulated surface (as in Figure 3.24) was interpolated onto a 10 mm point mesh grid to create dike breach surface profiles for visualization purposes, including information on added points, as in Figure 3.26b.

Note that with the improved ProSurf software, it became possible to directly capture the dike surface profile along the glass wall \((y = 0)\) using the side camera of the AICON System and the same method as for the water surface profile illustrated in Figure 3.14. The measurement results of the two systems proved to be similar. The main limitation of the side wall camera for capturing the profile of granular material at the glass wall using the AICON System in an obscure room with the grid projected onto the dike surface is that the profile is perceived as a non-continuous line with alternate light and dark areas. This leads to maximal errors of the order of magnitude of the grid line thickness \(\pm 4\) mm. Nevertheless, this information is helpful for qualitative control, and to close gaps with missing surface data.

### 3.9 Test program

#### Preliminary tests

A number of preliminary tests were conducted to determine a simple test procedure allowing for the study of a spatial dike failure process due to overtopping. The preliminary tests indicated the significant effect of seepage on the general dike stability and the dike breach process. They highlighted the importance of the drainage discharge capacity and its position relative to the dike. The findings led to major improvements of the channel setup, so that the dike remained stable during the different test configurations of the erosion process. The results of the preliminary tests are presented in Chapter 4, but are not used for further evaluation.
Scale effect tests

A total of 19 tests were conducted (Table 3.2) to (1) verify test repeatability, (2) validate the symmetry assumption for half-model tests, and (3) to analyze Froude scalability. The results are presented in Chapters 4 and 6. The verification of test repeatability was conducted for dikes of height $w = 0.2 \, \text{m}$ by comparing the hydrographs, the breach profiles and the eroded sediment volume, while the hydrographs were compared for dikes of $w = 0.6 \, \text{m}$ and $d = 3.78 \, \text{mm}$ at the physical limits of the employed test setup. One symmetry test for a dike with $w = 0.3 \, \text{m}$ was conducted to confirm observations by Wallner (2014) that full-model and half-model tests lead to the same results. Froude scalability was analyzed using three scale families. According to Novak (1984), the scale factor $\lambda = L_m/L_p$ is the ratio of a variable in the model to the corresponding variable in its prototype. According to Schmocker (2011), dikes with $w \geq 0.2 \, \text{m}$ do not reveal large-scale effects. Thus, the dike with $w = 0.3 \, \text{m}$ and a pilot channel sediment height at the glass wall of 0.2 m was expected to satisfy the same requirements and was thus selected as reference dike $\lambda = 1$. The largest dike of $w = 0.6 \, \text{m}$ then corresponds to $\lambda = 2$, whereas the smallest dike with $w = 0.15 \, \text{m}$ corresponds to $\lambda = 0.5$. The properties are described in Table 3.2 and the Froude scales are specified in Chapter 2, Table 2.3.

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<th>$Q_{b,d}$ [l/s]</th>
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* Test 10: $x_D = 2.30 \, \text{m}$; Test 11: $x_D = 2.36 \, \text{m}
Breach process tests: Constant inflow discharge $Q_o$

A total of 17 tests were conducted using constant inflow discharge $Q_o$ as hydraulic boundary condition to systematically investigate the effect of specific parameters on the breach process (Table 3.3, Chapters 5 and 6). The main parameters varied were the design breach discharge $Q_{b,d}$, the sediment grain size $d$, the pilot channel width $b_p$, and the dike shape parameter $\mu$, which essentially describes the dike cross-section $A_D = (7/3)\mu w^2$ (see also 6.4.1). Further tests were conducted to study the effect when using a mobile sediment bed instead of a fixed channel floor. The water surface topography was captured using water colored white with TiO$_2$ and using the photogrammetric system.

Table 3.3  Test program for breach process half-model tests presented in Chapters 5 and 6

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* Test 28: $x_D = 3.77$ m due to fixed drainage, reservoir volume compensated so that $x_D \approx 3.44$ m
Experimental setup and procedure

Breach process tests: Different reservoir water surface areas and $Q_o=0$

A total of 10 tests were conducted using as hydraulic boundary condition $Q_o = 0$ and different horizontal reservoir water surface areas $A_R$. These tests were used to systematically investigate the effect of the reservoir water surface area $A_R$ and the reservoir shape on the breach process (Table 3.4). The results are presented in Chapters 5 and 6. The tests were conducted by simulating an additional reservoir water volume to the physical reservoir using a controller, as previously described. To insure test repeatability of this different hydraulic boundary condition, repeatability tests were conducted for tests with $A_R = 13.44$ m$^2$ and $A_R = \infty$.

Table 3.4  Test program for simulated reservoir water surface area half-model tests presented in Chapters 5 and 6

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* Triangular (V-shaped) reservoir shape with $A_R(h_o) = (h_o/h_{o,0})x_Db$
4 Experimental results of model limitation tests

4.1 Introduction

The hydraulic modelling of laboratory dike breaches is essential both to determine the general dike breach characteristics and to understand the underlying physical processes. To assure that the obtained results are reliable and scalable according to Froude similitude (Chapter 2), tests were conducted to determine the model limitations (Table 3.2). The temporal breach evolution was evaluated in terms of head water level, discharge, and sediment topography profiles. Characteristic hydraulic properties and breach channel shapes are presented in 4.7 for Test 1.

First, the influence of seepage was determined to avoid dike failures due to instability issues. Therefore, preliminary tests were conducted to optimize the test setup and to quantify the expected drainage discharge. Its effect on the design discharge is discussed and the observations are compared with the corresponding tests of Schmocker (2011).

In the second and third step, test repeatability and symmetry were considered and evaluated using the respective hydrographs and topography profiles. These tests are necessary to prove that the simplified half-test model and test procedure, the hydrograph evaluation method, the photogrammetric system and the subsequent data treatment are reliable. The results are discussed and compared with the corresponding tests of Schmocker (2011).

Lastly, Froude similitude was tested for dike heights ranging between 150 mm and 600 mm (Table 3.2). Three test series were conducted using different design breach discharges and sediment grain sizes. The results are discussed and compared with the Froude similitude tests for plane dike breaches of Schmocker (2011) and the flow hydrographs of quasi-exact and tilted (distorted) scale models of Al-Riffai (2014).

4.2 Seepage

Seepage has an important influence on dike breach test results, because it can lead to dike instability during the breach process. Therefore a dike breach could be dominated by sliding of the downstream dike portion instead of surface erosion. This effect has been noticed for plane dike breach tests for short periods at breach start when using small constant inflow discharges, large reservoir volumes, homogeneous sediment of grain size $d \geq$
4 mm and the setup of Schmocker (2011). A drainage setup at the upstream dike toe and an insufficient drainage capacity were found to be the main reasons for the observed instabilities for large grain sizes. Dike failure due to seepage was observed in various tests of Schmocker (2011). Test 63 is displayed in Figure 4.1a for \( t = 1.4 \text{ s}, \ d = 5.5 \text{ mm}, \ Q_o = 6 \text{ l/s}, \ x_D = 1 \text{ m} \), with the drainage positioned below the upstream dike face slope. For the majority of tests though, the rise of the water level and overtopping appeared to occur fast enough, so that the dike maintained its internal stability. For larger test times, the effect of seepage on the breach process was shown to be negligible.

This setup is not applicable to dikes built from homogeneous granular material if the rise of the headwater level is slow under the constant headwater scenario with a large reservoir volume. Seepage-induced dike failure is then inevitable if no countermeasures are taken. Figure 4.1b shows a seepage-induced dike breach of a preliminary test for \( t = 0 \text{ s}, \ d = 3.78 \text{ mm}, \ Q_o = 8 \text{ l/s}, \ x_D \approx 4 \text{ m} \) and a drainage below the entire dike. The sliding of the downstream dike face slope occurs due to relatively large sediment size, low drainage discharge capacity and a slow rise of the headwater level due to the relatively large reservoir. Note that the downstream dike surface slides at the locations where the seepage water table reaches it.

To reduce the limitations for the same 2D dike breach test setup, a drainage of length \( L_{dr} \approx 220 \text{ mm} \) was installed at the downstream dike toe and the discharge capacity was increased (Figure 4.1c,d). For \( t = 0 \), the seepage water table had not reached the downstream dike surface, while at \( t = 2 \text{ s} \), the downstream dike surface was eroded by surface erosion. It was clearly visible in the image sequence that the single sediment grains did not move before the erosive force of the overtopping water reaches the grains, indicating that the internal dike stability is intact.

The seepage line profile varied depending on the drainage position relative to the dike and drainage capacities, as qualitatively shown in Figure 4.1e. In the test setup using drainage 1 (small drainage capacity \( Q_{dr,M,1} \) at the upstream dike toe as used by Schmocker 2011), the seepage line is steep in the upstream but less steep in the downstream dike portion. Seepage discharge \( Q_{s,1} \) at the downstream dike toe thus may influence the dike stability. In the test setup using drainage 2 (sufficient drainage capacity \( Q_{dr,M,2} \), positioned at the center of the downstream dike reach), the seepage line falls smoothly and becomes steeper above the drainage. If seepage exceeds the drainage capacity, the seepage line becomes less steep thereby increasing the downstream seepage line level and the pressure
Influence of seepage on plane dike breach process (a) Test 63 (Schmocker 2011) for $t = 1.4$ s, (b) preliminary reservoir test for $t = 0$ s failing due to seepage; reservoir test for $t =$ (c) 0 s and (d) 2 s with intact internal dike stability during surface erosion process; (e) seepage lines for (---) drainage 1 with $Q_{dr} > Q_{dr,M}$, (--) drainage 2 with $Q_{dr} < Q_{dr,M}$, (•••) drainage 2 with $Q_{dr} > Q_{dr,M}$

on the drainage. The rising level can lead to seepage outflow $Q_{s,2}$ in the center of the downstream dike surface and influence the dike stability. If seepage exceeds the drainage capacity for more than some seconds, seepage will escape at the downstream dike toe and influence the dike stability for the constant headwater scenario.

For spatial dike breach tests, the test setup was adapted as described in Chapter 3. The limit for practicable test setups was found for Tests 5 to 8 with dikes of initial height $w = 600$ mm, pilot channel height $w_p = 200$ mm, crest length $L_k = 200$ mm, up- and downstream dike slopes and transverse pilot channel slope $S_o$, $S_u$ and $S_p$, respectively, of 1:2, sediment size $d = 3.78$ mm and constant inflow discharge $Q_o = 50$ l/s.

Preliminary tests indicated that poor positioning of the drainage leads to a sliding of the downstream dike face because of seepage outflow, as previously described. The relative drainage position was therefore adjusted until the dike remained stable for headwater levels close to $w$ and a drainage position $x_{dr} = 2.14$ m. Still, for $Q_o = 50$ l/s, seepage water escaped at mid-height of the downstream dike surface and led to instabilities during the initial breach phase in Tests 5 and 6. The drainage position was reduced to $x_{dr} = 2.00$ m.
Experimental results of model limitation tests

Figure 4.2  Influence of seepage on spatial dike breach for Test 5 at \( t = (a) 0 \), (b) 18 s and for Test 8 at \( t = (c) 0 \), (d) 18 s with inflow discharge \( Q_o \), breach discharge \( Q_b \) and seepage outflow discharge \( Q_s \) for Tests 7 and 8, which remained stable (Figure 4.2). Nevertheless, these tests were at the limit of success, because the maximum drainage capacity \( Q_{dr,M} \cong 10 \text{ l/s} \) was reached for high headwater levels. The hydraulic properties of Tests 5 to 8 are detailed in 4.3.

The different seepage line profiles have been described above. For Test 5 in Figure 4.2a,b, the seepage line has a reduced slope in the upstream dike reach, steepening only close to the drainage. Hydrostatic pressure increases and thereby leads to a higher drainage capacity. If the less steep and steeper seepage lines intersect outside the dike body, the dike becomes unstable. For spatial dike breach tests with \( d \leq 3.78 \text{ mm} \), \( w \leq 600 \text{ mm} \), or \( Q_o \leq 50 \text{ l/s} \), the limits of drainage positioning and its capacity have been determined so that internal dike stability issues were excluded for the applied test procedure with constant inflow discharge.

For the planning of the test program (Chapter 3), the drainage design discharge \( Q_{dr,d} \) resulting from seepage was estimated during pre-tests by impounding dikes of initial crest heights \( 150 \text{ mm} \leq w \leq 600 \text{ mm} \) and sediment of grain sizes \( 0.43 \text{ mm} \leq d \leq 3.78 \text{ mm} \). Expected values of \( Q_{dr,d}/Q_o \) were in the range of 4 to 29\%, which highlights the importance of the choice of the relevant discharge to compare results. In accordance with Coleman et al. (2002), the breach discharge \( Q_{b,t} \) was therefore selected as the relevant discharge for comparison of results, given by Eq. (3.1). Accordingly, a design breach discharge \( Q_{b,d} \) was determined for each test, taking \( Q_{dr,d} \) into account.
4.3 Repeatability

Before conducting the main test series, it was necessary to check the repeatability of the test procedure and possible influences of the various factors affecting the dike breach process. Two test series were conducted with initial dike heights of $w = 200$ and 600 mm. The small dikes represent typical dike dimensions as selected by Schmocker (2011) for plane dike breach tests, while the larger dikes were used to test repeatability at the limits of the test procedure in terms of channel size, inflow and drainage discharge capacity, and for applying the photogrammetric system. Repeatability was considered for the flow hydrographs and the topographic profiles.

Tests 1 to 4 with $w = 200$ mm

The test with a dike height $w = 200$ mm was repeated four times with a dike toe distance from the inlet wall $x_D = 3.77$ m, dike width $b = 1000$ mm, crest length $L_K = 100$ mm, pilot channel height $w_p = 100$ mm and width $b_p = 200$ mm, uniform non-cohesive sediment of grain size $d = 1.25$ mm and design breach discharge $Q_{b,d} = 13$ l/s. Figure 4.3 shows the hydraulic properties of the four compared tests. At all times, the headwater level $h_o(t)$, breach discharge $Q_b(t)$, inflow discharge $Q_o(t)$ and drainage discharge $Q_{dr}(t)$ correlate well. The discharge $Q_o(t)$ is almost equal for the four tests, whereas $h_o(t)$ has a deviation of around ±1% in relation to $w$ once $h_M$ is reached and for $t > 150$ s. The maximum deviation for $Q_M$ is ±0.25 l/s (2% of $Q_{b,d}$), which also applies to the general trend of $Q_b(t)$. The maximum local deviation was determined at $t = 103$ s with around 8.8% of $Q_{b,d}$. This deviation results from lumps of wetted sediment falling into the breach channel at different times in these tests, leading to a distinguishable drop in breach discharge $Q_b(t)$, followed by a rise when the lump is eroded, e.g. for Test 1 at $t = 83$ and 100 s and for Test 2 at $t = 90$ and 113 s. The largest deviations of $Q_o(t)$ are between 0.5 and 1 l/s and therefore 3.8 to 7.7% of $Q_{b,d}$. The results indicate that the drainage discharge $Q_{dr}(t)$ of repeated tests can deviate up to ±12% having a slight influence on the breach discharge $Q_b(t)$. Nevertheless, both the breach discharge and headwater elevation were well repeatable.
Experimental results of model limitation tests

Figure 4.3  Repeatability of hydraulic properties for dike of height $w = 200$ mm and sediment grain size $d = 1.25$ mm with (a) headwater leve l, (b) breach discharge, (c) inflow discharge, (d) drainage discharge for Tests (—) 1, (•••) 2, (—•) 3, (——•) 4

For the described test setup, the topography of Tests 1 to 3 was evaluated using the photogrammetric system as described in Chapter 3. No photogrammetric data was available for Test 4. For Tests 1 and 2, the grid projector was positioned 1.55 m above the channel axis, while it was positioned above the front glass wall for comparison of data quality for Test 3. The topography was evaluated for $t = 0, 10, 20, 40, 60, 100$ and 200 s from which four streamwise and transverse sections were extracted (Figure 4.5). The profiles of Test 1 were presented by Frank and Hager (2015).

Figure 4.4  Plan view of dike (schematic) with (—) streamwise sections at $y$ [mm] = 0, 100, 200 and 500, and (—) transverse sections at $x$ [mm] = 300, 450, 600 and 900 for Tests 1 to 3. The pilot channel at the glass wall is marked in blue.
The streamwise surface profiles $z(x)$ are illustrated in Figure 4.5a-d. Figure 4.5a shows the streamwise section at $y = 0$ mm along the glass wall. During the first 20 s after breach start, the downstream portion of the pilot channel – with an initial height of $z(x) \approx 100$ mm – is eroded. For $t > 20$ s the main erosion occurs along the upstream pilot channel reach, leading to a smaller breach surface slope and a rotational movement around the coordinate $(735;0;0)$. A similar erosion pattern is observed in Figure 4.5b at $y = 100$ mm, the breach shape evolution resembles that of 2D-dike breaches, e.g. Schmocker (2011). At $y = 200$ mm (Figure 4.5c), a parallel erosion pattern is noted for $t > 10$ s. At the channel axis at $y = 500$ mm (Figure 4.5d), erosion starts for $t \geq 40$ s, with the upstream dike surface having the characteristic erosion pattern due to the typical hourglass breach shape. Deviations at $x \approx 225$ mm and $x \approx 850$ mm are due to the refraction effect assumption as explained in Chapter 3 and described by Frank and Hager (2014).

The transverse surface profiles $z(y)$ are illustrated in Figure 4.5e-h. Figure 4.5e shows the transverse section at $x = 300$ mm from the upstream dike toe. After breach initiation, the initial transverse pilot channel slope $S_p = 2$ at $y \leq 200$ mm is reduced and remains constant for $t > 20$ s. The deviation of $z(y) > 150$ mm is again due to refraction effects. For $t > 20$ s, the breach develops quite uniformly, both in the horizontal and vertical directions, while the transverse slope hardly varies. At $t = 200$ s, the slope becomes steep due to apparent cohesion. This shape was validated by visual inspection. Similar observations occurred for $x = 450$ and $600$ mm (Figure 4.5f-g), though the vertical erosion is faster than the lateral, and the transverse breach slope is steeper as compared with the initial pilot channel slope. Figure 4.5h shows the evolution of the breach tail at $x = 900$ mm. At $t = 10$s, the breach material is first transported along the glass wall and then forms its own sediment bed for $t \geq 20$ s, which grows in width over time. For $t = 60$ s, data points were not available for all tests at all times, so that triangulation led to a bed level with $z = 0$ and an inaccurate mean profile. The data evaluation process at the breach tail was work-intensive because of grid recognition difficulties, the informative value being relatively low. Therefore, for the remaining tests, the breach tail profiles will be shown but not discussed.

A comparison of the mean profile with the respective three test profiles shows that they match remarkably well, especially when considering the multiple factors affecting the dike breach, e.g. the test equipment, the test setup and the photogrammetric evaluation method. The positioning of the grid projector in Test 3 had no noticeable influence on the evaluation process. The deviation of the mean profile perpendicular to the respective profiles is generally around $\pm 3$ mm or $\pm 1\%$ of $w$. For the steep dike surface slopes, the deviation is slightly larger, as described in Chapter 3 and by Frank and Hager (2014).
Experimental results of model limitation tests

Figure 4.5  Spatial dike breach evolution $z(x)$ at $y$ [mm] = (a) 0, (b) 100, (c) 200, (d) 500 and $z(y)$ at $x$ [mm] = (e) 300, (f) 450, (g) 600, (h) 900 for $t$ [s] = (-) 0, (--) 10, (-) 20, (----) 40, (-x-) 60, (-o-) 100 and (-□-) 200. Test (-- 1, (--) 2, (--) 3, (--) mean value
Larger deviations are attributed to the steep slope and apparent cohesion, e.g. for \( y = 200 \text{ mm} \) and \( t = 40 \text{ s} \) at \( x = 600 \text{ mm} \), as well as refraction effects and a poor grid quality, e.g. for \( y = 100 \text{ mm} \) and \( t = 0 \) at \( x < 150 \text{ mm} \) in Test 2.

The initial pilot channel base \( w_{p,0} \) of Test 1 was about \( 4 \text{ mm} \) above the planned elevation at the glass wall, while for \( t > 40 \text{ s} \), the respective profiles were below the mean profile. This is an indication that a slightly higher pilot channel base \( w_{p,0} \) will lead to a slightly lower channel breach elevation \( z_b \) for \( t > 40 \text{ s} \) due to the resulting marginally higher maximum headwater level \( h_M \) (Figure 4.3). Considering the many factors influencing spatial dike breaches in the laboratory, the test procedure proved to be repeatable and reliable for a dike height \( w = 200 \text{ mm} \) and inflow discharge of \( 15 \text{ l/s} \) in terms of flow hydrographs and surface topography. A qualitative comparison of topography repeatability between these results and those of Schmocker (2011) shows good agreement.

**Tests 5 to 8 with \( w = 600 \text{ mm} \)**

The test with a dike height \( w = 600 \text{ mm} \) was repeated four times with \( b = 1000 \text{ mm} \), \( L_K = 200 \text{ mm} \), \( w_p = 200 \text{ mm} \), \( b_p = 400 \text{ mm} \), \( d = 3.78 \text{ mm} \) and \( Q_{b,d} = 41.8 \text{ l/s} \). Figure 4.6 shows the hydraulic properties of these four tests, which were presented by Frank and Hager (2015). Test 5 was the first test conducted with \( x_{dr} = 2.14 \text{ m} \) and \( x_D = 2.30 \text{ m} \). As described in Chapter 3, this dike failed due to excessive seepage, as visible in Figure 4.6a for \( h_o(t > 10 \text{ s}) \) and in Figure 4.6b for \( Q_b(t > 18 \text{ s}) \). Furthermore, for \( t > 70 \text{ s} \), values of \( h_o \) and \( Q_b \) start to fluctuate. The water loss through the drainage system caused air to enter the inlet pipe, which led to fluctuations of \( Q_o \) (Figure 4.6c). Drainage discharge reached its upper capacity limit \( Q_{dr,M} \cong 10 \text{ l/s} \) (Figure 4.6d) and maximum headwater level \( h_M \cong 600 \text{ mm} \) for this specific test setup, causing the observed seepage at the downstream dike toe, as described in 4.2. For \( t > 90 \text{ s} \), the drainage measurement basin was full so that the drainage discharge \( Q_{dr}(t) \) was no more measurable. Therefore, \( Q_{dr}(t > 90 \text{ s}) \) was assumed constant using the last determined value for \( Q_{dr} \) before the basin was filled.

For Test 6, the dike stability was slightly improved by reducing \( x_{dr} \) to \( 2.08 \text{ m} \). To avoid air entering the closed pumping system, a water volume of \( 0.73 \text{ m}^3 \) was added immediately after test start and a hose was installed downstream of the dike to compensate the water loss due to drainage. The issue of air entrainment was successfully postponed by \( 110 \text{ s} \) to \( t > 180 \text{ s} \) as shown in Figure 4.6c. To further improve dike stability in Tests 7 and 8, \( x_{dr} \) was further reduced to \( 2.00 \text{ m} \) and thereby \( x_D \) increased to \( 2.44 \text{ m} \). These modifications and the added water volume led to stable dikes during an entire test, without air entrainment by the pumping system (Figure 4.6).
Experimental results of model limitation tests

Figure 4.6  Repeatability of hydraulic properties for a dike of height \( w = 600 \text{ mm} \) and sediment grain size \( d = 3.78 \text{ mm} \) with (a) headwater level (b) breach discharge (c) inflow discharge (d) drainage discharge and tests \( \bullet \bullet \bullet \) 5, \(-\bullet\) 6, \(--\) 7, \(\_) \) 8

Repeatability of Tests 5 to 8 was only tested for the hydraulic properties. For Tests 5 to 7, one ultrasonic probe was used. As described in Chapter 3, its range is 100 to 700 mm, which is the limit for this test setup of \( w = 600 \text{ mm} \). Therefore, no data were available during 7 s for the highest values of \( h_0(t) \). For Test 8, an additional probe was installed and the two measured data sets were combined. The missing data of Tests 5 to 7 were also complemented using this additional data set; possible inaccuracies were considered negligible. For the successful Tests 7 and 8 involving stable dikes, the values of \( h_0(t > 10 \text{ s}) \) and \( Q_b(t > 18 \text{ s}) \) are slightly higher compared with these of the unstable dikes in Tests 5 and 6. This indicates that the values of \( h_0(t) \), \( Q_b(t) \), \( Q_o(t) \) and \( Q_{dr}(t) \) correlate well for tests where dike failure due to seepage is excluded.

Summarizing the results of these two repeatability test series, repeatability was demonstrated for test procedures using dikes of height \( 200 \text{ mm} \leq w \leq 600 \text{ mm} \) for both flow hydrographs and breach topographies. Furthermore, the reliability of the test procedure, the hydrograph evaluation method, the stereo-photogrammetric system and the subsequent data treatment were demonstrated. The hydraulic properties indicate that seepage-induced dike breaches lead to altered results and should therefore be avoided.
4.4 Symmetry

The standard spatial dike breach test is a half-model test, in which the pilot channel used for breach initiation is located along the glass wall. Given the smooth channel surface, flows are assumed to be in the turbulent smooth regime. These half-model tests permit the measurement of the dike breach surface using the photogrammetric system (3.8). To verify that results of the half-model are representative for full models, two tests were conducted and the results compared in terms of hydraulic properties and breach surface profiles in Figure 4.7 and Figure 4.8, respectively. Both test dikes have a dike height of \( w = 300 \text{ mm} \) and were built of uniform non-cohesive sediment of grain size \( d = 1.75 \text{ mm} \).

The half-model Test 9 has a channel width of \( b = 500 \text{ mm} \), a design breach discharge \( Q_{h,d} = 7.4 \text{ l/s} \) with the pilot channel along the glass wall. The water surface profile was determined using the photogrammetric system. The full-model Test 10 has \( b = 1,000 \text{ mm} \), \( Q_{h,d} = 14.8 \text{ l/s} \) with the pilot channel axis along the channel axis. As the water surface profile cannot be measured along the channel/breach axis, values of Test 9 were used, assuming symmetry. Adaptations were implemented in the AICON ProSurf software (v. 8.60.00), enabling to manually introduce the water surface profile coordinates.

![Comparison of hydraulic properties](image)

**Figure 4.7** Comparison of hydraulic properties (a) headwater level, (b) breach discharge, (c) inflow discharge, (d) drainage discharge for (--) half-model Test 9, (--) full-model Test 10 of dike height \( w = 300 \text{ mm} \) and grain size \( d = 1.75 \text{ mm} \). Test 9 is used as reference
The hydraulic properties of the half- and full-model tests are compared in Figure 4.7, with the half-model Test 9 serving as reference (discharges of Test 10 are halved). The headwater level evolution $h_a(t)$ is almost identical for both tests during the initial breach phase. For $25 \, s < t < 55 \, s$, the headwater level of the half-model test (Test 9) drops more slowly as compared with Test 10. For $t > 55 \, s$, a parallel evolution of the two headwater level curves is noted and the deviation remains at 3.5% compared to the dike height $w$. The breach discharge evolution $Q_b(t)$ is almost the same for both tests until $Q_M$ is reached at $t = 20 \, s$. For $t > 20 \, s$, $Q_b(t)$ of the half-model is up to 0.5 l/s lower compared to the full model (6.8% of $Q_{b,d}$). For $t > 60 \, s$, both breach discharge curves are again similar. The curves of the two inflow discharges $Q_a(t)$ and drainage discharges $Q_d(t < 20 \, s)$ are almost identical. For $t > 20 \, s$, the drainage discharge of the half-model is higher by 3% compared to $Q_{b,d}$. These results indicate that the half-model test is slightly influenced during some 30 s following the maximum breach discharge $Q_M$. The small deviation of $Q_d$ (3% of $Q_{b,d}$) is not considered to be the main reason for the difference in the breach evolution. Deviations are attributed to the presence of the glass wall (shear stress, turbulent structures, stabilizing effect on sediment), small differences in the dike setup or the exact hydraulic characteristics.

Topography profiles of the half- and full-model tests are compared in Figure 4.8 with (a) to (d) showing the streamwise and (e) to (h) the transverse sections for the initial dike shape up to $t = 200 \, s$. For the comparison of the profiles, the half-model was taken as reference for the graphs and the full model divided into two sections at the channel axis. The left side was shifted by $-500 \, mm$ along the $y$-axis and the right side was mirrored at $y = 250 \, mm$.

Figure 4.8a shows the streamwise sections at the breach centerline ($y = 0 \, mm$). For $t = 0 \, s$, the initial pilot channel base curve of the full-model test is clearly above the half-model. This is partially attributed to the evaluation method using the photogrammetric system for the full model, where corners are not perfectly depicted as described in Chapter 3 and by Frank and Hager (2014), whereas the data for the half-model is accurately extracted using the sidewall camera. For $t > 10 \, s$, deviations between the full- and half-model tests become visible in the upstream breach reach. The full-model curve is more rounded and lower compared to the half-model curve at $t = 20 \, s$, which becomes more accentuated for $t = 200 \, s$ up to $\Delta z_{max} \approx 18 \, mm$. Nevertheless, the deviations of the curve shape at the breach center and end are small. The deviation in the upstream breach reach is attributed to reduced flow velocities close to the glass wall, which leads to a reduced bed shear stress $\tau_b$ and thus erosion. The sediment along the glass wall will then tend to
stabilize the neighboring sediment grains. This phenomenon has also been observed in plane dike breach tests with similar values of $\Delta z_{\text{max}}$ by Müller (2015). For $y = 100\, \text{mm}$ in Figure 4.8b, the roundness and position of the curves are similar to the curves at $y = 0\, \text{mm}$, since the stabilizing effect of the glass wall is strongly reduced. Larger deviations are mostly noted for $t \geq 60\, \text{s}$ and $x > 950\, \text{mm}$, since grid recognition proved difficult for half-test models. In Figure 4.8c-d, the curves of both tests agree well, especially considering that the left- and right-side breach profiles of Test 10 deviate from each other in the same order of magnitude.

The transverse section at $x = 400\, \text{mm}$ at the upstream end of the pilot channel base is shown in Figure 4.8e. The half-model curve generally lies above the full-model curves, but agree reasonably well for $y > 100\, \text{mm}$. For $y < 100\, \text{mm}$, deviations up to $14\, \text{mm}$ are observed along the glass wall for $t = 100\, \text{mm}$ and are partially attributed to stabilizing sidewall effects in the half-model test. For $t = 60\, \text{s}$, the curve shape of the half-model is similar to the full model curve, albeit at a higher elevation across the entire width. The transverse sections at $x = 650\, \text{mm}$ of the half- and full-model tests are similar (Figure 4.8f). A slight difference is observed at the pilot channel base for the initial dike, where the full-model value is too high at the glass wall ($y = 0\, \text{mm}$). A small deviation is also observed for $y < 150\, \text{mm}$. In the transverse section $x = 900\, \text{mm}$ (Figure 4.8g), relatively large deviations are visible in the half-model test profiles near $y = 150\, \text{mm}$ for $t = 60$ to $200\, \text{s}$. These systematic differences were observed in several tests under similar conditions resulting from transverse water surface slopes, as described in Chapter 3. A general agreement between the tests at the breach tail is noted in Figure 4.8h.

As a general conclusion, the data extracted from half-model dike breach tests apply to describe the full-model breach, in accordance with conclusions of Wallner (2014) and Cestero (2010). A slight stabilizing effect attributed to the glass wall was observed due to the hydraulic properties and topography profiles. It was impossible to exclude marginal effects in the full model due to the different dike setup and the applied assumption of equal water surface profile at the same time for the photogrammetric evaluation. The full-model data were surprisingly good and superior to half-model data in terms of number of points and the resulting topography consistency. The described method is laborious though, since two companion tests have to be conducted to extract the water level profile of a half-model test for each full-model test.
Experimental results of model limitation tests

Figure 4.8 Spatial dike breach surface evolution for streamwise sections $y_{ref}$ [mm] = (a) 0, (b) 100, (c) 200, (d) 400 and transverse sections $x$ [mm] = (e) 400, (f) 650, (g) 900, (h) 1300 for $t$ [s] = (–) 0, (–−−) 10, (−−−) 20, (−−−−) 40, (−−−−−) 60, (−−−−−−) 100, (−−−−−−−) 200. (−−−−−−−−−−−−−) Half-model Test 9, (–−−−−−−−−−) full-model Test 10 left side of axis, (−−−−−−−−−−−) full-model Test 10 right side of axis
4.5 Scale effects

Introduction

Three scale effect tests were conducted for dike heights $w = 150, 300$ and $600$ mm with the respective scale factors $\lambda = 0.5, 1$ and $2$ (Table 3.2). Tests with dikes of height $w = 300$ mm were chosen as reference (subscript $\text{ref}$) test $\lambda = 1$. Parameters were scaled according to Froude similitude (Chapter 2), except for the streamwise length of the downstream PVC floor (deemed negligible for the present setup) and the drainage discharge, determined prior to the test. This estimation of the design drainage discharge $Q_{d,\text{dr}}$ as described in 3.3 and 4.2 proved challenging, and led to deviations in the design breach discharge $Q_{b,\text{dr}}$. Nevertheless, the accuracy of the comparison was improved as compared to when neglecting drainage discharge. For the photogrammetric evaluations, the time of origin $t_0$ was set at the moment the water front reached the downstream end of the pilot channel, as explained in Chapter 3. To compare the hydraulic properties in Figure 4.9, the moment $t_p$ when the water level reached the base of the pilot channel (i.e. $h_{\text{ref}} = 200$ mm for the reference dike) was determined from the photogrammetric images. The following parameters were used for the three scale test series:

- Scale effects a (Tests 11,9,8) $Q_{b,\text{dr},\text{ref}} = 7.4$ l/s $d_{\text{ref}} = 1.75$ mm $t_{p,\text{ref}} = -6$ s
- Scale effects b (Tests 12-14) $Q_{b,\text{dr},\text{ref}} = 3.7$ l/s $d_{\text{ref}} = 1.75$ mm $t_{p,\text{ref}} = -10$ s
- Scale effects c (Tests 15-17) $Q_{b,\text{dr},\text{ref}} = 7.4$ l/s $d_{\text{ref}} = 0.86$ mm $t_{p,\text{ref}} = -5$ s

For each series, first the hydraulic properties are presented, followed by the topography profiles. For better visibility of the latter, the curves for one section were divided onto two graphs. The subfigures (a) to (d) show curves for $t_{\text{ref}}$ [s] = 0, 10, 40, 100, whereas subfigures (e) to (h) show curves for $t_{\text{ref}}$ [s] = 0, 20, 60, 200. The streamwise sections are presented in the first figure, the transverse sections in the second figure. Due to the considerable dike length of 2.6 m and measurement limitations for $\lambda = 2$, no data were available for the transverse section $x = 2,600$ mm or $x_{\text{ref}} = 1,300$ mm. Due to high headwater levels for $\lambda = 2$, the water surface profile was not visible along the glass wall but estimated using the ultrasonic probe data. For simplicity, subscript $\text{ref}$ will not be employed in the following scale effect descriptions. The dimensionless vertical coordinate $Z$ is defined as the vertical coordinate divided by the dike height ($Z = z/w$), the dimensionless streamwise coordinate $X$ as the streamwise coordinate divided by the dike length ($X = x/L_D$), and the dimensionless transverse coordinate $Y$ as the transverse coordinate divided by the dike width ($Y = y/b$).
Scale effects

The Froude-scaled hydraulic properties for the first scale test series are shown in Figure 4.9. For \( \lambda = 2 \), the drainage capacity was at its limit for \( t < 28 \) s, which led to higher values of \( Q_{b} \). For test \( \lambda = 1 \), drainage discharge \( Q_{dr} \) was underestimated, leading to an overestimation of the design breach discharge \( Q_{b,d} \). Consequently, \( Q_{b}(t) \) for \( \lambda = 1 \) was generally too low during the entire test. The values for \( Q_{b}(t \geq 40) \) of the tests \( \lambda = 0.5 \) and 2 are close, while the difference in the peak breach discharge \( Q_{M} \) was 1.1 l/s (10\% of \( Q_{M} \) or 15\% of \( Q_{b,d} \)). While the timing \( t_{M} \) of \( Q_{M} \) for \( \lambda = 0.5 \) and 1 was the same, it was reached five seconds earlier for \( \lambda = 2 \). This difference is partially attributed to the higher inflow discharge \( Q_{o} \) for \( \lambda = 2 \) due to the limited drainage discharge capacity for \( t < 28 \) s. The headwater level evolutions \( h_{o}(t) \) for \( \lambda = 0.5 \) and 1 are similar, with the maximum value \( h_{M} \) being slightly larger for \( \lambda = 0.5 \) due to the generally higher breach discharge value of \( Q_{b}(t) \). For \( \lambda = 0.5 \) at \( t \approx 185 \) s, \( h_{o} \) and \( Q_{b} \) suddenly rise and then gradually drop due to a major lump of wetted sediment falling into the breach channel and being gradually eroded. For \( \lambda = 2 \), the maximum headwater level \( h_{M} \) is reached at \( t = 9 \) s, about 4 s faster compared to the smaller test setups, which indicates faster erosion at breach start. In contrast, for \( t > 29 \) s, \( h_{o}(t) \) lies above the values of the small test setups with the difference increasing up to 5.5\% of the initial dike height \( w \).

These tests indicate that the choice of the breach discharge \( Q_{b} \) as design discharge instead of \( Q_{o} \) is adequate. For the largest test \( \lambda = 2 \), the drainage discharge capacity was too low during 28 s, which in turn led to higher values of the breach discharge. Surprisingly for \( t > 29 \) s, the headwater level descended more slowly compared to the smaller tests, since tests by Schmocker (2011) indicated that Froude similitude applies if \( t > 40 \) s. This result is even more surprising, since the breach discharge \( Q_{b} \) is chosen as the design discharge and the values were equal or above those of the smaller tests. Had the inflow discharge \( Q_{o} \) been chosen as design discharge (as e.g. Schmocker 2011), the constant inflow discharge value would have to be reduced (Figure 4.9c) and the difference in headwater levels \( h_{o}(t > 29) \) be expected to increase further (Figure 4.9a). Figure 4.9c shows that the drainage discharge \( Q_{dr} \) varies for larger sediment sizes relative to the headwater level and breach discharge. For the small sediment size with \( \lambda = 0.5 \), no significant variation for \( Q_{dr}(t > 0) \) is observed.
Experimental results of model limitation tests

Figure 4.9  Scale test series a: Comparison of hydraulic properties (a) headwater level, (b) breach discharge, (c) inflow discharge, (d) drainage discharge for $\lambda = (---) 0.5, (- -) 1, (-- ) 2$. (Tests 11/9/8)

The dimensionless streamwise profiles $Z(X,t_{ref})$ for the first scale test series are shown in Figure 4.10, the transverse profiles $Z(Y,t_{ref})$ in Figure 4.11. The evolutions of the streamwise dike profiles of the tree test setups $\lambda = 0.5, 1$ and 2 is similar for $y < 200$ mm. A distinctive convex curve shape is noted for $\lambda = 0.5$ at $y = 0$ mm, $t = 200$ s and $X = 0.7$. This is a result of the previously mentioned lump of sediment falling into the upstream pilot channel at $t \approx 185$ s deviating the flow towards the glass wall and leading to local erosion. The different evolution of the profiles is best seen for $y = 200$ mm. The profiles of $\lambda = 2$ erode faster for $t \leq 20$ s, which is attributed to effects of apparent cohesion, but erode considerably more slowly for $t \geq 40$ s. For $y = 400$ mm, major differences become visible in the profile shapes. At $t = 20$ s, first indications of erosion were only observed for $\lambda = 2$. For $t \geq 40$ s, the height in the upstream breach reach was similar for all $\lambda$, while the horizontal development – related to the erosion process typical for the hourglass breach shape – was clearly fastest in the downstream breach reach for $\lambda = 2$ and slightly in advance for $\lambda = 1$ as compared with $\lambda = 0.5$. This different evolution is attributed to a difference in apparent cohesion properties.
Experimental results of model limitation tests

Figure 4.10 Scale effects series a: Spatial dike breach evolution $Z(X,t_{ref})$ at streamwise sections $y_{ref} [\text{mm}] = (a,e) \ 0$, (b,f) 100, (c,g) 200, (d,h) 400 for $t_{ref} [\text{s}] = (\ -\ ) -0$, (---) 10, (- -) 20, (----) 40, (- -x-) 60, (-○-) 100 and (-□-) 200. $\lambda = (\ -\ ) 0.5$, (---) 1, (----) 2
The transverse profiles are displayed in Figure 4.11. At the upstream end of the pilot channel base at $x = 400$ mm, the transverse profiles look similar for tests $\lambda = 0.5$, 1 and 2 up to $t = 60$ s. For $t > 60$ s the profiles for $\lambda = 2$ are well above these of the other two tests at the base of the breach channel, while the profiles for $\lambda = 0.5$ tend to lie lower. In contrast, the slopes of the breach channel are eroded faster for $\lambda = 2$ compared to the smaller test setups during the entire test duration. A similar trend is observed in the different transverse profiles and also for $\lambda = 1$ in comparison with $\lambda = 0.5$, but to a much smaller degree. It was noted that for $\lambda = 2$, the flank of the breach channel reaches the channel wall at $Y = 1$ for $t \cong 45$ s, while this happens for $\lambda = 0.5$ only for $t \cong 180$ s. Thus, the typical hourglass shape does not evolve freely and the surface profiles will not be comparable for $t > 40$ s, since this has an influence on the breach channel hydraulics and the sediment balance. The different evolution of the breach flanks is attributed to the apparent cohesion, whose effect is larger for smaller grain sizes in $\lambda = 0.5$ compared with $\lambda = 1$, disappearing completely for $\lambda = 2$. Higher apparent cohesion stabilizes the breach flanks and leads to steeper flank slopes. As a result, the additional amount of eroded flank material will inevitably lead to increased accumulation at the base of the breach channel thereby influencing the hydraulic properties.

The previously described hydraulic properties in Figure 4.9 are explained using these profiles. Shortly after breach start at $t = 10$ s, the center and downstream reaches of the pilot channel are eroded, whereas the upstream base of the pilot channel is eroded only shortly before $t = 20$ s in the streamwise section $y = 0$ mm (Figure 4.10a,e). The control section therefore moves upstream for $t < 20$ s, as observed in the transverse sections. The transverse sections for $t = 10$ and 20 s at $x = 650$ mm (Figure 4.11b,f) indicate that the flank slopes of test $\lambda = 2$ are less steep. This indicates that the control section is wider compared to the smaller tests and, because of the constant inflow discharge condition, will lead to smaller values $h_M$ and $Q_M$ among other differences in the hydraulic properties.

This test series highlights that the apparent cohesion plays a decisive role in the dike breach development and the resulting flow hydrograph. Due to the differences in breach flank slopes and the resulting breach widening velocity, the flank of the largest test $\lambda = 2$ reached the channel wall long before the flanks of the smaller tests did, resulting in larger deviations of the breach topography, hydraulics and sediment transport for $t > 40$ s. Consequently, it should be avoided to restrict the development of the hourglass breach shape in the transverse direction for a better comparison.
Figure 4.11  Scale effects series a: Spatial dike breach evolution $Z(Y,t_{ref})$ at transverse sections $x_{ref}$ [mm] = (a,e) 400, (b,f) 650, (c,g) 900, (d,h) 1300 for $t_{ref}$ [s] = (−−−−) 0, (−−−−−) 10, (−−−−) 20, (−−−−−) 40, (−−−−−) 60, (−−−−−) 100 and (−−−−−) 200. $\lambda = (---) 0.5, (----) 1, (-----) 2$
Scale effects $b$

The Froude-scaled hydraulic properties for the second scale test series $b$ are shown in Figure 4.12. The hydraulic properties of this test series look similar to the previous because of the comparable relative breach discharges. The drainage discharge $Q_{dr}$ was underestimated for $\lambda = 2$ and 0.5, leading to by 10% too large values of $Q_{b}(t)$ for $\lambda = 2$, and 5% for $\lambda = 0.5$. Therefore, as in the previous series, $Q_{b}(t)$ for $\lambda = 1$ was generally lower during the whole test. For $\lambda = 2$, drainage was only briefly at its discharge capacity limit.

The difference in peak breach discharge $Q_{M}$ between the tests $\lambda = 0.5$ and 2 was 1 l/s (15% of $Q_{M}$ or 26% of $Q_{b,ref}$). As in the previous test series, the timing $t_{M}$ of $Q_{M}$ for $\lambda = 0.5$ and 1 was identical, while $Q_{M}(\lambda = 2)$ was again reached 5 s earlier. The evolution of headwater levels $h_{o}(t)$ for $\lambda = 0.5$ and 1 is again similar, with $h_{M}$ being slightly larger for $\lambda = 0.5$ due to the generally higher breach discharge value of $Q_{b}(t)$. In contrast to the previous test series for $\lambda = 2$, $h_{o}$ and $h_{M}$ agree well with test $\lambda = 0.5$ in terms of values and timing. For $t > 28$ s though, $h_{o}(t)$ again lies above these of the small test setups. The difference increases up to 5.2% of the initial dike height $w$ at $t = 60$ s and remains identical later, in contrast to the previous test series.

Figure 4.12  Scale test series b: Comparison of hydraulic properties (a) headwater level, (b) breach discharge, (c) inflow discharge, (d) drainage discharge for $\lambda = (---) 0.5, (- -) 1, (---) 2$. (Tests 12/13/14)
The dimensionless streamwise profiles $Z(X,t_{ref})$ for the second scale test series are shown in Figure 4.13, and the transverse profiles $Z(Y,t_{ref})$ in Figure 4.14. Due to measurement issues, the grid was partially not recognized by the photogrammetric system in parts of the breach channel roughly for $t \geq 60$ s, $X > 0.5$ and $0 < Y < 0.25$. This of course led to flawed profile shapes during triangulation in these areas.

The evolution of the streamwise dike profiles of the three test setups $\lambda = 0.5$, 1 and 2 is similar for $y < 200$ mm. Interestingly, the values for the profiles at the glass wall in test $\lambda = 2$ are lower compared to the smaller tests for $t = 100$ s and $X > 0.6$ and for $t = 200$ s and $X > 0.3$. Deviations are noted in the profiles at $y = 200$ mm for $t = 20$ to $40$ s and in $y = 400$ mm for $t = 100$ to $200$ s and are attributed to effects of apparent cohesion. The observations of the general breach development are similar to the previous scale test series, but deviations in the profile advance are far less pronounced; the breach process is slower due to a smaller design breach discharge.

The evolution of the transverse dike profiles of the three test setups $\lambda = 0.5$, 1 and 2 is similar for $x = 400$ mm, the profile of $\lambda = 2$ tends to slightly lie above the other two. For $x = 650$ to $900$ mm, the flanks of the breach channel tend to be less steep as the grain size increases, e.g. the flanks of $\lambda = 2$ are less steep compared to $\lambda = 1$ and those of $\lambda = 1$ compared to $\lambda = 0.5$. As previously noted, this is attributed to apparent cohesion, but the effect appears to be less pronounced compared to the first scale test series. The breach flank at transverse section $x = 650$ mm reaches the channel wall at $Y = 1$ at test end for $t \approx 200$ s. The profiles of the three tests tend to be similar at the base of the breach channel.

The previously described hydraulic properties shown in Figure 4.12 are explained by these profiles. Shortly after breach start at $t = 10$ s, the downstream reach of the pilot channel is eroded and reaches its upstream base at $t = 20$ s in the streamwise section $y = 0$ mm (Figure 4.13a,e). The control section therefore moves upstream for $t \leq 20$ s, as is observed at the transverse sections $x = 900$ and $650$ mm. The transverse profiles for $t = 10$ s; $x = 900$ mm (Figure 4.14c) and for $t = 20$ s; $x = 650$ mm (Figure 4.14f) show that the flank slopes of test $\lambda = 2$ are the least steep and those of test $\lambda = 0.5$ are the steepest. This indicates that the control section is the wider the larger $\lambda$ and – because of the constant inflow discharge condition – will lead to a smaller $h_M$ and $Q_M$. Interestingly, the hydraulic properties and the topography seem better scalable compared to the previous test series, specifically for $h_a(t)$ and the streamwise profiles $Z(X,t_{ref})$. This is due to the hourglass shape not being confined during test duration. Differences in hydraulic properties and sediment transport are mainly influenced by apparent cohesion, therefore.
Figure 4.13  Scale effects series b: Spatial dike breach evolution $Z(x, t_{ref})$ at streamwise sections $y_{ref} [mm] = (a,e) 0, (b,f) 100, (c,g) 200, (d,h) 400$ for $t_{ref} [s] = (\cdots) 0, (\cdots) 10, (\cdots) 20, (\cdots) 40, (\cdots) 60, (\cdots) 100, (\cdots) 200$. $\lambda = (\cdots) 0.5, (\cdots) 1, (\cdots) 2$
Experimental results of model limitation tests

Figure 4.14 Scale effects series b: Spatial dike breach evolution $Z(Y,t_{ref})$ at transverse sections $x_{ref} \text{[mm]} = (a,e) 400, (b,f) 650, (c,g) 900, (d,h) 1300$ for $t_{ref} \text{[s]} = ($---$) 0, ($\cdots$) 10, ($-$) 20, ($\cdots$) 40, ($-$x-) 60, ($\cdots$-) 100, ($\cdots$-) 200. $\lambda = ($---$) 0.5, ($$-) 1, ($$-) 2
Experimental results of model limitation tests

Scale effects

The Froude-scaled hydraulic properties for the third scale test series are shown in Figure 4.15. Contrary to the two previous test series, the drainage discharge $Q_d(t)$ was underestimated for $\lambda = 2$ and overestimated for $\lambda = 0.5$, which led to deviations of $Q_b(t)$ by $-6\%$ and $+2\%$, respectively. $Q_M$ of 10.4 l/s was first reached at $t = 19$ s for $\lambda = 2$, followed by $\lambda = 1$ with $Q_M = 11.1$ l/s at $t = 26$ s, and $\lambda = 0.5$ with $Q_M = 12.1$ l/s at $t = 35$ s. The headwater level evolution $h_o(t)$ is similar for all three tests while rising, with the maximum headwater level $h_M$ higher for $\lambda = 0.5$ compared to the two other test, overtopping the dike crest for a short moment without influencing the general dike breach.

Major lumps of wetted sediment regularly fell from the breach flanks into the breach channel for all three tests. This is observed from the breach discharge curves $Q_b(t)$ and often to a lesser degree in the headwater level curves, e.g. for $\lambda = 2$ at $t = 24, 33, 52, 90$ s, for $\lambda = 1$ at $t = 17, 52, 153, 180, 189, 207$ s, and for $\lambda = 0.5$ at $t = 17, 31, 81, 212$ s. The lumps of wetted sediment and the time intervals appeared to be smaller, the smaller the effects of apparent cohesion and the larger $\lambda$ were, e.g. $\lambda = 2$ resulted in $\Delta t \approx 10, 20, 40$ s, $\lambda = 1$ in $\Delta t \approx 35, 100, 40$ s, and $\lambda = 0.5$ in $\Delta t \approx 15, 50, 130$ s. For $\lambda = 2$, the headwater level drops faster after having reached $h_M$ compared to the smaller test setups, indicating faster erosion at breach start. For $t > 49$ s, $h_o(t)$ then lies above the values of the smaller tests and the difference increases up to $6.6\%$ of the initial dike height $w$.

The dimensionless streamwise profiles $Z(X,t_{ref})$ for the scale test series are shown in Figure 4.16, and the transverse profiles $Z(Y,t_{ref})$ in Figure 4.17. The evolution of the streamwise dike profiles of the three test setups $\lambda = 0.5, 1$ and 2 is similar for $y = 0$ mm. For $y \leq 200$ mm, the profiles of $\lambda = 2$ tend to erode fastest and those of $\lambda = 0.5$ most slowly for $t \leq 20$ s. For submerged dike portions, this trend is reversed for longer times. In the non-submerged areas, e.g. for $y = 200$ mm and $X \approx 0.6$ at $t = 40$ s, the profile for smaller $\lambda$ tends to be higher due to apparent cohesion. For $y = 400$ mm, test $\lambda = 2$ erodes fastest and forms the typical hourglass shape. The deviation of the profile shape for $\lambda = 1$ at $t = 200$ s and $X \approx 0.6$ is due to a large slump of sediment detaching itself slowly from the main dike body. The evolution of the streamwise profiles is in accordance with the previous test series (e.g. scale effect a as discussed above) demonstrating an important influence of apparent cohesion.
Experimental results of model limitation tests

Figure 4.15 Scale test series c: Comparison of hydraulic properties (a) headwater level, (b) breach discharge, (c) inflow discharge, (d) drainage discharge for \( \lambda = (- -) 0.5, (- -) 1, (-) 2 \). (Tests 15/16/17)

The evolution of the transverse dike profiles of the three test setups \( \lambda = 0.5, 1 \) and 2 is similar for \( x = 400 \text{ mm} \), with the profile of \( \lambda = 2 \) tending to slightly lie above the others. For \( x = 650 \) to \( 900 \text{ mm} \), the flanks of the breach channel tend to be less steep the larger the grain size is, e.g. the flanks of \( \lambda = 2 \) are less steep compared to \( \lambda = 1 \) and those of \( \lambda = 1 \) compared to \( \lambda = 0.5 \). As previously described, this is attributed to apparent cohesion, but the effect appears to be smaller compared to the first two scale test series. It was noted that for \( \lambda = 2 \), the flank of the breach channel reaches the channel wall at \( Y = 1 \) for \( t \approx 60 \text{ s} \), while this occurs for \( \lambda = 1 \) only for \( t \approx 200 \text{ s} \). For \( \lambda = 0.5 \), the breach flank does not reach the channel wall at \( Y = 1 \). This means that the typical hourglass shape will not evolve freely and the surface profiles will not necessarily be comparable for \( t > 60 \text{ s} \) due to effects on the breach channel hydraulics and the sediment balance. In this test series, though, the influence of the hourglass shape constriction appears to be moderate. The profiles at the base of the breach channel close to the glass wall tend to be more elevated the larger \( \lambda \), which has also been observed in the first test series.
Figure 4.16  Scale effects series c: Spatial dike breach evolution $Z(X, t_{ref})$ at streamwise sections $y_{ref}[\text{mm}] = (a,e) 0$, (b,f) 100, (c,g) 200, (d,h) 400 for $t_{ref} [\text{s}] = (- -) 0$, ( - ) 10, ( - - ) 20, ( - - - ) 40, ( -x-) 60, ( -○-) 100, ( -□-) 200. $\lambda = (---) 0.5$, (――) 1, (——) 2
Figure 4.17  Scale effects series c: Spatial dike breach evolution $Z(Y,t_{ref})$ at transverse sections $x_{ref} [\text{mm}] = (a,e) 400, (b,f) 650, (c,g) 900, (d,h) 1300$ for $t_{ref} [\text{s}] = (-) 0, (--) 10, (-.) 20, (---) 40, (-\times-) 60, (-\odot-) 100, (-\square-) 200. \lambda = (-) 0.5, (--) 1, (---) 2.
The previously described hydraulic properties in Figure 4.15 are explained using these profiles. Shortly after breach start at $t = 10 \text{ s}$, the downstream reach of the pilot channel is eroded and reaches its upstream base at $t = 20 \text{ s}$ in the streamwise section $y = 0 \text{ mm}$ (Figure 4.16a,e). The control section therefore moves upstream for $t \leq 20 \text{ s}$, as observed in the transverse sections. The transverse sections for $t = 10 \text{ s}$ and $x = 900 \text{ mm}$ (Figure 4.17c) and for $t = 20 \text{ s}$ and $x = 650 \text{ mm}$ (Figure 4.17f) show that the flank slopes of test $\lambda = 2$ are less steep. This indicates that the control section is wider compared to the smaller tests and – because of the constant inflow discharge condition – will lead to a smaller $h_M$ and $Q_M$.

Al-Riffai (2014) assessed the Froude criterion using tilted (distorted) and quasi-exact geometric scales for dikes of heights $w = 300$ and 100, 150 and 300 mm, respectively, under very low inflow. Tilted geometric scale tests were used because the Reynolds number is higher and dynamic similarity is improved, and the larger vertical scale results in a more accurate depth measurement (Chanson 2004), but the lateral and vertical time-scales represented by the breach channel morphology become irrelevant due to model distortion (Julien 2002). The flow hydrographs of the tilted scaled tests by Al-Riffai (2014) showed good agreement with the benchmark test. The quasi-exact geometric scaled tests were geometrically scaled according to Froude similitude, except for the channel width and the sediment size, which were kept constant. The flow hydrographs of the quasi-exact scaled tests showed large differences in peak discharge values: While the benchmark test with $w = 300 \text{ mm}$ reached $Q_M = 69 \text{ l/s}$, $Q_M = 42 \text{ l/s}$ for $w = 100 \text{ mm}$. Compared to these results, the previously presented results of the scale effect series are considered excellent, therefore. Concluding remarks on scale effects are presented in 4.8.

4.6 Sediment erosion volume

The evolution of the eroded sediment volume $V_E(t)$ was determined for the previous test series involving test repeatability, symmetry and scale effects in Figure 4.18. The data were extracted using the sediment surfaces determined using data from the photogrammetric measurement and the Delaunay triangulation method as shown in Figure 3.26. The volume beneath the sediment surface was determined in the reach $0 \leq x \leq L_D$ and $0 \leq y \leq b$. Deviations due to photogrammetry-related refraction effects (Chapter 3) have a slight influence on the results. Furthermore, at breach start and at $t \cong 10 \text{ s}$, the applied method appears sensitive. For Test 2, therefore, the reference volume was selected for dry conditions and not at $t = 0$, as was done for the remaining tests. For the scale effect series, it
was not possible to capture the entire dike surface using the photogrammetric system for the largest tests $\lambda = 2$. Therefore, only the reach $0 \leq x_{\text{ref}} (\lambda = 1) \leq 1020$ mm was used for comparison of the eroded dike volumes for the scale effect tests.

The eroded dike volume $V_E(t)$ curves for the \textit{repeatability} Tests 1 to 3 agree well (Figure 4.18a). The eroded dike volume for the \textit{symmetry} tests is shown in Figure 4.18b; the half-model is used as reference test. The curves for both tests are similar in shape, but for $10 \, \text{s} < t < 40 \, \text{s}$ the full-model dike volume (Test 10) decreases faster than in Test 9. For $t > 40 \, \text{s}$, a similar dike volume is eroded, which leads to near parallel curves and is plausible when comparing the hydraulic data of these tests. Figure 4.18c shows a good agreement of the Froude-scaled dike volume evolution for Tests 8, 9 and 11 of \textit{scale test series a}. At breach start for $t < 10 \, \text{s}$, the volume of the largest dike (Test 8, $\lambda = 2$) erodes marginally faster than the others, in accordance with previous observations in the evolution of the breach discharge and topography profiles. For $10 \, \text{s} < t < 60 \, \text{s}$ though, the evolution of the scaled $V_E(t)$ is similar. For the remaining test duration ($t > 60 \, \text{s}$), $V_E(t)$ is slightly larger for $\lambda = 2$ compared to the other tests. As mentioned above, this deviation is attributed to the faster transverse breach development due to missing effects of apparent cohesion for $\lambda = 2$ and the development of the hourglass breach shape being restricted in the transverse direction due to the channel wall, which has an impact on the breach hydraulics and thus on the breach shape and eroded sediment volume.

The eroded dike volume for \textit{scale test series b} is shown in Figure 4.18d. The eroded volume is largest again for $\lambda = 2$ (Test 14) for $t < 20 \, \text{s}$. For $t > 20 \, \text{s}$, the evolution of the eroded dike volume is similar for the three tests, since the curves are almost parallel. Only for $\lambda = 1$ (Test 13), the eroded dike volume increases more slowly as compared with the other tests for $t > 60 \, \text{s}$. This is attributed to apparent cohesion stabilizing the transverse breach slope, as observed at $x = 450$ mm in Figure 4.14f when comparing the profiles for $t = 60$ and $200 \, \text{s}$. Only at $t = 229 \, \text{s}$ this breach flank collapses into the breach. Therefore, apparent cohesion can lead to differences limited in time of the eroded sediment volume $V_E(t)$. In \textit{scale test series c} (Figure 4.18e), as for the previous scale series tests, the eroded volume is again slightly larger for $\lambda = 2$ (Test 14) if $t < 20 \, \text{s}$, while the curves for $V_E(t)$ agree well for all three tests for $t > 20 \, \text{s}$.

The analysis of temporal eroded dike volume was useful to validate and extend previous results in which the hydraulic properties and the topography profiles for different test series were analyzed. Test \textit{repeatability} was again proven. For the symmetry tests, the previous observations were confirmed concerning a limited faster erosion of the full-
model test compared with the half-model test. These two tests also validate the evaluation method using dikes of height $w = 200$ and $300$ mm. For the three scale test series $a$, $b$, and $c$, the eroded dike volume of the largest tests $\lambda = 2$ was slightly larger at test start compared to the smaller tests, which is attributed to the lack of apparent cohesion and the resulting smaller breach channel flank slopes for the sediment grain size $d = 3.78$ mm. This confirms previous observations. Otherwise, the curves of the respective three tests agree well when the hourglass is not restricted in transverse direction due to channel wall presence. Interestingly, the curves of scale test series $a$ and $c$ are similar, indicating a negligible effect of the sediment diameter on the eroded sediment volume for $0.43 \ mm \leq d \leq 3.78$ mm as will be discussed below.

Figure 4.18 Eroded dike volume $V_E(t)$ for spatial dike breach tests series: (a) Repeatability, (b) symmetry, (c) scale effects $a$, (d) scale effects $b$, (e) scale effects $c$
4.7 Breach channel shape and hydraulic properties

*Hourglass shape, breach inflow crest and submerged breach profiles*

The procedure for extracting the hydraulic parameters during a spatial dike breach is subsequently presented for Test 1. For the extraction of mean hydraulic values, the breach channel limit of submerged and non-submerged breach areas is of interest. This limit typically develops into the hourglass shape, as e.g. described by Coleman *et al.* (2002). To extract the coordinates of these breach shape features, the respective water surface profile along the glass wall (Figure 4.19a) was extrapolated along the transverse $y$-axis and its intersection with the triangulated dike surface was automatically determined at selected times $t$. The resulting hourglass shapes are shown in Figure 4.19b. The breach widens rapidly for $t < 40$ s, developing into the typical hourglass shape from $t > 20$ s. For $t \geq 40$ s the hourglass shape expands in the streamwise and transverse directions. The point closest to the breach channel axis moves downstream for $t \geq 40$ s, while its transverse movement is then negligible, and therefore also the breach channel widening.

![Figure 4.19](image-url)  
*Figure 4.19*  
Spatial dike breach evolution for Test 1 with (a) streamwise (−) sediment surface profiles $z(x)$ and (−) water surface profiles $h(x)$ at $y = 0$ and $t$ [s] = (−−) 10, (- - -) 20, (− − −) 40, (−x−) 60, (−○−) 100, (−−−−) 200, and (−) initial dike shape at $y > 200$ mm, (b) plan of (−) hourglass shape, (−) inflow crest and (−) initial pilot channel shape, (c) inflow crest profile points projected on $y$-$z$ plane with (−) initial pilot channel shape, and (d) dimensionless comparison of hourglass shapes in $X$-$Y$ plane. Points determined at $t$ [s] = (▲) 10, (●) 20, (■) 40, (x) 60, (○) 100, (□) 200.
The breach inflow crest is the connection between the highest breach channel points along the streamlines. The inflow crest of the breach channel was manually determined in a simplified way each 50 mm across the breach channel considering the streamwise breach surface profiles \( z(x) \). A qualitative fit was inserted in Figure 4.19b,c due to the irregularity of many breach profiles. The breach channel inflow crest only develops for \( t \geq 20 \) s, consistent with observations in Figure 4.19a,b. It develops transversally during breach widening for \( t \leq 40 \) s, from when it slowly moves upstream. For \( t > 40 \) s, the crest increases its radius and gradually widens while moving upstream. The projection of the inflow crest profile data onto the \( y-z \) plane is shown in Figure 4.19c. The profiles are regularly spaced, indicating that the vertical erosion slows down during the dike breach.

The normalized hourglass shapes \( Y(\lambda) \) are shown in Figure 4.19d, with the normalized width \( Y = y_{hg}/y_{hg,d} \), where \( y_{hg} \) is the transverse position of each point relative to the smallest \( y_{min} \) of the hourglass shape and \( y_{hg,d} \) is the difference between the extremes \( y_{max} \) and \( y_{min} \) (Figure 3.12). The normalized hourglass length is \( X = x_{hg}/x_{hg,d} \) with \( x_{hg,d} \) as the difference of the \( x \)-values corresponding to \( y_{max} \) and \( y_{min} \), \( x_{hg} \) being the streamwise position of each point relative to \( x_{min} \). The normalized hourglass shapes \( Y(\lambda) \) are similar. At \( t = 40 \) s, the shape deviates slightly from the general trend, which is partially attributed to difficulties in determining the upstream end of the hourglass shape in the transverse direction, since the data were interpolated on a 10 mm grid along the x-axis.

From the submerged breach portions of Figure 4.19b, submerged bed profiles \( z(y) \) at \( x = 450 \) mm were extracted as shown in Figure 4.20a, with the highest point located at the water surface elevation \( h(x=450 \) mm). Figure 4.20b shows the normalized submerged breach profiles \( Z_b(Y) \) with \( Y_b = y/b \) and \( Z_b = z/h_b \) with \( y \) and \( z \) as the respective coordinates in the submerged breach half-profile of width \( b \) and water depth \( h_b(y=0) \).

**Figure 4.20** Spatial dike breach evolution of Test 1 with (a) transverse submerged sediment surface profiles \( z(y) \) at \( x = 450 \) mm, (b) dimensionless transverse submerged sediment surface profiles \( Z_b(Y) \) at \( t \) [s] = (---) 10, (- - -) 20, (---) 40, (---) 60, (---) 100, (+-+) 200, (-----) initial breach shape, (-----) parabolic approximation \( Y_b = Z_b b^{0.5} \) for \( n = 2 \).
Hydraulic characteristics

To determine the hydraulic characteristics of Test 1, the following available data were combined, namely:

- Breach discharge $Q_b(t)$ (as determined in 4.3)
- Submerged transverse breach surface profile $z(y)$ (as determined in 4.7)
- Streamwise water surface profile $h(x)$ (as determined in 4.7)

The data were interpolated on a 10 mm grid along the $x$-axis. The cross-sectional flow area $A(x)$ was then determined using $z(y)$ and $h(x)$ from Figure 4.20a, in which $h(x)$ defines the highest point of the submerged breach profile $z(y)$ at the position $x$. The mean velocity $v(x)$ was subsequently determined by dividing $Q_b(t)$ from Figure 4.3b and the respective value of $A(x)$. The energy head $H$ was computed by adding the potential and kinetic energy heads $h(x) + v^2(x)/2g$. The Froude\(^1\) number was determined as $F(x) = Q_b b_{bh}^{1/2} g^{-1/2} A^{-3/2}$.

The resulting hydraulic data $F(x)$, $H(x)$, $A(x)$ and $v(x)$ are shown in Figure 4.21 together with the breach surface section $z_b(x)$ and the water surface profile $h(x)$ along the glass wall for $t = 10, 20, 40, 60, 100, 200$ s with $Q_b(t) = 1.3, 6.8, 16.2, 17.3, 14.7$ and $13.5$ l/s, respectively. The initial dike shape $z(x)$ for $y > 200$ mm is included as reference. Since all data except for $Q_b(t)$ refer to $x$, the descriptions are simplified for better readability. At $t = 10$ s, the downstream portion of the pilot channel is eroded and a steep breach channel slope develops for $x > 500$ mm. The water surface profile $h$ and energy head $H$ drop, while $v$ and $F$ increase for $x > 200$ mm until supercritical flow $F > 1$ is reached at $x > 480$ mm. The cross-sectional flow area $A$ drops linearly up to the pilot channel at $x = 300$ mm, from where it drops less. For $x > 500$ mm, the cross-sectional flow area $A$ drops to low values, due to a lack of measurement points in the breach channel. Therefore, due to the fixed value for $Q_b$, $v$ and consequently $H$ and $F$ increase to unrealistic values and were therefore dropped. Note that $H$ drops and slightly rises again at all times in the reach between where $h$ starts dropping and $h$ crosses the initial dike shape in grey, which is of course physically incorrect. This is attributed to the simplification of the 3D flow (e.g. curved inflow crest) with transverse cross-sections, leading to abrupt changes in the cross-sectional area, but also refraction effects due to the 2D water surface profile assumption described in Chapter 3.

\(^1\)Alternatively, Frank and Hager (2015) used parabolic submerged transverse sediment profiles to determine critical flow depths $h_c(x)$ and critical flow velocities $v_c(x)$ to determine the Froude number $F(x)$, in accordance with Bolrich (2013). This method overestimates Froude numbers slightly though and is not retained for further evaluations.
Experimental results of model limitation tests

Figure 4.21 Hydraulic properties of Test 1 at $t$ [s] = (a) 10, (b) 20, (c) 40, (d) 60, (e) 100, (f) 200 with (-- streamwise breach surface section $z_b(x, y=0)$, (--- water surface profile $h(x, y=0)$, (----- Froude number $F(x)$ [-], (----- energy head $H(x)$ [mm], (···) flow cross-section $A(x)$ [2*dm²] and (···) mean flow velocity $v(x)$ [m/s]

At $t = 20$ s, the erosion of the surface profile reaches the upstream reach of the pilot channel. The values of $h$, $v$, $A$ and $F$ increase, with supercritical flow through the breach channel for $x > 450$ mm. The energy head drops significantly for $x > 520$ mm, which is partially attributed to high transverse water surface slopes and inaccuracies of the breach topography. At $x = 770$ mm, the water surface profile crosses the original dike surface. The determination of the hydraulic properties is not possible past this limit, since the cross-section would encompass the entire channel width, overestimating $A$ and resulting in unrealistic hydraulic values, which were therefore dropped also for the remaining times. At $t = 40$ and 60 s, the upstream inflow crest is eroded, $A$ increases fast and values of $h$ are high in the breach channel. The energy head drops moderately in the streamwise direction while the remaining values are similar as for $t = 20$ s. For larger test times $t = 100$ and 200 s, all values except for $A$ drop, resulting in supercritical flow $F > 1$ at $t = 100$ s for $x > 570$ mm and subcritical flow $F < 1$ at $t = 200$ s.
4.8 Summary

Hydraulic model tests were conducted to determine the modelling limitations, and to assure that the results obtained in the following test series are reliable and follow the Froude similitude.

First, the influence of seepage was determined, which can lead to dike instabilities. This can result in dike failure not attributed to surface erosion during 2D and 3D dike breach tests thereby influencing the hydraulic test properties as proven in the repeatability tests. This issue was resolved by installing a well-positioned drainage of sufficient discharge capacity. The drainage discharge ranged from 4 to 29% of the inflow discharge, so that it cannot be neglected. For better comparability between the tests, the breach discharge was therefore considered relevant and determined by subtracting the drainage discharge from the inflow discharge.

In a second step, test repeatability was considered. The tests proved to be repeatable if dikes of height $200 \text{ mm} \leq w \leq 600 \text{ mm}$ are employed in terms of flow hydrographs, topography profiles, and eroded dike volumes. This demonstrates that the hydrograph evaluation method, the stereo-photogrammetric system and the subsequent data treatment are repeatable and reliable.

The assumption of symmetry was checked in a third step. The results of the simplified half-test model indicated a satisfactory correlation with those of the full model. Therefore, data extracted from half-model dike breach tests adequately describe the full-model breach. A slight stabilizing effect of the glass wall was noted, however.

Froude similitude was tested for dikes of initial crest height $w = 150$, $300$ and $600$ mm. Three test series were conducted using different design breach discharges and sediment grain sizes. The largest test setup with $w = 600$ mm, sediment size of $3.78$ mm and constant inflow discharge of $50 \text{ l/s}$ proved to be the upper limit for successful testing. A comparison of the scaled results indicates that the breach flanks tend to erode with a less steep surface slope for larger sediment size, leading to a wider breach. The surplus of eroded material deposits in the main breach channel, lifting the breach base and influencing the hydraulic properties of the dike breach. These results therefore indicate that apparent cohesion has an effect on the breach process in terms of breach topography evolution and flow hydrographs. Also, as the breach flanks reach the channel wall, the evolution of the hourglass shape is constricted, which leads to deviations in the physical modelling of the breach hydraulics and sediment balance. This can be countered by using a wider channel.
As described in Chapter 2, plane dike breach tests of Sametz (1981) show that the sediment grain size has no influence on the erosion rate, while plane dike breach scale tests of Schmocker (2011) indicate, that when the dike is not influenced by seepage-induced failure during the main breach phase for $t < 40$ s, the dike appears to be scalable, while for $t > 40$ s scalability proved to be good. Note that for 2D dike breaches, the breach is completely submerged and apparent cohesion has no effect.

The observed differences between 2D and 3D dike breaches are therefore attributed to apparent cohesion for the hydraulic boundary condition of constant inflow discharge. Results from the literature and Chapter 6 show that if breach discharge is large, as e.g. for large reservoir water surface areas, the effect of apparent cohesion is negligible. Based on these arguments, 3D dike breaches are scalable according to Froude similitude for the studied parameters, namely sediment grain sizes $d = 0.43$ to 3.78 mm (or a particle parameter $D* = 9.7$ to 85.4), dike heights $w = 0.15$ to 0.6 m and design breach discharges $Q_{b,d} = 0.65$ to 41.8 l/s for half-model tests.

As the sediment erosion rate is independent of sediment grain size (Sametz 1981), using a similar (or equal) sediment grain size in the model and the prototype should be considered to reduce effects of apparent cohesion on the breach flanks for the tested range of sediment grain size. The presented Froude-scaled test series are the first fully successful undistorted Froude scale tests for dike breaches adding to the distorted Froude scale tests by Al-Riffai (2014). Therefore, the tests of seepage, repeatability, symmetry, and Froude similitude indicate that the test setup is well suited to analyze the breach process of non-cohesive dikes and to generalize the results for prototype dike breaches in the test range.
5 Experimental results of parametric study

5.1 Introduction

To evaluate possible effects of different parameters on the spatial dike breach, tests were conducted using the test setup as described in Chapter 3. The standard dike properties are as follows: dike height $w = 300$ mm, distance of upstream dike toe from upstream channel wall $x_D = 3.44$ m, dike width $b = 1000$ mm, crest length $L_K = 100$ mm, pilot channel notch height $w_p = 100$ mm, top width $b_p = 200$ mm and bottom width $b_{p,u} = 0$, uniform non-cohesive sediment of grain size $d = 1.75$ mm. All tests were conducted under constant inflow discharge. The following parameters were varied:

- Design breach discharge $Q_{b,d}$
- Sediment grain size $d$
- Initial breach width $b_{p,u}$
- Reservoir water surface area $A_R$

For each test series, the headwater level $h_o(t)$, the breach discharge $Q_b(t)$, the eroded sediment volume $V_E(t)$, and the longitudinal and transverse profiles are comparatively described.

5.2 Breach discharge

Four tests (Tests 18 to 21, Table 3.3) were conducted with design breach discharges $Q_{b,d} = 2.2, 4.4, 9.2$ and $17.6$ l/s, respectively, by setting constant inflow discharges to $Q_o = 2.8, 5, 10.2, 18$ l/s. The drainage discharge was underestimated for Test 20, leading to a 7% higher design breach discharge than planned.

The hydraulic properties are compared in Figure 5.1. Considering the temporal headwater level evolution $h_o(t)$, the maximum $h_M$ is reached faster and its value is larger the higher $Q_{b,d}$, which is also the general trend of $h_o(t)$. The same temporal trend is further noted for the maximum breach discharges $Q_M$. For larger values, the maximum breach discharge is reached faster. For $Q_{b,d} = 17.6$ l/s, $Q_M = 25.5$ l/s is reached at $t = 38$ s, while for $Q_{b,d} = 2.2$ l/s, $Q_M = 6.8$ l/s is reached at $t = 82$ s. For smaller discharges, the relative maximum $Q_M / Q_{b,d}$ is larger though, e.g. 145% for $Q_{b,d} = 17.6$ l/s and 310% for $Q_{b,d} = 2.2$ l/s. This difference in relative peak discharge is attributed to the effect of the upstream reservoir.
Experimental results of parametric study

Figure 5.1  Effect of discharge: Comparison of hydraulic properties (a) headwater level $h_o(t)$, (b) breach discharge $Q_b(t)$ for $Q_{b,d} [l/s] = (---) 2.2, (- -) 4.4, (--) 9.2, (-•-) 17.6$ (Tests 18/19/20/21)

For small discharges, the value resulting from the accumulated volume in the reservoir $Q_R(t)$ is a more important component of the breach discharge $Q_b(t)$ than the inflow discharge $Q_o$ right after breach initiation. For large discharges on the contrary, the reservoir volume is negligible.

The temporal evolution of the eroded volume $V_E(t)$ for different design breach discharges $Q_{b,d}$ is shown in Figure 5.2. The dike volume is eroded faster for larger $Q_{b,d}$ and $t < 60 \, s$. For $t \geq 60 \, s$ though, the curve evolution is similar for all $Q_{b,d}$, which indicates that the erosion rate is comparable.

Figure 5.2  Effect of discharge: Comparison of eroded dike volume $V_E(t)$ for $Q_{b,d} [l/s] = (---) 2.2, (- -) 4.4, (--) 9.2, (-•-) 17.6$
The topographic breach profiles are compared in Figure 5.3 with (a) to (d) for the streamwise and (e) to (h) for the transverse sections from the initial dike shape up to $t = 200$ s. Figure 5.3a shows the streamwise sections at the breach centerline ($y = 0$ mm). For $t = 20$ s, the breach process still develops for the smallest discharge $Q_{b,d}$, while for the largest discharge the downstream breach slope is fully developed and the upstream portion is about to erode. The breach channel slope is large for small breach discharges at $t = 60$ s, while these are almost equal and the profile shapes similar for all tests at $t = 200$ s. Analogous properties are also observed for the streamwise sections $y = 100$ and 200 mm, the profile shapes are similar in the submerged flow portions but lower for larger $Q_{b,d}$. Note the still non-eroded portion in Figure 5.3c due to the hourglass shape from $x = 800$ to 1100 mm. For $y = 500$ mm, the characteristic erosion pattern due to the typical hourglass breach shape is noted except for the small discharge test.

The transverse section $z(y)$ at $x = 400$ mm at the upstream end of the pilot channel base is shown in Figure 5.3e. At $t = 20$ s, practically no erosion is visible. At $t = 60$ s, the breach develops both horizontally and vertically. This process is faster the larger $Q_{b,d}$. The horizontal erosion velocity increases with $Q_{b,d}$. The vertical erosion advance is also fast at breach start for large $Q_{b,d}$, but reaches a similar height close to the glass wall at $t = 200$ s. The profile shapes $z(y)$ change from the parabola-like shape at $t = 60$ s to a trapezoidal-like shape at $t = 200$ s. These shapes are also noted for $x = 650$ and 900 mm with the same general breach process, but with a more pronounced vertical erosion for all $Q_{b,d}$.

As a conclusion, higher design breach discharges $Q_{b,d}$ lead to higher maximum headwater levels $h_M$, smaller relative maximum discharges $Q_M/Q_{b,d}$, a faster time-to-peak $t_M$, faster erosion of the dike volume $V_E$, faster vertical erosion until a minimum equilibrium level is reached, and a faster horizontal erosion during the entire process as compared with smaller design breach discharges $Q_{b,d}$. 
Figure 5.3  Effect of discharge: Spatial dike breach evolution $z(x)$ at $y \text{ [mm]} = \begin{align*} (a) & \ 0, \ (b) & \ 100, \ (c) & \ 200, \ (d) & \ 500, \ \text{and} \ z(y) \text{ at } x \text{ [mm]} = \begin{align*} (e) & \ 400, \ (f) & \ 650, \ (g) & \ 900, \ (h) & \ 1300 \ \text{for} \ t \text{ [s]} = \begin{align*} (\cdot-\cdot) & \ 0, \ (-\cdot-\cdot) & \ 20, \ (-x-) & \ 60, \ \text{and} \ (-\square-) & \ 200. \ \text{Test} \ (-\cdot-\cdot) & \ 18, \ (-\cdot-\cdot) & \ 19, \ (-x-) & \ 20, \ (-\square-) & \ 21
5.3 Sediment grain size

Tests 22, 20 and 23 were conducted using different sediment grain sizes $d = 0.86$, 1.75 and 3.78 mm, respectively. The design breach discharge $Q_{b,d}$ was set to 9.2 l/s for all tests. Due to the different sediment permeability properties, drainage discharges were different and inflow discharges were set to $Q_o = 9.6$, 10.2 and 11.5 l/s, therefore. The hydraulic properties are compared in Figure 5.4. The temporal headwater level evolution $h_o(t)$ is similar for the three tests. The maximum headwater level is slightly lower and reached earlier for the larger the sediment grain size. The breach discharge $Q_b(t)$ increases slightly faster for the largest sediment grain size $d$ but reaches a smaller maximum breach discharge $Q_M$ compared to the test with the smallest grain size. At $t = 170$ s, the breach discharges tend to the same value of $Q_b = 10$ l/s. The temporal evolution of the eroded dike volume $V(t)$ for different sediment sizes $d$ is shown in Figure 5.5. For the tested sediment, the erosion speed increases slightly with $d$ for $t > 10$ s, but decreases again for $t > 60$ s, until the eroded sediment volume is almost identical at $t = 200$ s. Faster erosion for coarse sand compared to fine sand was also observed by Schmocker (2011) for 2D dike breaches and Pickert et al. (2011) for 3D dike breaches.

![Figure 5.4](image_url)  
**Figure 5.4** Effect of sediment grain size: Comparison of hydraulic properties for (a) headwater level $h_o(t)$, (b) breach discharge $Q_b(t)$ for $d$ [mm] = (— —) 0.86, (- -) 1.75, (— —) 3.78 (Tests 22/20/23)
The experimental results of parametric study

Figure 5.5  Effect of sediment grain size: Comparison of eroded volume for $d$ [mm] = 
(- -) 0.86, (- -) 1.75, (---) 3.78

The topographic profiles are compared in Figure 5.6 (a) to (d) relating to the streamwise $z(x)$, and (e) to (h) to the transverse sections $z(y)$ from the initial dike shape up to $t = 200$ s. Considering the streamwise sections, the erosion process is fastest for the largest $d = 3.78$ mm for $y = 0$ to 200 mm and $t = 20$ to 60 s, while it tends to be slowest for small $d = 0.86$ mm, as also noted by Schmocker (2011) for the 2D dike breach process. At $t = 200$ s, the profile $z(x)$ for $d = 3.78$ mm has a slightly undular shape with the upstream crest slightly higher as compared with the other two tests, whose profiles do not reveal any undulations and are of nearly equal shape. For $y = 500$ mm, the characteristic erosion pattern due to the typical hourglass breach shape is again observed. The erosion speed increases with $d$, which is partially attributed to effects of apparent cohesion.

The evolution of the transverse dike profiles is similar for the three tests with different sediment grain sizes $d$. The vertical and horizontal erosion is faster for $d = 3.78$ mm as compared with $d = 0.86$ mm at $x = 650$ and 900 mm for $t = 20$ s and at $x = 400$ mm for $t = 60$ s, respectively. The differences in the vertical erosion speed is reduced for $x = 650$ and 900 mm at $t = 60$ s, tending to similar profiles for all tests at $t = 200$ s. At $x = 650$ mm, the transverse dike surface slope for $d = 3.78$ mm is smaller than for $d = 0.86$ mm. Generally, the breach profiles have a parabola-like shape up to $t = 60$ s from when the trapezoidal-like shape develops.

As a conclusion, larger sediment grain sizes $d$ lead to lower maximum headwater levels $h_M$, smaller maximum discharges $Q_M$, initially faster erosion of the dike volume $V_E$, and smaller transverse breach channel slopes as compared with smaller sediment sizes.
Figure 5.6  Effect of grain size: Spatial dike breach evolution $z(x)$ at $y$ [mm] = (a) 0, (b) 100, (c) 200, (d) 500, and $z(y)$ at $x$ [mm] = (e) 400, (f) 650, (g) 900, (h) 1300 for $t$ [s] = (--) 0, (- -) 20, (-x-) 60, and (-□-) 200. Test (---) 22, (---) 20, (---) 23
5.4 Initial breach width

Five tests (Tests 20, 24 to 27) were conducted for pilot channel base widths of $b_{p,u} = 0, 50, 100, 200, 400$ mm, respectively, for the design breach discharge $Q_{b,d} = 9.2$ l/s. The hydraulic properties are compared in Figure 5.7. The temporal headwater level evolution $h_o(t)$ differs particularly at breach start for the three tests. The maximum headwater level $h_M$ increases as the initial breach width reduces. Note first the almost identical curves $Q_b(t)$ for Tests 20, 24 and 25. For larger breach widths $b_{p,u}$ differences occur both in discharge increase and maximum breach discharge due to the reduced headwater level (Figure 5.7a). Note that $Q_M$ is visibly higher for $b_{p,u} = 400$ mm as compared with $b_{p,u} = 200$ mm, even though inflow and drainage discharge values were similar for all five tests.

The temporal evolution of the eroded sediment volume $V_E(t)$ for different values of $b_{p,u}$ is shown in Figure 5.8. Larger values of $b_{p,u}$ lead to a larger eroded sediment volume at $t \approx 40$ s, while the curves tend to converge at $t = 200$ s for all tests.

![Figure 5.7](image1.png)

*Figure 5.7* Effect of initial breach width: Comparison of hydraulic properties (a) headwater level $h_o(t)$, (b) breach discharge $Q_b(t)$ for $b_{p,u} [\text{mm}] = (---) 0, (- -) 50, (-•-) 100, (-••-) 200, (+-•) 400$ (Tests 20/24/25/26/27)

![Figure 5.8](image2.png)

*Figure 5.8* Effect of initial breach width: Comparison of eroded dike volume $V_E(t)$ for $b_{p,u} [\text{mm}] = (---) 0, (- -) 50, (-•-) 100, (-••-) 200, (+-•) 400$
The topographic profiles are compared in Figure 5.9 (a) to (d) relating to the streamwise $z(x)$, and (e) to (h) to the transverse sections $z(y)$ from the initial dike shape up to $t = 200$ s.

Considering the streamwise sections $y = 0$ to 500 mm, the erosion speed increases with the initial breach width $b_{p,u}$. At $y = 0$ and 100 mm the breach profile shapes are similar for all tests except for Test 27, for which the sediment at the upstream breach channel crest is eroded faster as compared with the remaining tests, and sediment accumulation is noted in its downstream reach. Further, the shapes of the streamwise breach profiles hardly vary from $y = 0$ to 200 mm. The pivot point of spatial dike breach tests is typically located at $y = 0$ mm and $x < 1300$ mm, whereas in this test it moves downstream to $x > 1300$ mm, which is typical for plane dike breaches (Schmocker 2011).

Considering the transverse dike profiles $z(y)$ at $x = 400$ mm, all have similar shape except for Test 27. At $x = 650$ mm, the vertical and horizontal erosion speeds increase with $b_{p,u}$ at $t = 20$ s. At both $t = 60$ and 200 s though, all profiles converge except for Test 27 again. Up to $t \approx 40$ s, the dike erosion progress is mainly vertically from when the horizontal erosion is initiated, depending in addition on $b_{p,u}$. At $x = 900$ mm, the effect of horizontal erosion becomes evident given the gradient of the profiles except for Test 27. Note the relatively steep submerged transverse slope of the breach channel due to the typical hourglass shape which diverts the breach flow toward the glass wall at $x \approx 900$ mm. Only the profiles for the largest $b_{p,u} = 400$ mm have a nearly horizontal breach channel bed, again pointing at typical plane dike breach characteristics. Both the streamwise and the transverse dike breach profiles thus indicate that the breach features of Test 27 with $b_{p,u} = 400$ mm are closer to 2D than 3D breaches if $y < 250$ mm.

As a conclusion, larger initial breach widths $b_{p,u}$ lead to lower maximum headwater levels $h_M$, smaller maximum discharges $Q_M$, and initially faster but later slower erosion based on the curve $V_E(t)$. In contrast, if the initial breach width $b_{p,u}$ is too large, the spatial dike breach characteristics tend to the plane dike breach.
Effect of breach width: Spatial dike breach evolution $z(z) \text{ at } y [\text{mm}] = (a) 0, (b) 100, (c) 200, (d) 500, \text{ and } z(y) \text{ at } x [\text{mm}] = (e) 400, (f) 650, (g) 900, \text{ and } (h) 1300 \text{ for } t [\text{s}] = (-) 0, (- -) 20, (-x-) 60, \text{ and } (-□-) 200. \text{ Test } (\quad -) 20, (\quad -) 24, (\quad -) 25, (\quad -) 26, (\quad -) 27
5.5 Dike shape

Three tests (Tests 28, 20, 29) were conducted for dike shape values $\mu = 0.64, 1, 1.43$, respectively, with $\mu = (3/7)(1/2)(S_o + S_d + L_K/w)$ and $\mu = 1$ for the standard spatial dike breach test with $S_o = S_d = 2$, $L_K = 0.1$ and $w = 0.3$. The initial dike cross-sections are shown e.g. in Figure 5.12c. The design breach discharge was set at $Q_{h,d} = 9.2$ l/s. The hydraulic properties are compared in Figure 5.10. Considering the temporal headwater level evolution $h_o(t)$, $h_o$ increases and $h_M$ is reached later for larger dike shape values. The maximum breach discharge $Q_M$ is similar for $\mu = 1$ and 1.43, while it is slightly higher for $\mu = 0.64$, while the time-to-peak increases with $\mu$. At $t = 170$ s, the breach discharges $Q_b(t)$ tend to the same value of $Q_b = 10$ l/s. Inflow and drainage discharge were similar for all tests (Figure 5.10c,d).

The temporal evolution of the eroded dike volume $V_E(t)$ for different dike shape values $\mu$ is shown in Figure 5.11. For $t \leq 20$ s, the eroded sediment volume is similar for all tests. For $t > 20$ s, the eroded sediment volume increases for larger $\mu$. This effect cannot be solely explained with the evolution $Q_b(t)$, but is rather a result of the increased headwater level $h_o(t)$ and the coupled increased energy head.

![Figure 5.10](image-url)  
**Figure 5.10** Effect of dike shape: Comparison of hydraulic properties for (a) headwater level $h_o(t)$, (b) breach discharge $Q_b(t)$, (c) inflow discharge $Q_o(t)$, and (d) drainage discharge $Q_d(t)$ for $\mu [-] = (---) 0.64, (---) 1, (---) 1.43$. (Tests 28/20/29)
Figure 5.11  Effect of dike shape: Comparison of eroded dike volume $V_E(t)$ for $\mu = (- -) 0.64, (--) 1, (--) 1.43$ (Tests 28/20/29)

The topographic profiles are compared in Figure 5.12 and Figure 5.13 with (a) to (d) and (i) to (l) relating to the streamwise $z(x)$, and (e) to (h) and (m) to (p) to the transverse sections $z(y)$ from the initial dike shape up to $t = 200$ s.

Considering the streamwise sections $y = 0$ to 500 mm, the erosion speed increases with smaller dike shape values $\mu$ in all streamwise and transverse sections at all times after breach initiation. For smaller $\mu$, due to the shorter initial pilot channel length at $y = 0$, the erosion of the downstream portion of the breach channel reaches the upstream end of the pilot channel faster, leading to a faster vertical and horizontal growth of the upstream flow cross-section. These observations have a similar effect on the further streamwise sections $y > 0$ and are in accordance with the finding in the respective hydrographs. The pivot point is situated at $x \approx 0.75L_D$ for all tests and the downstream breach channel slope is similar for $y < 200$ mm, which is due to the faster erosion of the upstream inflow crest for smaller $\mu$.

Considering the transverse dike profiles $z(y)$, vertical and horizontal erosion decreases for larger $\mu$. The width of the profiles close to the inflow crest at $x = 400$ mm for $t \geq 60$ s is similar for all tests, though.

As a conclusion, larger dike shape values $\mu$ lead to larger headwater levels $h_o(t)$, a later reaching of maximum headwater levels $h_M$ and maximum breach discharges $Q_M$, and initially similar but later faster erosion based on the curve $V_E(t)$, which is essentially attributed to $h_o(t)$ and $Q_o(t)$. A pivot was found for the streamwise surface profiles at $y = 0$ at $x \approx 0.75L_D$. 
Figure 5.12  Effect of dike shape: Spatial dike breach evolution \(z(x)\) at \(y \text{ [mm]}\) = (a) 0, (b) 100, (c) 200, (d) 500 and \(z(y)\) in the middle of the upstream slope, the crest and the downstream slope of the respective dike at \(x \text{ [mm]}\) = (e) 300, (f) 450, (g) 600, (h) 900 for Test ( ) 28, \(z(y)\) at \(x \text{ [mm]}\) = (e) 400, (f) 650, (g) 900, (h) 1300 for Test ( ) 20, \(z(y)\) at \(x \text{ [mm]}\) = (e) 400, (f) 800, (g) 1200, (h) 1600 for Test ( ) 29 for \(t \text{ [s]}\) = ( ) 0, ( ) 10, ( ) 40, and ( ) 100
Figure 5.13 Effect of dike shape: Spatial dike breach evolution $z(x)$ at $y$ [mm] = (i) 0, (j) 100, (k) 200, (l) 500 and $z(y)$ in the middle of the upstream slope, the crest and the downstream slope of the respective dike at $x$ [mm] = (m) 300, (n) 450, (o) 600, (p) 900 for Test (—) 28, $z(y)$ at $x$ [mm] = (m) 400, (n) 650, (o) 900, (p) 1300 for Test (---) 20, $z(y)$ at $x$ [mm] = (m) 400, (n) 800, (o) 1200, (p) 1600 for Test (––) 29 for $t$ [s] = (•) 0, (-) 20, (x-) 60, and (o-) 200
5.6 Mobile bed

Three tests (Tests 30 to 32) were conducted with constant inflow discharges $Q_o = 5, 10.2$ and $18$ l/s using a standard dike placed on top of a mobile bed of $0.1$ m height and $2.6$ m length, starting $0.5$ m upstream of the upstream dike toe. The mobile bed was built of the same non-cohesive granular material as the dike itself with a grain size $d = 1.75$ mm. The tests using the same experimental setups except with a fixed channel floor (Tests 19 to 21) are used as reference, and the same inflow discharges $Q_o$ were chosen. The following discussion of the results will focus on the comparison of the tests with fixed and mobile bed, since the hydrographs using different inflow discharge lead to very similar conclusions as for Tests 19 to 21.

The hydraulic properties are compared in Figure 5.14. While $Q_o$ was the same for fixed and mobile beds in the respective tests (Figure 5.14c), the higher position of the dike by $0.1$ m due to the mobile sediment bed led to higher drainage discharge ($\approx 1$ l/s) as compared to the fixed bed (Figure 5.14d) and as a result lower breach discharge $Q_b$ (Figure 5.14b). This difference is negligible for larger discharge values, while for Test 19 with small $Q_o$, the difference in $Q_b$ is up to $20\%$ in the rising limb of the outflow hydrograph compared with the fixed floor test, which also leads to a shift in the temporal evolution. Nevertheless, the fixed and mobile bed hydrographs agree well, except for a shift in the timing $t_M$ of the peak breach discharge $Q_M$, and no influence due to the mobile bed is noticed.

The temporal evolution of the eroded volume $V_E(t)$ for different design breach discharges $Q_{b,d}$ is shown in Figure 5.15. As a general observation, the dike volume is eroded similarly at breach start for $t \leq 60$ s, and is then eroded faster for $t > 60$ s for dikes with mobile bed as compared to tests with a fixed channel bed. Only for the tests with inflow discharge $Q_o = 5$ l/s is the breach faster for $t \geq 40$ s. The faster erosion rate is attributed to the scour in the mobile bed at the downstream dike toe, which leads to a lower downstream filling body and therefore a lower breach surface level with a similar slope as for the respective fixed channel bed test. This is discussed below. Another effect having possibly an influence on the downstream filling body is the length of the downstream mobile bed section, which acts like a platform of certain length. A shorter platform length reduces the downstream filling body height, and leads therefore to faster dike erosion. This effect should be investigated separately.
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Figure 5.14  Effect of mobile bed: Comparison of hydraulic properties for (a) headwater level $h_o(t)$, (b) breach discharge $Q_b(t)$ for $Q_{bd}$ [l/s] = (--- 4, (----) 8, (-----) 16 with (----) mobile bed (Tests 30/31/32) and (---) fixed bed (Tests 19/20/21)

Figure 5.15  Effect of mobile bed: Comparison of eroded dike volume $V_E$ [dm³] for $Q_{bd}$ [l/s] = (--- 4.4, (----) 9.2, (-----) 17.6 and mobile bed to $Q_{bd}$ [l/s] = (--- 4.4, (----) 9.2, (-----) 17.6 and fixed bed
The topographic profiles of mobile and fixed bed tests are compared in Figure 5.16 (Tests 19, 30), Figure 5.17 (Tests 20, 31), and Figure 5.18 (Tests 21, 32), with (a) to (d) relating to the streamwise $z(x)$, and (e) to (h) to the transverse sections $z(y)$ from the initial dike shape up to $t = 200$ s. Since the difference between the profiles using a mobile or fixed bed is similar for all discharges, the results are highlighted only for $Q_o = 10.2$ l/s (Figure 5.17).

Considering the streamwise section $y = 0$ at the glass wall, the sediment surface profiles of both tests with fixed and mobile bed evolve similarly for $t \leq 40$ s, while minor differences are attributed to differences in drainage discharge $Q_{dr}$ and its effect on breach discharge $Q_b(t)$. For $t \leq 40$ s, scour reaching up to $z = -22$ mm below the initial sediment bed level can be observed at the downstream dike toe, leading to a pivot point for $t \geq 60$ s at $x \approx 0.85L_D$ and $z \approx -17$ mm. During the scour development, the sediment surface profiles of the mobile bed test erode faster in a near parallel movement in the order of magnitude of the scour depth. The trend of faster parallel erosion and its timing is also observed for $y > 0$.

Considering the transverse dike profiles $z(y)$, the horizontal breach profile width development is similar for fixed and mobile beds at all times and profiles. Vertical erosion is faster though for $t \geq 60$ s for the mobile bed test as compared to fixed bed test, leading to a lower breach bed in the order of magnitude of the scour depth.

As a conclusion, when considering a test using mobile bed instead of a fixed bed, the hydrographs are similar when drainage discharge $Q_{dr}$ is properly compensated and breach discharge $Q_b(t)$ is comparable. The initial erosion pattern is similar for both, while starting at $t \geq 40$ s scour occurs at the downstream dike toe of the mobile bed test, leading to increased sediment transport and vertical erosion in the order of magnitude of the scour depth. A possible influence on the breach development not covered in this test series is the necessary length of the mobile bed or a platform to exclude effects of the downstream filling body due to different sediment deposition downstream.
Figure 5.16  Effect of mobile bed: Spatial dike breach evolution $z(x)$ at $y$ [mm] = (a) 0, (b) 100, (c) 200, (d) 500, and $z(y)$ at $x$ [mm] = (e) 400, (f) 650, (g) 900, (h) 1300 for $t$ [s] = (→) 0, (→-→) 10, (→-→) 20, (→-→) 40, (→-→) 60, (→-→) 100, and (→-→) 200. Test (→) 30 with mobile bed, Test (→) 19 with fixed bed with $Q_{,d} = 4.4$ l/s
Figure 5.17  Effect of mobile bed: Spatial dike breach evolution $z(x)$ at $y [\text{mm}] = (a) 0$, (b) 100, (c) 200, (d) 500, and $z(y)$ at $x [\text{mm}] = (e) 400$, (f) 650, (g) 900, (h) 1300 for $t [\text{s}] = (-) 0$, (--) 10, (--) 20, (---) 40, (x-x) 60, (o-o) 100 and (□-□) 200. Test (---) 31 with mobile bed, Test (----) 20 with fixed bed with $Q_{b,p} = 9.2 \text{l/s}$
Figure 5.18  Effect of mobile bed: Spatial dike breach evolution $z(x)$ at $y$ [mm] = (a) 0, (b) 100, (c) 200, (d) 500, and $z(y)$ at $x$ [mm] = (e) 400, (f) 650, (g) 900, (h) 1300 for $t$ [s] = (−) 0, (−−−−) 10, (−−) 20, (−−−−−) 40, (−−−−−−) 60, (−−−−−−−) 100 and (−−−−−−−−) 200. Test (−−−−−) 32 with mobile bed, Test (−−−−−−) 21 with fixed bed with $Q_{b,d} = 17.6$ l/s
5.7 Water surface

In all previous dike breach tests, the sediment surface coordinates were determined using a photogrammetric system and refraction effects were eliminated in the submerged breach areas assuming a 2D water surface profile, as determined along the glass wall. To determine the 3D water surface characteristics, the water was colored white using 4 g TiO$_2$/l water and the photogrammetric system was applied to measure the water surface coordinates, thereby following the procedure of Evers and Hager (2015, 2016) in impulse wave research based on the photogrammetric AICON system. Due to the small quantity of TiO$_2$ required, no surface film effects were found by Przadka et al. (2012), so that the breach process is assumed to be comparable with tests using non-colored water.

Tests 33 to 35 with $Q_{b,d} = 4.4, 9.2$ and 17.6 l/s, respectively, were conducted using colored water, to replicate the identical hydraulic conditions as in Tests 19 to 21. The effect of apparent cohesion at breach start was assumed negligible and the dike of Tests 34 and 35 were installed using wetted sediment. The water surface data of Tests 33 to 35 were then combined with the sediment surface data of Tests 19 to 21, assuming test repeatability as presented in 4.3. Figure 5.19 shows the combination of the water surface of Test 35 with the sediment surface of Test 21.

![Figure 5.19](image)

Figure 5.19 Triangulated photogrammetric results of water surface height $h$ measured from channel floor using colored water (Test 35) combined with sediment surface $z_b$ in grey (Test 21) for $Q_{b,d} = 17.6$ l/s
The transverse water surface profiles $h(y)$ are presented in Figure 5.20 to Figure 5.22 for Tests 33 to 35 and combined with the transverse sediment surface profiles $z(y)$ for Tests 19 to 21 for $Q_{b,d} = 4.4, 9.2$ and $17.6$ l/s, respectively. For better visibility, the profiles are grouped in (a) to (d) for the times $t \, [s] = 0, 10, 40$ and $100$ and (e) to (h) for $t \, [s] = 0, 20, 60$ and $200$. For illustrative purposes, the water surface profiles were extended horizontally towards the glass wall up to $30$ mm in case data were missing.

In Tests 19 and 33 (Figure 5.20), the water level has no noticeable inclination during the initial breach development for all profiles at $t \leq 20$ s. When the inflow section of the pilot channel at $x = 400$ mm starts being eroding at $t > 20$ s and breach discharge increases (Figure 5.1), the water surface profile is lower at the glass wall compared to $y > 300$ mm and becomes inclined and concave, as does also the respective sediment surface profile below it due to erosion. For $t > 100$ s, the transverse water surface inclination is reduced and surface waves become visible. In the dike crest center at $x = 650$ mm, the water surface is almost horizontal, except for $t = 40$ s, where the water level is highest at the glass wall and has a convex shape. At $x = 900$ mm, the water surface profiles at all times are highest at the glass wall and convex. These features are due to streamline curvature in the plan, pointing at the effects of centrifugal accelerations of the flow. At $t = 60$ and $100$ s, the water surface profile is markedly inclined and has a strong curvature, leading to problems with the photogrammetric system; missing data points and questionable shapes at the breach channel bed occurred, as is noted from the respective sediment bed profiles.

Note that the water surface touches the sediment bed approximatively at the sharp bend of the sediment surface profile for $t > 20$ s. While the submerged breach portions are unnatural at $t = 60$ and $100$ s as described above, the curvature in the sediment surface profile between submerged and non-submerged portions appears natural at $t = 200$ s, and is in agreement with the sediment surface profiles and curvatures for $x \leq 900$ mm. As previously noted, photogrammetric sediment surface measurements proved difficult at $x = 1300$ mm, due to the presence of surface and capillary waves, as well as large water surface inclinations and bends. The water surface measurement proved to be simpler and more reliable. The visible sediment deposition for $t > 20$ s represents the own sediment bed created during the breach, and was observed to be continuously but only slightly overflown, increasing in height depending on the instantaneous discharge, sediment load and sedimentation. The maximum height of the sediment deposition and its correct slope can therefore be qualitatively compared to the water surface profile in this section, indicating good agreement despite some refraction effects for the sediment bed measurement.
These results allow for visualizing the differences of the assumed 2D and the effective 3D water surfaces and the resulting limitations of the photogrammetric system when determining sediment topographies. Refraction effects remained at $x = 400$ mm and $t = 60$ s, where the water level at the glass wall is visibly lower compared to $y > 400$ mm, which leads to a too high sediment bed $y > 400$ mm, and the opposite for $x = 900$ mm and $t = 60$ s, where the water level is high at the glass wall and bends down up to $S_y = 0.5$, inhibiting the photogrammetric system from capturing surface data altogether.

Considering Tests 20 and 34 in Figure 5.21 for $Q_{b,d} = 9.2$ l/s, as previously observed, the water level profiles at $x = 400$ mm have a concave shape when sediment erosion due to higher flow velocities at the glass wall leads to the typical concave breach profile shape at $t > 20$ s. At $x = 650$ mm, the water surface profiles are mostly horizontal, only at breach start at $t = 20$ s the water surface profile is highest at the glass wall with a convex curvature. At $x = 900$ mm for $t > 10$ s, all water surface profiles are highest at the glass wall and tend to drop in a convex shape until they reach the sediment surface profile.

Considering Tests 21 and 35 in Figure 5.22 for $Q_{b,d} = 17.6$ l/s, as for the previous two tests, the water surface profile at $x = 400$ mm is lowest at the glass wall and has a concave shape when the sediment surface profile is eroded for $t > 10$ s. As in the previous tests, the water surface profiles tend to be horizontal at $x = 650$ mm and to be highest at the glass wall, having convex shape at $x = 900$ mm.

Summarizing these three tests relating to the water surface, the results indicate that:

- Tests were repeatable and profiles of respective two tests match well, despite Test 34 and 35 involved initially wetted sediment;
- Angle of repose in the submerged flow cross-section is lower as compared to that non-submerged, the limit being indicated by a kink in the sediment profile;
- Water surface profile tends to be lowest at the glass wall with a concave shape in the upstream reach $x = 400$ mm, then becomes horizontal when passing the dike crest center at $x = 650$ mm and is then highest at the glass wall with a convex shape at the downstream end of the channel at $x = 900$ mm, where the water flow is diverted towards the glass wall due to the hourglass shape of the breach channel;
- Profiles at the downstream dike toe at $x = 1300$ mm show plausible sediment and water surface profiles, the sediment profiles tending to be slightly too low due to the photogrammetric limitations and refraction effects.
Figure 5.20  Evolution of transverse (—) water surface profiles $h(y)$ (Test 33) and (—) sediment surface profiles $z(y)$ (Test 19) at $x$ [mm] = (a,e) 400, (b,f) 650, (c,g) 900, (d,h) 1300 for $t$ [s] = (—) 0, (—) 10, (—) 20, (—) 40, (—) 60, (—) 100 and (—) 200, for $Q_{b,d} = 4.4$ l/s
Figure 5.21 Evolution of transverse (---) water surface profiles \( h(y) \) (Test 34) and (---) sediment surface profiles \( z(y) \) (Test 20) at \( x \) [mm] = (a,e) 400, (b,f) 650, (c,g) 900, (d,h) 1300 for \( t \) [s] = (−) 0, (− −) 10, (− − −) 20, (− − − −) 40, (− − − − −) 60, (− − − − − −) 100 and (− − − − − − −) 200, for \( Q_{b,d} = 9.2 \) l/s
Figure 5.22 Evolution of transverse (—) water surface profiles $h(y)$ (Test 35) and (—) sediment surface profiles $z(y)$ (Test 21) at $x$ [mm] = (a,e) 400, (b,f) 650, (c,g) 900, (d,h) 1300 for $t$ [s] = (−) 0, (−−) 10, (−−−) 20, (−−−−) 40, (−x−−) 60, (−○−−) 100 and (−□−−) 200, for $Q_{b,d} = 17.6$ l/s
Figure 5.23 to Figure 5.25 show plan views of the temporal evolution of the transverse water surface inclination $S_y = \Delta h_y/\Delta y$ for Test 33, 34 and 35 with $Q_{b,d} = 4.4, 9.2$ and 17.6 l/s, respectively. The white areas without water surface data represent the non-submerged dike surface, while those along the figure border are due to missing data. Areas where the transverse water surface profile is nearly horizontal are shown in grey, blue highlights rising water surface portions ($S_y > 0.01$), while red relates to falling water surface portions ($S_y < -0.01$) along the $y$-axis. The determined surface coordinates were triangulated and then interpolated on a 10 mm grid for visualization.

For Test 33 (Figure 5.23), the headwater surface inclination remains always horizontal ($S_y = 0$) in the transverse direction. At $t = 10$ s, no information is available in the breach channel since it is too narrow. The breach channel is still developing at $t = 20$ s with a water-sediment mixture transported along the glass wall. For $t > 20$ s, the water surface inclination is available in the entire breach channel. At $t = 40$ s, the areas with previously observed flow separation at the breach channel inflow are colored green up to $S_y > 0.01$, its distribution indicating the formation of the inflow crest. Due to the shape of the inflow section, the water flow is deflected toward the glass wall, passes through $S_y \approx 0$ at $x \approx 600$ and reverses its inclination to $S_y < -0.3$ for $x > 600$ mm mainly near to the non-submerged sediment and in the region where the breach width is smallest. Although the hourglass shape of the breach channel has considerably changed at $t = 60$ and 100 s, the patterns are similar to those observed at $t = 40$ s. The flow separation zone with $S_y > 0.01$ remains at the breach channel inflow along the upstream end of the hourglass shape, the position of $S_y \approx 0$ remains at $x \approx 600$ mm, while the maximum inclination $S_y < -0.3$ moves downstream and remains close to the smallest breach width at $x \approx 1000$ mm. The maximum inclinations $S_y < -0.03$ occur at $t = 40$ and 60 s, which coincides with maximum headwater levels and breach discharges, as well as the development of the breach channel and the hourglass shape. For $t \geq 100$ s, similar patterns are observed with reduced inclinations, due to the lower headwater level and the hydrodynamically efficient shape of the breach channel. The striped color patterns for $t \geq 100$ s and 300 mm $< x < 800$ mm represent standing surface waves, moving from the point of flow separation across the breach channel toward the glass wall, as previously described in the photogrammetric system limitations.

For Test 34 with $Q_{b,d} = 9.2$ l/s (Figure 5.24) and Test 35 with $Q_{b,d} = 17.6$ l/s (Figure 5.25), the observations are similar as for Test 33. Due to the higher discharge, the inclinations $|S_y|$ are larger (down to $S_y < -0.5$) and the observed patterns move in the streamwise direction.
Figure 5.23  Transverse water surface gradient $S_y = \Delta h_y/\Delta y$ [-] with positive values for rising water level in $y$ direction for Test 33 with design breach discharge $Q_{b,d} = 4.4$ l/s at $t [s] = \{a\} \text{ 10, } (b) \text{ 20, } (c) \text{ 40, } (d) \text{ 60, } (e) \text{ 100 and } (f) \text{ 200}
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Figure 5.24  Transverse water surface gradient \( S_y = \frac{\Delta h_y}{\Delta y} \) with positive values for rising water level in \( y \) direction for Test 34 with design breach discharge \( Q_{h,d} = 9.2 \text{ l/s} \) at \( t \text{ [s]} = (a) 10, (b) 20, (c) 40, (d) 60, (e) 100 \) and (f) 200
Figure 5.25  Transverse water surface gradient $S_y = \Delta h_y/\Delta y$ [-] with positive values for rising water level in $y$ direction for Test 35 with design breach discharge $Q_{b,d} = 17.6$ l/s at $t [s] = (a) 10$, (b) 20, (c) 40, (d) 60, (e) 100 and (f) 200
5.8 Reservoir volume

As described in Chapter 2, one of the main parameters influencing a dike breach is the reservoir water volume $V_R$. Since testing of a wide spectrum of different $V_R$ is constricted by the channel dimensions, the test procedure was modified by adding a simulated reservoir water volume to the physical channel water volume using a controller, as described in Chapter 3. For test comparison, the reservoir water surface area $A_R$ at breach start $t = 0$ is used rather than $V_R$, since it better describes the driving forces on the breach process, as discussed in Chapter 2 and 6. As in the previous chapters, all $A_R$ and breach discharges $Q_b$ are described for the half-model tests and have to be doubled when compared to full-model tests. A total of ten half-model tests with different $A_R$ were conducted, of which nine tests involved the rectangular reservoir shape (reservoir shape factor $c = 1$ by Kühne 1978) and $A_R$ [m$^2$] = 3.44, 13.44, 16.56, 33.44, 63.44, 103.44, $\infty$ (Tests 40, 36/37, 41, 42, 43, 44, 38/39); one test with triangular reservoir shape ($m = 2$) and $A_R = 13.44$ m$^2$ was also conducted (Test 45). For $c = 1$, $A_R = x_D b$ is nearly independent from the headwater level $h_o$, with $x_D$ = distance of the upstream dike toe from the upstream channel wall and $b$ = channel width. For $c = 2$, $A_R$ varies with $h_o$ as $A_R(h_o) = (h_o/h_o,0)x_D b$, with $h_o,0$ as the headwater level at breach start. The pilot channel for these half-model tests was again positioned at the glass wall with a height and width of $w_p = 0.02$ m and $b_p = 0.04$ m. For breach initiation, the headwater level $h_o$ was raised with constant velocity $\Delta h_o/\Delta t = 1$ mm/s up to $h_o = 0.3$ m, from which the effect of $A_R$ was simulated.

The resulting hydrographs for this test series are shown in Figure 5.26. Figure 5.26a indicates that the headwater level at breach start is $h_o,0 = 0.3$ m for all tests, and drops faster for small reservoir water surface areas $A_R$ as compared to large $A_R$ during the breach. At the end of the breach process – when water stops flowing through the breach channel – $h_o$ tends toward the maximum dike height at the breach center $z_M$ ($h_o-z_M \approx 0$). The headwater level then tends to fluctuate for the small breach discharges $Q_b<1$ l/s due to the much larger pump capacity. For Tests 36 and 37 with $A_R = 13.44$ m$^2$, repeatability of the headwater level evolution $h_o(t)$ is satisfactory. Tests 38 and 39 with a constant headwater level ($A_R = \infty$) also show a good repeatability, although the headwater level drops slightly for both tests after breach initiation, since the controller is not allowed to regulate aggressively, as described in Chapter 3. The breach widening velocity reaches some 0.5 m/min (half-model test). At $t = 80$ s, the width of the inflow section reaches the channel wall, influencing the further breach evolution, while at the same time the headwater level drops when the maximum pumping capacity is reached at around $Q_o = 96$ l/s (Figure 5.26b).
Figure 5.26 Effect of reservoir water surface area $A_R$: Comparison of hydraulic properties (a) headwater level $h_o(t)$, (b) breach discharge $Q_b(t)$ for $A_R$ [m²] = (---) 3.44, (---)(---) 13.44, (---) 16.56, (---) 33.44, (---) 63.44, (---) 103.44, (---)(---) $\infty$ (Tests 40, 36/37, 41, 42, 43, 44, 38/39). (---) triangular shaped reservoir with $A_R(h_o=300$ mm) = 13.44 m² (Test 45)

For $A_R>60$ m², Figure 5.27 shows that the breach channel reaches the channel wall at the inflow section, possibly influencing the breach process. While this effect is negligible on Test 43 with $A_R = 63.44$ m² (Figure 5.27e), in Test 44 with $A_R = 103.44$ m² in Figure 5.27f the dike is completely submerged for $t > 135$ s. This appears to have a slight effect on the maximum breach discharge in Figure 5.26b, which at the same time reaches the maximum pump capacity. Therefore, Test 44 with $A_R = 103.44$ m² can be considered for future evaluations only with certain reservations, while Test 43 with $A_R = 63.44$ m² is considered fully valid and represents the test limit using this setup. For the tests with $A_R \leq 63.44$ m², the evolution of the breach discharge $Q_b(t)$ is comparable in shape, with larger peak discharge $Q_M$ – which occur at a later time $t_M$ – for large $A_R$ as compared to small $A_R$.

When comparing Test 45 involving the triangular reservoir shape to these with rectangular shape (Tests 36 and 37) with the same initial value $A_R = 13.44$ m² in Figure 5.26, the headwater level drops similarly up to $t = 50$ s but then faster for the triangular shaped reservoir up to $t = 250$ s, when $Q_b \approx 0$. The peak breach discharge $Q_M$ is reached at the same time $t \approx 50$ s with the same value $Q_M \approx 24.3$ l/s. For $t > 50$ s though, $Q_b$ drops visibly faster for the triangular reservoir shape as compared to the rectangular reservoir shape. This is attributed to reduced $A_R$ as $h_o$ drops in the triangular reservoir, leading to less water available over time during the breach process and thus less erosion. The question then arises, why $Q_M$ of the triangular test is similar to the rectangular tests. $Q_M$ is reached for all tests at $t \approx 50$ s, when the water level has dropped by 50 mm to $h_o = 250$ mm and $A_R$ in the triangular reservoir is reduced by $1/6$th to 11.2 m². Assuming $A_R$ at the time of $Q_M$ according to Eq. (6.17) developed in Chapter 6, a reduction of $A_R$ by $1/6$th reduces $Q_M$ by $1/10$th. Comparing the cumulated volumetric breach discharges of the simulated triangular
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Figure 5.27 Effect of reservoir water surface area $A_R$ on final breach shape: Top view from upstream (camera 1) with flow direction from top to bottom, glass wall on right side and $A_R$ [m²] = (a) 3.44, (b) 13.44, (c) 16.56, (d) 33.44, (e) 63.44. Test 44 is shown in (f) for $A_R = 103.44$ m² at $t = 135$ s

and rectangular reservoir shapes in Figure 5.28 indicate a good agreement between the simulated and measured $Q_b$ and the resulting cumulated breach discharge $V_W$ in general, although small differences exist for the triangular breach simulation. Possibly, not only the instantaneous value of $A_R$ at $t = 50$ s is effective for $Q_M$, but the entire development of the breach up to $Q_M$ plays a role. This would then lead to a difference of $A_R$ smaller than 1/6th and therefore a differences in $Q_M$ smaller than 1/10th. The effect of the reservoir shape on $Q_M$ is therefore an interesting topic for future research.

Figure 5.28 Comparison of cumulated breach discharge volume $V_W$ for (a) triangular and (b) rectangular reservoir shape of $A_R = 13.44$ m² determined with (-) water balance method with input $Q_o, Q_\delta, Q_\theta$ in physical channel and (—) $h_o$ and simulated $A_R$
5.9 Summary

Hydraulic model tests were conducted to determine the effects of the parameters breach discharge, sediment grain size, initial breach width, dike cross-section, mobile bed, and reservoir water surface area on the spatial dike breach process.

Large inflow discharge leads to a high maximum headwater level, high peak breach discharge, high sediment erosion rates and a faster breach process in terms of time-to-peak.

Large sediment grain sizes lead to an initially high sediment erosion rate and an initially fast breach process, although these trends are not pronounced. The submerged breach channel shape is similar for all grain sizes, while small grain sizes lead to steep non-submerged side slopes. This indicates that for larger hydraulic loads (e.g. large reservoir water surface areas) the effect of grain size on the breach process is further reduced.

Large initial breach widths lead to low maximum headwater levels and low peak breach discharge, but the erosion rates and the breach process are comparable. If the initial breach width is too large though, the spatial dike breach features are similar to those of plane dike breaches with an accelerated breach process and a large initial sediment erosion rate.

Large dike cross-sections lead to high headwater levels, small peak breach discharge and a slow breach process in terms of time-to-peak. An increase in the erosion rate is visible only for larger times due to the high headwater level.

A mobile bed below the dike leads to similar breach hydrographs and erosion rates as compared with tests on a fixed channel bed. Only after the breach channel has developed, scour occurs at the downstream dike toe, which leads to increased sediment transport and increased vertical erosion in the order of magnitude of the scour depth.

Large reservoir water surface areas lead to large peak breach discharge and a slow breach process in terms of time-to-peak. Reservoirs with different water surface areas lead to similarly shaped breach discharge hydrographs. For constant reservoir level, no peak discharge occurs and the breach widening velocity is around 0.5 m/min (half-model test) or 1 m/min (full-model test) for dikes 0.3 m high and with a sediment grain size of 1.75 mm.

The effect of the reservoir shape was determined for a triangular and a rectangular reservoir shape, i.e. a reservoir shape factor $c = 2$ and $1$, respectively, according to Kühne (1978) and Table 2.2, with the same initial horizontal reservoir water surface area $A_R$. While the peak breach discharges are nearly the same, deviations in the flow hydrograph increase with time. The good agreement of the peak breach discharge indicates that the
initial reservoir water surface area describes the breach process better than the reservoir volume, as is also demonstrated in the normalized results described below.

In an additional test series, the photogrammetric system was applied to measure the white-colored water surface topography. The water surface data were then combined with the sediment surface data of previous tests, assuming test repeatability. The results indicate good repeatability, since the two data sets match well. The angle of repose in the submerged flow cross-section is lower as compared to that of non-submerged breach portions, the limit being indicated by a kink in the sediment profile. The inclination and curvature of the transverse water surface profile varies along the breach channel and gives an indication on the influence on the photogrammetric measurement results when capturing the breach topography.

In summary, the parameters breach discharge, dike cross-section and reservoir water surface area have a strong effect on the spatial dike breach process. The effect of the mobile bed only appears after the downstream dike toe is scoured, while only small effects were visible for the parameters sediment grain size and initial breach width. The findings on the effect of specific parameters on the breach process agree well with literature findings. In an additional test series with white-colored water, the features of the water surface topography during the spatial breach process are described and coupled with the sediment surface topography. These measurements highlight the different angles of repose in submerged and non-submerged breach profiles and the effect of the water surface inclination and curvature on the photogrammetric measurement of the submerged breach channel.
6 Normalized results

6.1 Introduction

The dimensional results from the physical model found in Chapters 4 and 5 are presented hereafter in non-dimensional form. For a systematic analysis of the spatial dike breach, the following data were extracted from half-model tests as presented in Chapters 4 and 5 in dimensional form:

- Hydrographs ($h_o, Q_o, Q, Q_{dr}$)
- Sediment surface topography ($z(x), z(y)$)
- Eroded sediment volume ($V_E$)
- Breach shape (breach profile, hourglass shape, inflow crest)
- Hydraulic properties ($A, v, H, F$)

In this chapter, the previous dimensional data are used to describe the breach process in non-dimensional form for full-model tests to facilitate comparison and field application. Extending the dimensional analysis by Schmocker (2011), dimensionless results are presented for tests with constant inflow discharge $Q_o$ as a hydraulic boundary condition:

- Dimensionless time
- Dimensionless peak breach discharge
- Dimensionless maximum head water level
- Dimensionless evolution of maximum dike height at breach center
- Dimensionless evolution of eroded sediment volume
- Normalized hourglass shapes and transverse submerged breach profiles

Considering tests with the hydraulic boundary condition $Q_o = 0$ and a variation of reservoir water surface areas $A_R$, no constant inflow discharge $Q_o$ and therefore no critical flow depth $h_c, \nabla$ is available, assuming for $h_c, \nabla$ a triangular pilot channel of transverse slope $S_o = 2$. Hence, data acquired using $h_c, \nabla$ are related to $h_M$ in the governing parameters, leading to the generalized non-dimensional relation for the

- Dimensionless peak breach discharge as a function of $h_M, A_R, A_D$ and $Q_{b,d}$.

These results are then compared to results of other research institutions. The application of the results is highlighted for two test cases involving two different hydraulic boundary conditions, assuming scalability according to Froude as described in Chapters 2 and 4.
6.2 Tests and corresponding symbols for normalized results

The normalized results are subdivided into tests using as a hydraulic boundary condition (1) constant inflow discharge $Q_0$ (6.4 and Table 6.1) and (2) tests with varied reservoir water surface areas $A_R$ and $Q_0 = 0$ (6.5 and Table 6.2). The values from the half-model tests were doubled to facilitate comparison and field application.

Table 6.1 Constant inflow discharge tests and corresponding symbols for normalized results

<table>
<thead>
<tr>
<th>Test</th>
<th>Symbol</th>
<th>$w$ [mm]</th>
<th>$\mu$ [-]</th>
<th>$d$ [mm]</th>
<th>$Q_{b,d}$* [l/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,2,3,4</td>
<td>*</td>
<td>200</td>
<td>1.07</td>
<td>1.25</td>
<td>26</td>
</tr>
<tr>
<td>5,6,7</td>
<td>□</td>
<td>600</td>
<td>1</td>
<td>3.78</td>
<td>83.6</td>
</tr>
<tr>
<td>10</td>
<td>◇</td>
<td>300</td>
<td>1</td>
<td>1.75</td>
<td>14.8</td>
</tr>
<tr>
<td>11,12,15</td>
<td>▲,▲,▲</td>
<td>150</td>
<td>1</td>
<td>0.86, 0.86, 0.43</td>
<td>2.62, 1.3, 2.62</td>
</tr>
<tr>
<td>9, 13, 16</td>
<td>●,●,●</td>
<td>300</td>
<td>1.75, 1.75, 0.86</td>
<td>14.8, 7.4, 14.8</td>
<td></td>
</tr>
<tr>
<td>8, 14, 17</td>
<td>■,■,■</td>
<td>600</td>
<td>3.78, 3.78, 1.75</td>
<td>83.6, 41.8, 83.6</td>
<td></td>
</tr>
<tr>
<td>18, 19, 20, 21</td>
<td>◆,◆,◆,◆</td>
<td>300</td>
<td>1</td>
<td>1.75</td>
<td>4.4, 8.8, 18.4, 35.2</td>
</tr>
<tr>
<td>22, 23</td>
<td>▲,▲</td>
<td>300</td>
<td>1</td>
<td>0.86, 3.78</td>
<td>18.4</td>
</tr>
<tr>
<td>24, 25, 26, (27)</td>
<td>◆,◆,◆,◆</td>
<td>300</td>
<td>1</td>
<td>1.75</td>
<td>18.4**</td>
</tr>
<tr>
<td>28, 29</td>
<td>▲,▲</td>
<td>300</td>
<td>0.64, 1.43</td>
<td>1.75</td>
<td>18.4</td>
</tr>
<tr>
<td>30, 31, 32</td>
<td><em>,</em>,*</td>
<td>300</td>
<td>1</td>
<td>1.75</td>
<td>8.8, 18.4, 35.2</td>
</tr>
</tbody>
</table>

* Discharge from half-model tests doubled for full-model representation
** Pilot channel top notch width $b_p = 500, 600, 800, 1200$ mm
( ) Test not considered for normalized equations

Table 6.2 Reservoir water surface area tests and related symbols for normalized results

<table>
<thead>
<tr>
<th>Test</th>
<th>Symbol</th>
<th>$w=h_M$ [mm]</th>
<th>$\mu$ [-]</th>
<th>$d$ [mm]</th>
<th>$A_R$* [m$^2$]</th>
<th>Reservoir shape</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>●</td>
<td>300</td>
<td>1</td>
<td>1.75</td>
<td>6.88</td>
<td>□</td>
</tr>
<tr>
<td>36,37</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>26.88</td>
<td></td>
</tr>
<tr>
<td>41</td>
<td>●</td>
<td></td>
<td></td>
<td></td>
<td>33.12</td>
<td></td>
</tr>
<tr>
<td>42</td>
<td>□</td>
<td></td>
<td></td>
<td></td>
<td>66.88</td>
<td></td>
</tr>
<tr>
<td>43</td>
<td>□</td>
<td></td>
<td></td>
<td></td>
<td>126.88</td>
<td></td>
</tr>
<tr>
<td>(44)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>206.88</td>
<td></td>
</tr>
<tr>
<td>45</td>
<td>▲</td>
<td>300</td>
<td>1</td>
<td>1.75</td>
<td>26.88</td>
<td>▽</td>
</tr>
</tbody>
</table>

* Reservoir water surface area $A_R$ from half-model tests doubled for full-model representation
( ) Test not considered for normalized equations
6.3 Test range

The symbols in Table 6.1 were used to present the dimensionless results in the diagrams. The test ranges for full-model tests are presented in Table 6.3.

**Table 6.3** Test ranges referring to full-model tests

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dike height</td>
<td>0.15 m ≤ w ≤ 0.6 m</td>
</tr>
<tr>
<td>Dike shape</td>
<td>0.64 ≤ µ ≤ 1.43</td>
</tr>
<tr>
<td>Design breach discharge</td>
<td>1.3 l/s ≤ Q_{bd} ≤ 83.6 l/s</td>
</tr>
<tr>
<td>Reservoir water surface area</td>
<td>0.3 m² ≤ A_R* ≤ 126.88 m²</td>
</tr>
<tr>
<td>Sediment grain size</td>
<td>0.43 mm ≤ d ≤ 3.78 mm</td>
</tr>
<tr>
<td>Pilot channel notch width at top</td>
<td>0.04 m ≤ b_p** ≤ 1.2 m</td>
</tr>
</tbody>
</table>

* A_R = 206.88 m²: Breach shape and thus breach properties influenced by channel width
** b_p = 1.2 m: Breach properties closer to 2D than 3D dike breach processes

No major scale effects were found for the entire range during the constant inflow discharge tests in Table 3.2 and Table 6.1, as discussed in Chapter 4. Test 27 with b_p = 1.2 m was not considered for the fitting equations, since the breach process differed considerably from the remaining tests with breach characteristics similar to plane dike breaches. Test 44 with A_R = 206.88 m² was also not included in the fitting equations since the breach shape (and thus the breach properties) were influenced by the channel width (5.8). Due to the development of the reservoir simulation method at the end of the test program, the test range for large reservoir volumes A_R > 6.88 m² was limited to w = 0.3 m, µ = 1, d = 1.75 mm and mainly rectangular reservoir volumes, as shown in Table 6.2. The results of Wallner (2014) and Walder et al. (2015) validate the present tests and extend the range for large A_R to sediment grain sizes 0.21 mm ≤ d ≤ 1.75 mm in 6.6.

6.4 Constant inflow discharge tests

6.4.1 Dimensional analysis

For the spatial dike breach tests with constant inflow discharge Q_o, the main breach parameters to be considered according to Schmocker (2011), Schmocker et al. (2014) and Müller et al. (2016) for 2D dike breaches are extended for 3D dike breaches in Table 6.4. The main breach parameters can be categorized into properties describing the general
dike dimensions, the channel and reservoir dimensions, the sediment properties, the hydraulic boundary conditions, the fluid properties and the acceleration of gravity. The parameters were introduced in Chapters 2, 4 and 5 and are described in Table 6.4.

Table 6.4  Main breach parameters considered for spatial dike breaches

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dike height</td>
<td>$w$</td>
<td>[m]</td>
</tr>
<tr>
<td>Median sediment diameter</td>
<td>$d_{50}$</td>
<td>[m]</td>
</tr>
<tr>
<td>Dike crest length</td>
<td>$L_K$</td>
<td>[m]</td>
</tr>
<tr>
<td>Sediment density</td>
<td>$\rho_s$</td>
<td>[kg/m$^3$]</td>
</tr>
<tr>
<td>Dike slope up- and downstream</td>
<td>$S_o, S_d$[-]</td>
<td></td>
</tr>
<tr>
<td>Grain size distribution</td>
<td>$\sigma_g$[-]</td>
<td></td>
</tr>
<tr>
<td>Dike length</td>
<td>$L_D$</td>
<td>[m]</td>
</tr>
<tr>
<td>Angle of repose</td>
<td>$\theta$[-]</td>
<td></td>
</tr>
<tr>
<td>Dike shape parameter</td>
<td>$\mu$[-]</td>
<td></td>
</tr>
<tr>
<td>Submerged angle of repose</td>
<td>$\theta_s$[-]</td>
<td></td>
</tr>
<tr>
<td>Dike cross-section</td>
<td>$A_D$</td>
<td>[m$^2$]</td>
</tr>
<tr>
<td>Particle parameter</td>
<td>$D*$[-]</td>
<td></td>
</tr>
<tr>
<td>Pilot channel notch height</td>
<td>$w_p$</td>
<td>[m]</td>
</tr>
<tr>
<td>Inflow discharge</td>
<td>$Q_o$</td>
<td>[m$^3$/s]</td>
</tr>
<tr>
<td>Pilot channel notch width at top</td>
<td>$b_p$</td>
<td>[m]</td>
</tr>
<tr>
<td>Drainage discharge</td>
<td>$Q_{dr}$</td>
<td>[m$^3$/s]</td>
</tr>
<tr>
<td>Eroded mobile bed height</td>
<td>$w_{mobile}$</td>
<td></td>
</tr>
<tr>
<td>Water density</td>
<td>$\rho$</td>
<td>[kg/m$^3$]</td>
</tr>
<tr>
<td>Toe distance from inlet wall</td>
<td>$x_D$</td>
<td>[m]</td>
</tr>
<tr>
<td>Surface tension of fluid</td>
<td>$\sigma$</td>
<td>[kg/s$^2$]</td>
</tr>
<tr>
<td>Channel width</td>
<td>$b$</td>
<td>[m]</td>
</tr>
<tr>
<td>Porosity</td>
<td>$\varepsilon$[-]</td>
<td></td>
</tr>
<tr>
<td>Downstream platform length</td>
<td>$L_f$</td>
<td>[m]</td>
</tr>
<tr>
<td>Kinematic viscosity of fluid</td>
<td>$\nu$</td>
<td>[m$^2$/s]</td>
</tr>
<tr>
<td>Reservoir water surface area</td>
<td>$A_R$</td>
<td>[m$^2$]</td>
</tr>
<tr>
<td>Acceleration of gravity</td>
<td>$g$</td>
<td>[m/s$^2$]</td>
</tr>
</tbody>
</table>

The shape parameter $\mu = (3/7)(1/2)(S_o+S_d+L_K/w)$ was introduced in Chapter 5 and describes the dike cross-sectional area $A_D = (7/3)\mu w^2$, combining the individual parameters $S_o, S_d, L_K$ and $L_D$. The upstream reservoir water volume $V_R$ is described by the reservoir water surface area $A_R$, which combines $x_D$ and $b$, and the headwater level $h_o$. The design breach discharge $Q_{b,d}$ takes into account the loss of breach discharge due to seepage as $Q_{b,d} = Q_o - Q_{dr}$, which in turn is represented by the critical flow depth $h_{c,V}$, assuming a triangular pilot channel of transverse slope $S_p = 2$. According to Bolllrich (2013) this is given for full-model tests by

$$h_{c,V} = \left(\frac{2Q_{b,d}^2}{S_p^2 g}\right)^{1/5} = \left(\frac{Q_{b,d}^2}{2g}\right)^{1/5}$$

(6.1)
The best normalized time was obtained using $h_c \cdot \nabla$ compared to normalizations using $h_c$ for other flow cross-sections (Figure 6.1). The acceleration of gravity, and the sediment and water densities are replaced by the submerged specific gravity of the sediment as

$$g' = \left( \frac{\rho_s - \rho}{\rho} \right) g$$  \hspace{1cm} (6.2)

as described by Schmocker (2011). Specific gravity of the material was varied neither in this study nor by Schmocker (2011), however, and $g' = 1.65g$ was assumed for the evaluation. To describe apparent cohesion, the dimensionless particle parameter $D_*$ [-] as described by van Rijn (1984) and deduced from Shields (1936) was introduced in Eq.(2.5) as

$$D_* = d_{s0} \left( \frac{g'}{v^2} \right)^{1/3} \approx \frac{d_{s0}}{d_R}$$

with $v$ [m²/s] as kinematic viscosity, $d_{s0}$ [m] as the median sediment grain size and $d_R = 1/22,600 = 4.4 \times 10^{-5}$ m. While $D_*$ was introduced by van Rijn (1984) and Shields (1936) to describe initiation of sediment motion (motion is initiated faster for small $D_*$), it describes in this study the retaining force of apparent cohesion, which is larger for small $D_*$. According to Kovacs (1981), this linear relation is valid if the capillary force, often characterized by the capillary height, is linearly proportional to the surface tension $\sigma$ and inversely proportional to the horizontal pore size, which is proportional to $d_{s0}$. Both $\sigma$ and $v$ were assumed to remain constant and were therefore dropped. The standard tests were conducted by placing the dike on the fixed channel bed. If the bed consists of identical material as the dike, material is eroded below the original bed level ($z < 0$) thereby affecting the breach process. The maximum resulting scour depth is $w_{mobile}$. A relatively large pilot channel of notch height $w_p \approx (1/3)w$ and width $b_p \approx 2w_p$ was needed for $Q_{bd}$ to pass at breach start. The pilot channel sediment height at glass wall $w_{p,0} = w - w_p$ has an effect on the height of the headwater level $h_o$ and therefore on the breach process.

The breach process is described by setting the parameters accelerating the process divided by those decelerating the process as

$$\text{Breach process} \sim \frac{h_{c, V}, A_R, w_{p,0}, b_p, D_0, w_{mobile}, \epsilon}{w, \mu, \ell_f, \theta_s, \sigma_f, \nu, g'}$$  \hspace{1cm} (6.3)

The sediment grain size distribution $\sigma_g$ was found by Schmocker et al. (2014) to have a negligible effect on the plane breach, while $\theta$ and $\theta_s$ were sufficiently similar in all tests and $g'$ was not systematically varied. Porosity $\epsilon$ was found to have an effect only if no formwork was used for dike construction (Müller 2015), while variations of $\epsilon$ proved
negligible if standard formwork as Schmocker (2011) or more compaction work is applied. The downstream platform length \( L_f \), in part responsible for the stabilizing sediment body downstream of the breach, was not systematically varied and was assumed large enough to have no effect on the breach process. These parameters were therefore dropped, leading to the governing parameters of the spatial dike breach test with constant inflow discharge as boundary condition

\[
\text{Breach process} \sim \frac{h_{c,v}, A_g, W_{p,0}, h_p, D_w, w_{mobile}}{w, \mu}
\]  

The combination of \( w, g', \) and \( \rho \) allows for the description of the governing parameters in a dimensionless form as presented below.

### 6.4.2 Dimensionless time

As noted in Chapters 2, 4 and 5, the dike breach process with constant inflow discharge as hydraulic boundary condition can be subdivided into 3 stages for dikes built of non-cohesive sediment. During the first stage, the breach channel develops. During the second stage, the sediment erodes mainly vertically the downstream pilot channel reach until reaching its upstream end. This triggers the third stage, in which the inflow crest is fully developed and sediment is eroded both vertically and horizontally in the up- and downstream reaches. To compare the breach processes of the different tests, a dimensionless time \( T \) is introduced, which includes the main governing parameters and normalizes the evolution of the breach stages, such that the peak breach discharge \( Q_M \) or the maximum headwater level \( h_M \) occur at the same dimensionless time \( T \) for all tests (Figure 6.1). The non-normalized evolution of \( h_o \) and \( Q_b \) have a temporal shift as the maximum headwater level \( h_M \) and the peak breach discharge \( Q_M \) are reached. Once correctly normalized with \( T \), both \( h_M \) and \( Q_M \) occur in all tests at \( T \approx 7 \) and 13.12, respectively. This indicates that the different breach phases are well described by \( T \).

The effect of each governing parameter on \( T \) was determined by visualizing the breach phases in Figure 6.2 and using Figure 6.3. \( T \) was identified as

\[
T = t g^{h/2} \frac{h_{c,v} D_{s}^{1/5}}{\mu^{1/5} A_r^{7/10} w^{1/10}}
\]  

Its main governing parameters are therefore on the driving side \( t, g', h_{c,v} \) and on the retaining side \( \mu \) and \( A_r \), while the parameters \( D_s \) on the driving and \( w \) on the retaining side are less important for describing the breach process.
Figure 6.1  Normalization of (a) headwater level $h_o$, (b) breach discharge $Q_b$ as a function of
(—) dimensionless time $T$ of Eq. (6.5) and (—) if not normalized with $T$ (qualitative), for $w = 0.3$ m and $Q_{b,d}$ [l/s] = (—) 4.4, (—) 8.8, (—) 18.4, (—) 35.2,
(—) $w = 0.2$ m and $Q_{b,d} = 26$ l/s. (● ● ●) Maximum headwater level $h_M$ and peak
breach discharge $Q_M$ are reached at $T \approx 7$ and 13.1, respectively

The applicability of $T$ given in Eq. (6.5) for synchronizing the breach stages is shown in
Figure 6.2 for selected constant inflow discharge tests as defined in Table 6.1.

The dimensionless time at peak (subscript $M$) breach discharge was found to be $T_M = 13.12$. Notable deviations in $T_M$ occurred for Test 15 for the smallest dike height $w = 150$ mm and sediment grain size $d = 0.43$ mm, and Test 27 with an initial pilot channel
top notch width of $b_p = 1.2$ m. During quality control, as is also seen in Figure 6.3, Test
15 remained in the acceptable range of predicted to measured values of ±20%, whereas
Test 27 was outside of this range because of a breach behavior closer to 2D than 3D dike
breach tests, as was also concluded in Chapter 5, and was therefore excluded from the
data analysis.

Figure 6.2  Normalization of breach process based on $T$ of Eq. (6.5) for Tests (—) 01,
(—) 08, (—) 11, (—) 15, (—) 17, (—) 18, (—) 19, (—) 20, (—) 21,
(—) 26, (—) 27, (—is) 28, (—) 29 for $Q_b/Q_{b,d}(T)$. (● ● ●) indicates peak breach dis-
charge $Q_M$ for all tests at dimensionless time-to-peak $T_M \approx 13.12$
In Chapters 4 and 5, the constant inflow discharge tests are evaluated in a regular interval of $t [s] = 0, 10, 20, 40, 60, 100$ and 200. Since the stages of the breach process depend on $T$, the normalized breach characteristics are best compared by introducing the prominent ranges of $T$. The ranges for Tests 18 to 21 are shown in Table 6.5. Range 1 with $0 \leq T \leq 5$ is right after breach start with $h_o$ and $Q_b$ rising, in Range 2 with $5 < T \leq 10$, $h_o$ reaches its maximum whereas $Q_b$ is still rising, in Range 3 with $10 < T \leq 18$, $Q_b$ passes its maximum whereas $h_o$ falls, while both $Q_b$ and $h_o$ fall in Range 4 with $18 < T \leq 36$. In Range 5 with $T > 36$, only small variations in $Q_b$ and $h_o$ are noted, so that the entire breach process slows down considerably.

Figure 6.3  Effect of (a) $h_{c,v}$, (b) $\mu$, (c) $A_R$, (d) $w$, (e) $D_*$, (f) $t_{M,\text{meas}}$ for $T_M = 13.12$ on ratio of predicted to measured time-to-peak $t_M$ using Eq. (6.5) for tests of Table 6.1, (- - -) ±20% deviation

Dimensionless times ranges
The time ranges vary depending on the dike properties in accordance with $T$ in Eq. (6.5). As an example, for $t = 20 \text{s}$ and $g' = 1.65 \text{ m/s}^2$, $T = 5$ results from the following different base parameters for full-model tests:

- $h_{c,y}=0.11 \text{ m} (Q_{b,d}=18.4 \text{ l/s}), D_*=45.2 (d_{50}=0.002 \text{ m}), \mu=1, A_R=1 \text{ m}^2, w=0.3 \text{ m}$
- $h_{c,y}=0.06 \text{ m} (Q_{b,d}=4.4 \text{ l/s}), D_*=45.2 (d_{50}=0.001 \text{ m}), \mu=1, A_R=0.39 \text{ m}^2, w=0.15 \text{ m}$
- $h_{c,y}=0.21 \text{ m} (Q_{b,d}=86 \text{ l/s}), D_*=85.4 (d_{50}=0.00378 \text{ m}), \mu=0.7, A_R=3.5 \text{ m}^2, w=0.6 \text{ m}$

The dimensionless time $T$ and the dimensionless time ranges thus describe the spatial dike breach process in the following evaluations for dikes built of non-cohesive sediment for the test ranges previously described.

Comparison of dimensionless time $T$ to literature findings

Comparing $T$ of Eq. (6.5) for spatial dike breaches with the dimensionless time $T_{2D}$ of the plane (subscript 2D) dike breach, defined by Schmocker (2011) as

$$T_{2D} = tg^{\frac{1}{2}} \frac{h_c}{d^{\frac{1}{2}}w}$$  \hspace{1cm} (6.6)

the parameters $w$ and $d$ have a more prominent role in Eq. (6.6) and $d$ is included as a retaining instead of a driving parameter, while $\mu$ and $A_R$ (respectively $x_D$) are not included. As indicated by Schmocker (2011), the role of $d$ depends on the breach stage. Since erosion proceeds more slowly the larger $d$ for large test times, $d$ was introduced as a decelerating factor. The dike height $w$ describes the dike geometry, as the dike shapes are similar. For the spatial dike breach process described with Eq. (6.5), the dike cross-section $A_D$ is described using the dike height $w$ and the dike shape parameter $\mu$ as $A_D = (7/3)\mu w^2$. The prominent role of $w$ in $T_{2D}$ is therefore comparable to the role of $\mu$ and $w$ in $T$. Since the headwater level rises higher for larger $w$, which in turn increases sediment erosion, $w$ is not only a decelerating, but also an accelerating parameter, as highlighted by separating the parameters. The dimensionless time $T$ for spatial dike breaches of Eq. (6.5) agrees with the dimensionless time by Pickert et al. (2011) described in 2.8.6 concerning apparent cohesion as a retaining parameter, while it differs from that by Lüthi (2005) concerning the critical flow depth, which was inserted as a retaining parameter.

<table>
<thead>
<tr>
<th>$T$ ranges</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-5</td>
</tr>
<tr>
<td>5-10</td>
</tr>
<tr>
<td>10-18</td>
</tr>
<tr>
<td>18-36</td>
</tr>
<tr>
<td>36-72</td>
</tr>
</tbody>
</table>

**Table 6.5** Ranges for dimensionless time $T$ for Tests 18 to 21

<table>
<thead>
<tr>
<th>Test 18</th>
<th>Test 19</th>
<th>Test 20</th>
<th>Test 21</th>
</tr>
</thead>
</table>
| $t$ [s] | $T$ | $T$ | $T$ | $T$
| 0 | 0.0 | 0.0 | 0.0 | 0.0 |
| 10 | 1.5 | 2.0 | 2.7 | 3.5 |
| 20 | 3.1 | 4.1 | 5.4 | 7.1 |
| 40 | 6.2 | 8.2 | 10.8 | 14.2 |
| 60 | 9.3 | 12.2 | 16.1 | 21.3 |
| 100 | 15.4 | 20.4 | 26.9 | 35.5 |
| 200 | 30.9 | 40.8 | 53.8 | 71.0 |
### 6.4.3 Peak breach discharge

Figure 6.4 shows the dimensionless peak breach discharge \((Q_M/Q_{b,d})^{-1}\) versus the governing parameters of Eq. (6.4) for tests using constant inflow discharge as a hydraulic boundary condition. The mobile bed had no effect on \(Q_M\), so that the maximum scour depth \(w_{mobile}\) is not included.

![Figure 6.4](image-url)

**Figure 6.4** Dimensionless maximum breach discharge \((Q_M/Q_{b,d})^{-1}\) for full-model tests as a function of \(h_{c,\bar{y}}^{7/10} \mu^{1/5} b_p^{1/10} D_*^{1/10} (A_R w_{p,0} w)^{-1/5}\) with constant inflow discharge \(Q_o\) along with \((-\)) Eq. (6.7) \((R^2=0.93)\)

The fit equation for the data in Figure 6.4 may be expressed with \(R^2=0.93\) as

\[
\frac{Q_M}{Q_{b,d}} - 1 = 13 \exp \left[ -8.3 \frac{h_{c,\bar{y}}^{7/10} \mu^{1/5} b_p^{1/10} D_*^{1/10}}{A_R^{1/5} w_{p,0}^{1/5} w^{1/5}} \right] \text{ if } 0.23 \leq \frac{h_{c,\bar{y}}^{7/10} \mu^{1/5} b_p^{1/10} D_*^{1/10}}{A_R^{1/5} w_{p,0}^{1/5} w^{1/5}} \leq 0.47 \tag{6.7}
\]

Herein the maximum breach discharge is \(Q_M\) [m³/s], the design drainage breach discharge \(Q_{b,d}\) [m³/s], the critical flow depth \(h_{c,\bar{y}}\) [m], the dike shape parameter \(\mu\) [-], the pilot channel notch top width \(b_p\) [m], the pilot channel sediment height at the glass wall \(w_{p,0}\) [m], the reservoir water surface area \(A_R\) [m²], the dike height \(w\) [m], and the particle parameter is \(D_*\) [-]. The comparison of measured \(Q_{M,meas}\) to the predicted \(Q_{M,pred}\) in Figure 6.5 shows excellent agreement \((R^2 = 0.98)\). In summary, the ratio \(Q_M/Q_{b,d}\) becomes small for large \(h_{c,\bar{y}}, \mu, b_p\) and \(D_*\) and for small \(A_R, w, w_{p,0}\) up to \(Q_M/Q_o \rightarrow 1\) for which no peak discharge occurs, in agreement with observations in Chapters 4 and 5.
The dimensionless time at maximum breach discharge was previously found to be \( T_M = 13.12 \), so that the corresponding dimensional time at \( Q_M \) is from Eq. (6.5)

\[
t_M = 13.12 \frac{\mu^{3/4} A_{R}^{7/10} W^{1/10}}{h_c \sqrt{g} D^{1/5}}
\]  

(6.8)

The measured \( t_{M,\text{meas}} \) and predicted \( t_{M,\text{pred}} \) in Figure 6.6 agree well, with \( R^2 = 0.96 \). In summary, \( t_M \) becomes large for large \( \mu, A_R \) and \( w \), while it becomes small for large \( h_c, \nabla \) and \( D \ast \) (respectively small effect of apparent cohesion).
6.4.4 Maximum headwater level

Figure 6.7 shows the dimensionless maximum headwater level \( h_M/w \) versus the governing parameters of Eq. (6.4) for tests using constant inflow discharge as a hydraulic boundary condition. The mobile bed and pilot channel height \( w_p \) had no effect on \( h_M \), so that these were not included in the fit equation.

\[
\frac{h_M}{w} = \frac{h_{c,v}^{9/10} \mu^{1/2} b_p^{-1/2} A_R^{-1/5} D_*^{-1/20}}{A_R^{1/3} b_p^{1/2} D_*^{1/20}} \geq 0.07 \leq 0.17
\]  

As a conclusion, for the tests using constant inflow discharge as a hydraulic boundary condition, the ratio of \( h_M/w \) becomes small for small \( h_{c,v}, \mu \) and large \( A_R, b_p \) and \( D_* \), down to \( h_M/w \to 0.59 \), which is close to the ratio \( w_p,0/w \approx 2/3 \) for most tests. These trends are in accordance with observations in Chapters 4 and 5.

This almost linear relation between \( h_{c,v} \) and \( h_M/w \) is essential for combining the test data using different hydraulic boundary conditions, namely (1) constant inflow discharge \( Q_o \neq 0 \) (6.4), and (2) \( Q_o = 0 \) and varying reservoir water surface areas \( A_R \) (6.5). A generalized formulation for all conducted tests is therefore feasible in principle using this quasi-linear relation.
6.4.5 Maximum dike height

Figure 6.8 shows the dimensionless maximum dike height $Z_M$ [-] defined as

$$Z_M = \frac{z_M}{w}$$

(6.10)

with the maximum dike height $z_M$ [m] along the central streamwise breach profile ($\gamma = 0$) and the initial dike height $w$ [m] versus dimensionless time $T$ from Eq. (6.5) and further governing parameters. The pilot channel notch height $w_p$ had no effect on $Z_M$ and was therefore excluded from the fit equation. Although the mobile bed influences $Z_M$ (as described in Chapter 5), its inclusion does not improve the prediction of $Z_M$. Further tests are therefore necessary to assess its possible effect.

![Figure 6.8](image)

**Figure 6.8** Evolution of dimensionless maximum dike height $Z_M(TAR^{7/10} \mu^{-1} w^{-7/5} D_*^{-1/5})$ for full-model tests with constant inflow discharge $Q_o$ of Table 6.1 along with (-) Eq. (6.11) ($R^2=0.94$)

As to the data trend in Figure 6.8, $Z_M$ starts at the initial pilot channel sediment height $w_{p,0}$ at the breach center ($\gamma = 0$), and does not drop as long as the downstream portion of the pilot channel is eroded up to $TAR^{7/10} \mu^{-1} w^{-7/5} D_*^{-1/5} = 50$, when the upstream pilot channel reach starts eroding and $Z_M$ drops, as described in Chapters 4 and 5. For Tests 1 to 3 with $w = 200$ mm, a difference in initial values for $Z_M$ is noted due to the different ratio of pilot channel height to initial dike height. This does not change the erosion behavior, however, although the drop is only visible at $TAR^{7/10} \mu^{-1} w^{-7/5} D_*^{-1/5} = 100$. For the data evaluation below, data smaller than this limit value were dropped, therefore. The fit equation of Figure 6.8 may be expressed with $R^2 = 0.94$ as
\[ Z_M = -0.187 \ln \left( \frac{T A_r^{7/10}}{\mu w^{7/5} D_v^{1/5}} \right) + 1.4 \text{ for } \frac{T A_r^{7/10}}{\mu w^{7/5} D_v^{1/5}} > 50 \text{ and } Z_M < 0.7 \]  

(6.11)

The effect of the different parameters on \( z_M \) can be further highlighted when substituting \( T \) from Eq. (6.5), resulting in

\[ Z_M = \frac{z_M}{w} = -0.187 \ln \left( tg^{1/2} \frac{h_c \gamma}{\mu w^{1/5} w^{1/2}} \right) + 1.4 \]  

(6.12)

Eq. (6.12) indicates that the evolution \( z_M \) of is independent of \( A_R \) and \( D_* \), while \( z_M \) becomes small for large \( t, g', h_c, \gamma, w, \) and large for large \( \mu \). These results were tested (as previously done for \( T \)) concerning the effect of each governing parameter on \( z_M \) by plotting the ratio of predicted to measured values relative to the parameter of interest (Figure 6.9). Figure 6.9a indicates, that for \( T A_r^{7/10} \mu^{-1} w^{-7/5} D_*^{1/5} > 500 \), the predicted values for \( z_M \) tend to be slightly lower as compared to the measured values, which is possibly due to slight stabilizing effects at the inflow crest at the glass wall for large test times, leading to higher values of \( z_M \) compared to few centimeters from the glass wall, as described in Chapter 3. Figure 6.9b shows that the particle parameter is well balanced using Eq. (6.12), and consequently does not affect the temporal evolution of \( z_M \).

**Figure 6.9** Effect of (a) \( T A_r^{7/10} \mu^{-1} w^{-7/5} D_*^{1/5} \) and (b) \( D_* \) on temporal evolution of predicted to measured maximum dike heights \( z_M \) from Eq. (6.12) for tests of Table 6.1, (- - -) ±20% deviation
6.4.6 Eroded sediment volume

Figure 6.10 shows the dimensionless eroded sediment volume $V_E/(A_D A_R^{1/2})(w_{p,0}/w)$ versus time $T$ for constant inflow discharge tests. For $T < 5$, no substantial sediment erosion occurs, except for Test 27, due to properties similar to plane dike breaches so that it was excluded from the data analysis, as previously described. The transition from zero to substantial sediment erosion is gradual, but its description is difficult mainly because (1) the temporal resolution was limited as the eroded sediment volume $V_E$ was evaluated at well-defined times (Chapter 5), and (2) small changes at $T \approx 5$ are of the order of magnitude of the accuracy of the applied photogrammetric method and the subsequent evaluation. Nevertheless, for $5 < T < 18$, a clear trend is visible when plotting the data on an $x$-axis with linear scale, with the eroded sediment volume increasing fast due to the fast rising headwater level $h_o$ and the large breach discharge $Q_b$, before slowing down once $h_o$ and $Q_b$ drop. For tests involving a mobile sediment bed, $w$ was approximated as $w = w_{\text{fixed}} + w_{\text{mobile}} = w_{\text{fixed}} + |z_{\text{min}}|$, increasing the dike cross-sectional area available for erosion. The fit equation for the data in Figure 6.10 follows with $R^2=0.96$

$$\frac{V_E w_{p,0}}{A_D A_R^{1/2} w} = 0.092 T^{3/10} - 0.146 \quad \text{for } T \geq 5$$  \hspace{1cm} (6.13)

or

$$V_E = A_D A_R^{1/2} w w_{p,0}^{-1} \left(0.092 T^{3/10} - 0.146\right) \quad \text{for } T \geq 5$$  \hspace{1cm} (6.14)

Eq. (6.14) indicates that $V_E$ becomes large for large $t$, $h_c$, $\nabla$, $D_*$, $w$ and $\mu$, while $V_E$ becomes small for large $A_R$.

![Figure 6.10](image_url)  
**Figure 6.10** Dimensionless eroded sediment volume $V_E/(A_D A_R^{1/2})(w_{p,0}/w)$ versus dimensionless time $T$ with (---) Eq. (6.13) ($R^2=0.96$) for tests of Table 6.1
Figure 6.11 Effect of (a) $T$, (b) $D_*$ on temporal evolution of predicted to measured eroded dike volume $V_E$ based on Eq. (6.14) for tests of Table 6.1. (- - -), $\pm 20\%$ deviation

The ratio $V_{E,pred}/V_{E,meas}$ versus $T$ and $D_*$ by applying Eq. (6.13) is shown in Figure 6.11 indicating a satisfactory match and trend along the dimensionless time and the particle parameter, respectively. Deviations larger than $\pm 20\%$ occur for $T < 15$ in Figure 6.11a, which coincides approximatively with $t_M$, and are attributed to the small values compared to each other, which are therefore prone to larger deviations. On the contrary, the deviations in Figure 6.11b do now show a trend concerning sediment grain size, although sediment with the largest $D_*$ and thus smallest effect of apparent cohesion deviates the least.

### 6.4.7 Hourglass shape

For tests using constant inflow discharge as a hydraulic boundary condition, the headwater level $h_0$ rises at test start, surpassing the lowest point of the pilot channel base at $w_{p,0}$, initiating the breach and creating a breach channel. The latter then widens in the inflow reach, forming the typical hourglass shape, which marks the limit between the submerged and non-submerged dike portions (Figure 6.12). The normalization of the hourglass shapes $Y(X)$ (the limits) for Test 1 are shown in Figure 4.18d, with the normalized width $Y = y_{hg}/y_{hg,d}$, with $y_{hg}$ as the transverse position of each point relative to the smallest $y_{min}$ of the hourglass shape and $y_{hg,d}$ as the difference between the extremes $y_{max}$ and $y_{min}$ (Figure 3.11). The hourglass shape depends on the headwater level, first rising to $h_M$ and then falling, and the breach discharge $Q_b$, increasing up to $Q_M$ for falling $h_0$, before eventually reaching the value of the design breach discharge $Q_{b,d}$. 

![Normalized results](image-url)
Figure 6.12  Hourglass shape from Figure 4.18 for \( w = 300 \), with submerged portions in blue, and non-submerged portions in orange, (●●●) hourglass shape at \( t = 100 \) s

The hourglass shapes \( y_{hg}(x) \) of Tests 18 to 21 with \( Q_{h,d} = 2.2 \) to 17.6 l/s were determined and are considered representative for the remaining tests, since the effect of \( Q_{h,d} \) on the erosion rate is large compared to other parameters, as e.g. the sediment size \( d \). The normalized breach widths defined by the hourglass shape \( y_{min}/(h_c \cdot b_p)^{1/2} \) and \( y_{max}/(h_c \cdot b_p)^{1/2} \) are shown versus \( T \) in Figure 6.13. The smallest breach width \( y_{min} \) first widens fast up to \( T < 18 \), then continues widening more slowly. The breach width at the upstream end of the hourglass shape \( y_{max} \) widens roughly 2.5 times as fast as \( y_{min} \) for \( T < 18 \), before remaining almost constant at \( y_{max}/(h_c \cdot b_p)^{1/2} \approx 4 \). Figure 6.14 shows the normalized hourglass shapes \( Y(X) \) for Tests 18 to 21 with \( Q_{h,d} = 4.4 \) to 35.2 l/s (Table 6.1) versus \( T \). After breach start up to \( T < 5 \), the breach channel forms so that the scatter of hourglass shapes is large. For \( 5 < T < 10 \), the breach channel with its hourglass shape has formed and is described with the power function approximation \( Y = p X^{1/n} \) with \( p = x_{hg,d} y_{hg,d}^{1/n} \) for \( n = 3 \) (Bollrich 2013). The normalized shape then becomes less curved for \( T > 10 \), and is described using \( n = 2 \).

Figure 6.13  Normalized breach width versus \( T \) (a) at narrowest section of hourglass shape \( y_{min} \), (b) at upstream end of hourglass shape \( y_{max} \) for \( Q_{h,d} \) [l/s] = (●●●) 4.4, (●●●) 8.8, (●●●) 18.4, (●●●) 35.2 (Tests 18 to 21)
Figure 6.14  Dimensionless hourglass shape in $Y(X,T)$ for $Q_{b,d} [l/s] = (-) 4.4$, $(-) 8.8$, $(-) 18.4$, and $(-) 35.2$, (Tests 18 to 21) with dimensionless transverse submerged sediment surface profiles $Z_{b}(Y_{b})$ at $t [s] = (▲) 10$, (●) 20, (●) 40, (★) 60, (○) 100, and (□) 200. Power function exponents $n = (-) 2$, $(-) 3$
6.4.8 Transverse submerged sediment surface profiles

The transverse submerged flow profiles $z(y)$ of Tests 18 to 21 at $x$ [mm] = 400, 530, 650, 770, 900, 1030 were determined and are considered representative for the remaining tests, due to the important effect of $Q_{b,d}$ on the erosion rate as compared to other parameters, as e.g. the sediment size $d$. The transverse submerged flow profiles $z(y)$ were extracted using the same method as for Test 1 in Chapter 4, with the highest point located at the intersection of the respective water surface elevation $h(x,t)$ and the sediment surface profile $z(x,y,t)$, and the lowest point located at the breach center $z(x,y=0,t)$. Figure 6.15 shows the normalized submerged breach profiles $Z(Y)$ with $Y = y/b_b$ and $Z = z/h_b$ with $y$ and $z$ as the respective coordinates in the submerged breach half-profile of width $b_b$ and water depth $h_b(y=0)$ for different ranges of $T$.

After breach start ($T<5$), the breach profiles tend to follow the power function $Y_b = 0.5pZ_b^{1/n}$ with $p = 2b_b/h^{1/n}$ and $n = 2$, except for the downstream portion $x>650$ mm, for which $n = 3$ is better suited. As described in Chapter 4, the downstream cross-sections are difficult to determine using the photogrammetric system and its quality depends on the respective hydraulic conditions. For $5<T<10$, in which the maximum headwater level $h_M$ is reached, the spread of the profile shapes increases. For profile shapes which follow $n<2$, as for Test 21, this is due to its position at the limit of the inflow crest for $x = 530$ mm, and for $x = 700$ mm due to refraction and triangulation effects. The remaining profiles are contained in profiles with exponents $n = 2$ and $3$, with location $x = 1030$ mm deviating from the power function shapes with a relatively flat base. For $10<T<18$, once the headwater level $h_0$ drops and peak breach discharge $Q_M$ is reached, the profile shapes tend toward a shape with $n = 2.5$. For $18<T<36$, $Q_{b,d}$ and $h_0$ drop and the shape of tests with larger $Q_{b,d}$ tend toward $n = 2$, while for smaller $Q_{b,d}$ the shapes tend to $n > 3$, which is partially attributed to inaccuracies of the transverse water surface inclination, as described in Chapter 4. For $T>36$, the profile shapes vary between $n = 2$ and $3$, tending toward 2.5.

As a conclusion, the normalized transverse submerged flow profiles $Z(Y)$ is described with the power function $Y_b = 0.5pZ_b^{1/n}$ with $p = 2b_b/h^{1/n}$ and $n = 2.5$, even though inaccuracies in the profiles exist which are attributed to steep transverse water surface profiles, as described in Chapter 4. When considering this effect, the power function approximation with $n = 2$ is assumed to be valid and can be applied in parametric models or for the validation of numerical simulations of spatial dike breaches of non-cohesive sediment. On the contrary, the slope of the non-submerged breach channel portions depend mostly on apparent cohesion and need to be taken into account separately.
Figure 6.15 Spatial dike breach evolution of (—) Test 18 ($Q_{b,d} = 4.4$ l/s), (—) Test 19 ($Q_{b,d} = 8.8$ l/s), (—) Test 20 ($Q_{b,d} = 9.2$ l/s), (—) Test 21 ($Q_{b,d} = 35.2$ l/s), with dimensionless transverse submerged sediment surface profiles $Z_b(Y_b)$ at $x$ [mm] = (—) 400, (—) 530, (—) 650, (—) 770, (—) 900, (•••) 1030, at $t$ [s] = (▲) 10, (●) 20, (••) 40, (⋆) 60, (○) 100, and (□) 200. Power function $Y_b = 0.5pZ_b^{1/n}$ with $p = 2b_h/h^{1/n}$ for $n = (—) 2$, (—) 3.


6.5 Reservoir water surface area tests

6.5.1 New governing parameters

The reservoir test series described in 5.8 use as a hydraulic boundary condition simulated reservoirs of different water surface areas $A_R$ without inflow discharge, i.e. $Q_o = 0$ (Table 6.2). The values from the half-model tests are presented in this chapter as full-model test results to facilitate comparison and field application. To include the data from tests using constant inflow discharge $Q_o$ as a hydraulic boundary condition, common governing parameters are needed. Therefore, it was assumed that the peak breach discharge $Q_M$ does not depend on $h_{c,y}$, but on the maximum headwater level $h_M$ reached, and on the size of the water body upstream of the dike, described by the reservoir water surface area $A_R$. To validate this assumption, Tests 21 and 40 with a similar $h_M \approx 300$ mm and $A_R = 6.88$ m², but with different hydraulic boundary conditions $Q_{b,d} = 35.2$ and 0 l/s, respectively, were compared concerning the temporal evolution of $h_o$ and $Q_b$ in Figure 6.16. The different hydraulic boundary conditions are visible essentially in the evolution of $h_o$, while that of $Q_b - Q_{b,d}$ proves to be similar despite a difference of $Q_M - Q_{b,d} \approx 4$ l/s between the tests.

![Figure 6.16](image-url) Qualitative temporal comparison of (a) headwater level $h_o(t)$ and (b) breach discharge $Q_b(t)$ for $A_R = 6.88$ m² and $h_M \approx 300$ mm for (--) Test 40 with $Q_{b,d} = 0$ l/s and (--) Test 21 with $Q_{b,d} = 35.2$ l/s

As a conclusion, tests based on constant inflow discharge and tests involving simulated reservoirs without inflow discharge $Q_o = 0$ with the same maximum headwater level $h_M$ and water surface areas $A_R$ are similar concerning the temporal breach discharge evolution $Q_b - Q_{b,d}$, as also demonstrated below. Therefore, $h_{c,y}$ is replaced by $h_M$, so that the governing parameters of the spatial dike breach are expressed as

$$\text{Breach process } \sim \frac{h_M, A_R, w_{p,0}, b_p, D_s, w_{\text{mobile}}}{w, \mu} \quad (6.15)$$
6.5.2 Peak breach discharge

Figure 6.17 shows the dimensionless peak breach discharge \( \left( Q_M - Q_{b,d} \right) / (A_Rg^{0.5}w^{0.5}) \) versus the governing parameters in Eq. (6.15) for the tests of Table 6.1 and Table 6.2. The fit equation may be expressed with \( R^2 = 0.97 \) as

\[
\frac{Q_M - Q_{b,d}}{A_Rg^{\frac{1}{2}}w^{\frac{1}{2}}} = 0.0067 \left( \frac{h_d}{w} \right)^{13/10} w^{3/10} \left( \frac{A_R}{A_{Rd}} \right)^{2/5} \left( \frac{\mu}{\mu_d} \right)^{-2/5} < 0.68
\]  

(6.16)

Tests 27 and 44 and the repeatability Tests 2 and 3 were not included in the data evaluation, as previously described. For the remaining tests, the data representation by the fit equation is satisfactory. Only Test 14 deviates visibly from the fit, even though the test was successfully conducted. The deviation results from the relatively small inflow discharge \( Q_o \) and large peak drainage discharge \( Q_{dr} \), due to the large sediment grain size, leading to a reduced peak breach discharge \( Q_M \) of up to 26%, as described in Chapter 4. For the reservoir tests, this issue was solved by compensating \( Q_{dr} \) using the controller.

![Figure 6.17 Dimensionless maximum breach discharge for full-model tests versus governing parameters with (—) Eq. (6.16) and tests of Table 6.1 and Table 6.2](image)

With the dike cross-sectional area \( A_D = (7/3) \mu w^2 \), the equation is transformed to

\[
Q_M - Q_{b,d} = 0.0156g^{\frac{1}{2}} h_d^{13/4} A_R^{\frac{3}{8}} A_D^{-3/20}
\]  

(6.17)

When drainage discharge \( Q_{dr} \) is negligible, \( Q_{b,d} \approx Q_o \), so that the design drainage discharge \( Q_{b,d} \) may be replaced with the inflow discharge \( Q_o \). For the model tests, \( Q_{dr} \) was up to 29% of \( Q_o \), and needs to be considered. The values predicted using Eq. (6.17) are compared to measured values in Figure 6.18 and show satisfactory agreement, with \( R^2 = 0.97 \).
6.5.3 Reservoir shape

As described in Chapter 5, the reservoir tests were conducted using mainly rectangular reservoir shapes of $A_R = 6.88$ to $206.88$ m² and one triangular reservoir test with $A_R = 26.88$ m². The normalized breach discharge evolution $Q_b/Q_M$ versus $t/t_M$ is shown in Figure 6.19. While the tests using a rectangular reservoir shape have a similar evolution of $Q_b/Q_M$ described by the blue line, it differs considerably for the triangular reservoir shape for $t/t_M > 1$. Therefore, the normalized breach discharge hydrographs are similar for tests of identical dike geometry and reservoir shape for different $A_R$. For different reservoir shapes, the normalized breach discharge hydrographs differ. Nevertheless, $Q_M$ appears to be similar, due to the similar $A_R$ in the breach development leading to $Q_M$.

Figure 6.18 Comparison of measured and determined difference of peak breach discharge and design breach discharge $Q_M - Q_{b,d}$ applying Eq. (6.17) ($R^2=0.97$) for tests of Table 6.1 and Table 6.2. (—) Line of perfect agreement

Figure 6.19 Normalized evolution of peak breach discharge for reservoir tests of Table 6.2 with rectangular shape and $A_R$ [m²] = (—) 6.88, (———) 26.88, (---) 33.12, (—•—) 66.88, (—×—) 126.88, (---) 206.88, and (—) triangular shape with $A_R = 26.88$ m². (——) Typical normalized hydrograph for rectangular reservoirs
6.6 Comparison to further laboratory data

To validate the test results and the applied evaluation methods, the fit equations for maximum breach discharge \( Q_M \) are compared to laboratory test results of Wallner (2014) and Walder et al. (2015). Wallner’s tests were conducted with the dike properties \( w = 300 \text{ mm}, \mu \approx 0.9, d_{50} = 1 \text{ mm}, V_R = 1 \text{ to } 4 \text{ m}^3 (A_R \approx 3.3 \text{ to } 24.6 \text{ m}^2) \), rectangular and triangular reservoir shapes. As described in Chapter 2, the reservoir volume was used to normalize and compare the peak breach discharge, and two different partial regression lines were proposed for the rectangular and triangular reservoir shapes. Walder et al. (2015) studied dikes with the properties \( w = 0.57 \text{ to } 1 \text{ m}, \mu \approx 0.9 \text{ to } 2.1, d = 0.21 \text{ mm}, A_R = 18.07 \text{ to } 23.7 \text{ m}^2 \), the variation of \( A_R \) versus \( h_o \) being quasi-rectangular and thus negligible.

The results of both studies are compared based on Eq. (6.16) with \( Q_{b,d} = Q_o = 0 \) in Figure 6.20. The results of Wallner (2014) agree quite well for both the rectangular and triangular reservoir shapes, indicating that the reservoir water surface area \( A_R \) is a better parameter for determining \( Q_M \) as the reservoir water volume \( V_R \). The results of Walder et al. (2015) agree well for \( w \leq 0.87 \text{ m} \). Only tests with \( w = 1 \text{ m} \) differ notably from Eq. (6.16), with a certain scatter between the repeated tests themselves. Test 5 of Walder et al. (2015) agrees best with Eq. (6.16), while Test 4 deviates more. Differences between these two are due to the modeled distribution of pressure head, possibly an influencing factor in the breach process.

![Figure 6.20](image-url)  
**Figure 6.20** Comparison of (—) Eq. (6.16) for dimensionless maximum breach discharge for full-model tests versus governing parameters with results of Wallner (2014) for (□) rectangular and (△) triangular shape, and of Walder et al. (2015) for \( w [\text{ m}] \approx (\bigcirc) 0.57, (\bigodot) 0.66, (\bigtriangleup) 0.75, (\bullet) 0.87, (\bigcirc) 1.00 \)
In Figure 6.21, the predicted $Q_{M,pred}$ applying Eq. (6.17) are compared to the laboratory data of Wallner (2014) and Walder et al. (2015). The results agree well, except again for tests with $w = 1$ m of Walder et al. (2015), with $R^2 = 0.68$ when including and $R^2 = 0.99$ when excluding tests with $w = 1$ m.

Therefore, the reservoir water surface area $A_R$ proves to be the adequate parameter for describing the peak breach discharge $Q_M$ for tests using non-cohesive granular material for reservoirs of varied size and shape. Furthermore, the collapse between the three data sets using Eq. (6.17) and sediment grain sizes of $d = 0.21$ to $1.75$ mm for larger reservoir sizes $A_R$ indicates that the effect of $d$ is small, as also noted by Walder et al. (2015). Due to uncertainty concerning the large tests of Walder et al. (2015), further tests with larger $A_R$ and different sediment size $d$ should be conducted to clarify this issue.

The application of Eq. (6.17) to model dike breach tests with the hydraulic boundary conditions constant inflow discharge and reservoir with $Q_o = 0$ delivers satisfactory results, and is assumed to work well for reservoir tests with $Q_o = constant$ or breaches induced by wave overtopping. For the other hydraulic boundary conditions presented in 2.6, i.e. tests with a constant headwater level or fluvial tests, Eq. (6.17) cannot be directly applied. For a constant headwater level, $A_R$ and therefore also $Q_M$ would be infinitely large. For the fluvial tests, the downstream river condition will influence the headwater level evolution. One approach could be to consider the downstream river outflow discharge $Q_d$ in as $Q_{b,d} = Q_o - Q_{dr,d} - Q_{dd}$ with the design river outflow discharge $Q_{dl,d}$ (see also 3.5).
6.7 Comparison to prototype data

Eq. (6.17) was developed from spatial dike breach model tests with homogeneous dikes built of uniform non-cohesive sediment and shows good agreement with other laboratory model test data in 6.6. Prototype earth dams in contrast are often zoned and built with an impermeable surface layer or core of cohesive material. The building material usually consists of earth, sand, or rockfill material, which is generally compacted with an optimum water content to improve dike stability and reduce deformations and seepage. Earth dams therefore have a higher erosion resistance as compared to the easily erodible model sand dike in the laboratory. When comparing the predicted values of Eq. (6.17) to the reconstructed values of prototype dike breaches from Wahl (1998) and Froehlich (2016a), therefore, a systematic shift was observed in the data distribution. Thus, a fit parameter $k$ describing the higher erosion resistance of the dam material was introduced in Eq. (6.17), which can then be written as

$$Q_M - Q_{M,d} = 0.0156g^{1/2} h_{M}^{1/4} A_R^{3/8} A_D$$

For $k = 2.4$, the predicted values for the peak breach discharge and the reconstructed values agreed well for the dams built of earth, sand and rockfill material for the overtopping and the piping scenario (Figure 6.22). Two points have a strong deviation from the line of perfect agreement: (1) For the dam protected by a concrete layer, the maximum breach discharge value is smaller by a factor of 10, which is attributed to the stabilizing effect of the concrete layer; (2) for the dam built of coal waste, the reconstructed maximum breach discharge is much larger than the predicted value. This is in part attributed to the smaller sediment density of coal waste, which was not varied in this study and not considered for the comparison. Furthermore, the large dike shape parameter $\mu = 4.5$ is much above the maximum value studied in the test range with $\mu_{\text{max}} = 1.43$. For the fit using $k = 2.4$ in Figure 6.22, $R^2 = 0.98$ when including and $R^2 = 0.997$ when excluding the dike with a concrete layer and the dike built of coal waste.

Eq. (6.18) was applied to six data points only (Figure 6.22), since for most prototype dike breaches in the list of Walder et al. (1998) or in other literature not all necessary data are available, i.e. in most cases the reservoir water surface area $A_R$ or the peak breach discharge $Q_M$ are missing. Therefore, Eq. (6.18) needs more data for the calibration of the fit parameter $k$ and to determine the equations application limits. Nevertheless, these first results are promising.
Case studies

6.8.1 Case study 1: Landslide dam

As described in Chapter 2, the setup using constant inflow discharge as a hydraulic boundary condition for the spatial dike breach represents the optimum condition if a landslide suddenly blocks a river (similar to a dike), leading to impoundment upstream, until water overflows the lowest point and initiates the breach. A brief application of the present results is presented below.

The prototype dike values are assumed to be $w \approx 3$ m, $w_p \approx 1$ m, $d_{50} \approx 10$ mm, $S_o = S_d \approx 2$, and $L_K \approx 1$ m. Compared with the model tests, these scales are in a range of 1:5 to 1:10. The particle parameter is $D_{\text{max}} \approx 226$ according to Eq. (2.5). The river discharge at landslide occurrence is set to $Q_o \approx 5$ m$^3$/s, resulting according to Eq. (6.1) in $h_c, v \approx 1$ m if $Q_o$ equals $Q_{b, d}$ (i.e. drainage discharge assumed negligible). The reservoir water surface area is assumed to be $A_R = 500$ m$^2$ when the maximum headwater level $h_M = 2.85$ m is reached from Eq. (6.9), using the ratio $w_p, 0/ w = 0.66$. For the following equations to be valid, it is
assumed that the dike is internally stable during the entire breach process and is only subjected to surface erosion.

Applying Eqs. (6.7) and (6.8), the peak breach discharge $Q_M \approx 6.5 \text{ m}^3/\text{s}$ occurs at $t_M \approx 95 \text{ s}$ ($T=13.12$) after breach start. In accordance with Eq. (6.11), the downstream reach of the initial pilot channel is eroded up to $t \approx 65 \text{ s}$ ($T=8.9$), after which the inflow crest erodes fast at start and then slows down from initially $z_M \approx 2 \text{ m}$ to $z_M \approx 1.15 \text{ m}$ ($t \approx 5 \text{ min}$) and $z_M \approx 0.37 \text{ m}$ ($t \approx 20 \text{ min}$). From Eq. (6.14), the eroded dike volume $V_E$ increases rapidly for $t > 36.5 \text{ s}$ ($T>5$), with $V_E \approx 38 \text{ m}^3$ at $t \approx 1 \text{ min}$, $V_E \approx 190 \text{ m}^3$ at $t \approx 2.5 \text{ min}$ and $V_E \approx 280 \text{ m}^3$ after $t \approx 5 \text{ min}$.

The input parameters satisfy the limitations for the dimensionless relations as follows:

Eq. (6.7) for $Q_M$  
$$0.23 \leq \frac{h_{x,N}^{1/10} \beta_{p}^{1/5} D_{x}^{1/10}}{A_{R}^{1/5} \beta_{p}^{1/2} D_{x}^{1/20}} = 0.46 \leq 0.47$$

Eq. (6.9) for $h_M$  
$$0.07 \leq \frac{h_{x,N}^{1/10} \beta_{p}^{1/2}}{A_{R}^{1/5} \beta_{p}^{1/2} D_{x}^{1/20}} = 0.11 \leq 0.17$$

Eq. (6.11) for $z_M$  
$$\frac{T_{\text{min}} A_{R}^{1/10}}{\beta W^{3/5} D_{x}^{1/20}} = 50.03 \geq 50 \text{ and } Z_{M,\text{max}} = 0.66 < 0.7$$

Eq. (6.13) for $V_E$  
$$T_{\text{min}} = 5.02 \geq 5$$

Therefore, the maximum breach discharge, the time of occurrence, the drawdown of the inflow crest at the breach center and the eroded dike volume are determined for dike breach prototypes of constant inflow discharge as the hydraulic boundary condition. However, these results are only valid for embankments built of uniform granular material, without surface seals, impermeable core or grass cover. The embankment must remain stable during the breach process (not subjected to failure due to seepage or piping, geotechnical failure, etc.), so that the breach process is gradual and mainly subjected to surface erosion. The limitations of the obtained results should be carefully considered, especially since scaling is possible with certain restrictions, as described in Chapter 4.

### 6.8.2 Case study 2: 1975 Banqiao Dam failure

In August 1975, an extreme storm occurred in the Henan Province, China, leading to the breach of several dams. Banqiao Dam was one of the large breached dams, which led to a death toll of thousands. The event is described by Xu et al. (2008) and updated by Froehlich (2016a), who reduced the long established estimate to $Q_M \approx 56'300 \text{ m}^3/\text{s}$. An application of the obtained results for reservoirs is presented below. Although the dam is
a clay-core earthfill dam, it is assumed to consist of granular material. The reservoir-storage data in Figure 6.23 was kindly provided by Dr. Froehlich. Figure 6.23 shows $A_R$ versus $h_o$. The dam values are $w \approx 24.5$ m, $L_K \approx 6$ m, $S_o = S_d \approx 2$ (estimated), leading to $A_D = 1,347$ m². The inflow discharge during the breach was $Q_o \approx 6,000$ m³/s, reduced due to spillway discharge to $Q_o \approx 3,000$ m³/s. Drainage discharge is judged negligible so that $Q_{b,d} = Q_o$. The maximum headwater level was $h_M \approx 24.79$ m above the dike basis. The reservoir water surface area was determined from Figure 6.23 with $A_R(h_o=24.79$ m) $\approx 63.3$ Mm².

![Figure 6.23](image)

When applying Eq. (6.17), and considering $A_R \approx 63.3$ Mm² during maximum headwater level, $Q_M \approx 122,000$ m³/s. This is an overestimation of 116% as compared to Froehlich (2016a). This deviation is mainly attributed to the higher resistance of the sediment and the cohesive clay-core in the prototype compacted earthfill dam, which tends to slow down and change the breach process, whereas the applied equation applies to uniform, non-cohesive and non-compactable material. Also, due to the large and ramified reservoir surface, draw-down or retention effects are to be expected, reducing the breach discharge. Moreover, the application limits of the underlying Eq. (6.16) are not met for the present case, see below.

When applying Eq. (6.18), which was fitted to $k = 2.4$ for prototype data including the data presented in this case study, the predicted value is 52,570 m³/s, which is unsurprisingly very close to the reconstructed value of 53,000 m³/s.

However, the input parameters do not satisfy the limitations for Eq. (6.16) given as:

$$Q_M = 0.33 \sqrt{\frac{h_M}{w}} \left(\frac{h_M}{w}\right)^{13/10} \frac{w^{3/10}}{A_R^{2/20} \mu^{2/5}} = 0.189 < 0.68$$
The result therefore has to be considered with caution, and further model tests should be conducted to extend the application of the equations. The results of the applied Eq. (6.17) are on the safe side for determining $Q_M$, while the results of Eq. (6.18) appear better suited for prototype dikes with a higher erosion resistance, although the applicability of the equation needs to be controlled with more data. Further research should be conducted to determine the effect of reservoir shape on the breach process for non-cohesive sediment. The extension of the present test methods to cohesive material should be considered, especially with the advanced possibilities using simulated reservoir volumes.

6.9 Summary

In Chapter 6, non-dimensional results are deduced from the dimensional results of Chapters 4 and 5. The spatial dike breach tests are based on the hydraulic boundary condition (1) constant inflow discharge $Q_o$ and (2) $Q_o = 0$ along with different reservoir water surface areas $A_R$.

For tests with constant inflow discharge, the dimensional analysis of Schmocker (2011) is extended and the main governing parameters for the breach process are defined. The dimensionless time $T$ is then determined by observing the shift in the time-to peak and the impact of each governing parameter. It was thereby possible to scale the breach phases and to define ranges for $T$. The governing parameters for $T$ are compared to those of $T_{2D}$ for the plane dike breach tests of Schmocker (2011) and further authors, which differ considerably concerning the main governing parameters. The dimensionless maximum breach discharge and the maximum headwater level are then determined as a function of the main breach parameters. The dimensionless temporal evolution was determined for the maximum dike height along the central breach axis and the eroded sediment volume as a function of $T$ considering the effect of each parameter. The normalized evolution of the hourglass shape position, representing the limit between the submerged and non-submerged dike areas is also presented as a function of $T$. The normalized hourglass shapes and normalized submerged breach profiles are defined for different ranges of $T$. The constant inflow discharge results prove to be adequate in regard to plausibility and quality.

As to the tests with different reservoir water surface areas, the governing parameters for the breach process are redefined while also including tests with constant inflow discharge. The dimensionless peak breach discharge is then determined as a generalized function of the governing parameters resulting in a generalized peak breach discharge formula for
non-cohesive 3D dike breaches. Laboratory data of other institutions are then shown to compare well with the developed equation, which proves test reproducibility. The peak breach discharge equation is then discussed by comparison with reconstructed data of prototype dike breaches. The introduction of a fit parameter $k$ describing the higher erosion resistance of the prototype dam material leads to a good fit for $k = 2.4$. However, more prototype dam breach data need to be considered for the calibration of the fit parameter and to determine the application limits of the equation. Finally, two case studies highlight the applicability and limitations of this research.
7 Conclusions and outlook

7.1 General conclusions

The majority of research projects involving dike breach model tests have focused on few specific dike breach parameters, so that currently no general description of the breach process is available. The present work describes the spatial dike breach process due to overtopping considering the main breach parameters described in literature for homogeneous dikes built of non-cohesive sediment. For this purpose, the former VAW test setup was improved and a novel photogrammetric system considering refraction effects due to air/water was employed to capture the transient breach topography. These high-quality topographic data were coupled with flow hydrographs allowing for a sound analysis of the 45 tests conducted. The innovations of this study on spatial dike breaches due to overtopping for dikes of non-cohesive sediment include:

- Accurate topographic data of submerged spatial breaches
- Pump regulation to simulate large reservoirs
- Froude similitude verified for three non-distorted scale families
- Normalized transverse submerged breach profiles
- Dimensionless relations including the main parameters for
  - temporal evolution of breach process
  - peak breach discharge
  - maximum headwater level
  - maximum dike height at breach center
  - eroded sediment volume
- Dimensional peak breach discharge formulation

The accuracy of the photogrammetric system, test repeatability, symmetry for half-model tests, water surface topography, hydraulic properties and the effects of seepage and various other parameters were investigated by model tests and a systematic data analysis. The link between tests with constant inflow discharge and different reservoir water surface areas as hydraulic boundary condition was identified. The derived dimensionless relations and the peak breach discharge equation developed herein are discussed by comparison to literature and prototype data, while their applicability is demonstrated in two test cases. The test facility improvements and the conclusions for the test series are presented below.
7.2 Experimental setup and test procedure

The redesign of the experimental setup for the spatial dike breach tests is described in Chapter 3, which resulted in 45 tests and the acquisition of high-quality data.

As it is well known, air/water refraction issues limit the use and accuracy of photogrammetry to capture the highly transient topography of spatial dike breaches. To address this issue, a novel photogrammetric system was modified to properly eliminate refraction effects in the submerged breach areas, so that topographic breach data of adequate quality were obtained. The system accuracy was determined with 2 mm in non-problematic flow zones, and up to 10 mm in reaches with high transverse surface slopes.

A novel regulation method was applied to add a simulated reservoir to the physical reservoir in the laboratory channel by regulating the pump adequately. Both the reservoir size and shape were varied with this approach. This regulation method reduces the necessary laboratory space and simplifies the test procedure, enabling otherwise unattainable conditions in comparison to a standard regulation method.

7.3 Experimental results of model limitation tests

In Chapter 4, the effect of seepage is analyzed, followed by test series on test repeatability, symmetry of half-model tests, and non-distorted Froude scalability. The tests were conducted with constant inflow discharge as hydraulic boundary condition.

Seepage tests were conducted to optimize the drainage position and its maximum discharge capacity to ensure that the breach process is due to surface erosion only and is not affected by seepage.

Repeatability was demonstrated for 3D dike breach tests using dikes of height $200 \text{ mm} \leq w \leq 600 \text{ mm}$ by comparing the hydrographs and the breach topographies. Seepage-induced dike breaches lead to altered outflow hydrographs and should therefore be avoided.

The symmetry tests indicate that data extracted from half-model dike breach tests adequately describe the full-model breach. A slight stabilizing effect of the glass wall was observed, in agreement with literature observations.

Froude similitude was tested conducting three non-distorted scale families for dikes of initial height $w = 150, 300$ and $600 \text{ mm}$ and non-cohesive sediment of grain size $d = 0.43$ to $3.78 \text{ mm}$. The results indicate that apparent cohesion slightly influences the breach process in terms of breach topography evolution and outflow hydrographs. The scaled
erosion rate shows a satisfactory agreement, however. Interestingly, a comparison of scale test series with different grain sizes shows that the effect of the sediment diameter on the erosion rate is negligible for the tested range. This suggests that using the same sediment grain size in the model and the prototype could result in an improved modelling of the breach flank slopes. No limiting scale effect was observed for the tested ranges of initial dike heights and sediment grain sizes.

The results indicate that spatial dike breaches due to overtopping are scalable according to Froude similitude, although using a similar sediment grain size in the model and the prototype should be considered to reduce the effects of apparent cohesion on the breach flanks for the tested range of sediment grain size $d = 0.43$ to $3.78$ mm. These Froude-scaled test series add to the distorted Froude scale tests reported in literature.

### 7.4 Experimental results of parametric study

The effects of the parameters breach discharge, sediment grain size, initial breach width, dike cross-section, mobile bed, and reservoir water surface area on the spatial dike breach process are analyzed in Chapter 5.

Large inflow discharge leads to a high maximum headwater level, high peak breach discharge, high sediment erosion rates and a faster breach process in terms of time-to-peak.

Large sediment grain sizes lead to an initially high sediment erosion rate and an initially fast breach process, although these trends are not pronounced. The submerged breach channel shape is similar for all grain sizes, while small grain sizes lead to steep non-submerged side slopes. This indicates that for larger hydraulic loads (e.g. large reservoir water surface areas) the effect of grain size on the breach process is further reduced.

Large initial breach widths lead to low maximum headwater levels and low peak breach discharge, but the erosion rates and the breach process are comparable to those of small initial breach widths. If the initial breach width is too large though, the spatial dike breach features are similar to those of plane dike breaches with an accelerated breach process and a large initial sediment erosion rate.

Large dike cross-sections lead to high headwater levels, small peak breach discharges and a slow breach process in terms of time-to-peak, while an increase in the erosion rate is visible only for larger times due to the higher headwater levels.
A mobile bed below the dike leads to similar breach hydrographs and erosion rates as compared to tests on a fixed channel bed. Only after the breach channel has developed, scour occurs at the downstream dike toe, which leads to increased sediment transport and increased vertical erosion in the order of magnitude of the scour depth.

Large reservoir water surface areas lead to large peak breach discharge and a slow breach process in terms of time-to-peak. Reservoirs with different water surface areas lead to similarly shaped normalized breach discharge hydrographs. For tests with a constant reservoir level, the breach widening velocity is around 0.5 m/min (half-model test) or 1 m/min (full-model test) for dikes 0.3 m high and a sediment grain size of 1.75 mm.

The effect of the reservoir shape was determined for one triangular and two rectangular reservoir cross-sections with the same initial reservoir water surface area. While the peak breach discharges are nearly the same, deviations in the flow hydrograph increase with time. The good agreement of the peak breach discharge indicates that the initial reservoir water surface area describes the breach process better than the reservoir volume, as is also demonstrated in the normalized results described below.

In summary, the parameters breach discharge, dike cross-section and reservoir water surface area have a strong effect on the spatial dike breach process. The effect of the mobile bed only appears after the downstream dike toe is scoured, while only small effects were visible for the parameters sediment grain size and initial breach width. The findings on the effect of specific parameters on the breach process agree well with literature findings.

The photogrammetric system was also applied to measure the white-colored water surface topography of three tests with different inflow discharges. The water surface data were then combined with the sediment surface data of previous tests, assuming test repeatability. The two data sets match well, indicating good test repeatability. When combining the transverse topography profiles, the angle of repose in the submerged flow cross-section were found to be lower as compared to that in non-submerged breach portions, the limit being indicated by a kink in the sediment profile.

### 7.5 Normalized results

The previous dimensional data are combined in Chapter 5 to describe the breach process in non-dimensional form. For this purpose, a dimensional analysis was conducted to determine the governing breach parameters prior to determining the following dimensionless relations:
Dimensionless time $T$ to describe the temporal evolution of the breach process for tests with constant inflow discharge as hydraulic boundary condition. The inflow discharge, represented by the critical flow depth, was found to accelerate the breach process, while the parameters decelerating the process include the dike cross-section and the reservoir water surface area, and to a lesser degree the apparent cohesion and the initial dike height.

*Dimensionless relations* that describe the peak breach discharge, the maximum headwater level, the maximum dike height at breach center, and the eroded sediment volume versus the governing parameters. The evolution of the dimensionless plane breach shapes, the transverse submerged breach profiles and the submerged breach width is presented as a function of the dimensionless time.

Tests with the hydraulic boundary conditions (1) constant inflow discharge and (2) reservoir water surface areas for no inflow discharge were found to be related to the maximum headwater level. On the driving side of the governing parameters, therefore, inflow discharge, represented by the critical flow depth, is replaced by the maximum headwater level. This link leads to a *generalized dimensionless relation* for peak breach discharge including both boundary conditions mentioned above. Transforming this dimensionless expression, the dimensional *peak breach discharge equation* for spatial dike breaches due to overtopping of dikes of non-cohesive sediment is

$$Q_M = Q_{b,d} = 0.0156 g^{1/2} h_M^{13/4} A_R^{6/8} A_D^{-1}$$

with peak breach discharge $Q_M$ [m³/s], design breach discharge $Q_{b,d}$ [m³/s] (which is equal to inflow discharge when drainage discharge is negligible), maximum headwater level $h_M$ [m], reservoir water surface area $A_R$ [m²], submerged specific gravity $g'$ [m/s²] and the dike cross-sectional area $A_D$ [m²].

The dimensionless relation and the peak breach discharge equation are discussed by comparison with literature research results, showing excellent agreement for an extended test range of sediment grain sizes from $d = 0.21$ to $3.78$ mm. Furthermore, an excellent agreement was found for reservoir volume tests including triangular and rectangular reservoir shapes, highlighting that the initial reservoir water surface area describes the breach process better than the commonly used reservoir volume for rectangular and triangular reservoir shapes and dikes of non-cohesive sediment.

The peak breach discharge equation is then discussed by comparison with reconstructed values of prototype dike breaches. A systematic shift was observed in the data distribution, which is attributed to the higher erosion resistance of the dam material as compared
to non-cohesive sediment used in the model dikes. Therefore, a fit parameter $k$ describing the higher erosion resistance was introduced in the above peak breach discharge equation, which can then be written as

$$Q_{m} - Q_{o,d} = 0.0156 h_{w}^{1/2} g^{1/4} A_{D}^{5/8} \left( \frac{k}{A_{D}} \right)$$

For $k = 2.4$, a good fit was obtained for the reconstructed values of six prototype dam breaches of dams built of earth, sand, gravel and rockfill material for the overtopping and the piping scenario. More prototype dam breach data need to be considered for the calibration of this equation and to determine its application limits.

The above peak breach discharge equation is a novel equation developed from dike breach model data including the relevant governing parameters found in literature. The equation shows the importance of the dike cross-section, which is often absent in empirical breach models developed from prototype breach data captured after the incident. The equation shows strong similarities with the equation developed by Froehlich (2016a).

### 7.6 Outlook

In the present work, spatial dike breaches due to overtopping of homogeneous dikes built of non-cohesive sediment have been investigated. Failure due to seepage was avoided by placing an adequate drainage. Combining the dike topography and the hydrographs, the breach process including the most relevant breach parameters is described. Relevant parameters not investigated include the sediment density, the platform length downstream of the dike and the backwater effect.

Parameters investigated but whose range should be extended include sediment grain sizes, mobile beds below the dike and – applying the novel regulation system – reservoirs of different shapes and larger water surface areas up to a constant reservoir level as in large rivers whose water surface elevation is hardly affected by breach outflow. To study the fast progressing side erosion process, of which the relevance increases for large reservoirs, a wider channel setup with larger pumping capacity is required.

The peak breach discharge equation developed from the model tests can be applied to re-evaluate empirical or parametric models developed by regression analysis from prototype data, e.g. by Froehlich (2016a), specifically concerning the reservoir water surface area $A_{R}$ used instead of the reservoir volume $V_{R}$. 
Finally, the dike breach topography and the breach outflow hydrographs were determined in test series. These data are valuable to calibrate numerical models, extend findings numerically or validate sediment transport formulas, as e.g. by Boes et al. (2015), Vonwiller et al. (2015) or Volz et al. (2016).
### Notation

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
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</thead>
<tbody>
<tr>
<td>( a )</td>
<td>Reservoir specific constant according to Kühne (1978)</td>
<td>([\text{m}^3\cdot\text{c}])</td>
</tr>
<tr>
<td>( A )</td>
<td>Cross-sectional flow area</td>
<td>([\text{m}^2])</td>
</tr>
<tr>
<td>( A_D )</td>
<td>Cross-sectional dike area</td>
<td>([\text{m}^2])</td>
</tr>
<tr>
<td>( A_R )</td>
<td>Reservoir water surface area at breach start</td>
<td>([\text{m}^2])</td>
</tr>
<tr>
<td>( b )</td>
<td>Channel width or dike width</td>
<td>([\text{m}])</td>
</tr>
<tr>
<td>( b_b )</td>
<td>Transverse submerged breach channel width</td>
<td>([\text{m}])</td>
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<td>( b_p )</td>
<td>Pilot channel width at top</td>
<td>([\text{m}])</td>
</tr>
<tr>
<td>( b_{p,u} )</td>
<td>Pilot channel width at bottom</td>
<td>([\text{m}])</td>
</tr>
<tr>
<td>( c )</td>
<td>Reservoir shape factor according to Kühne (1978)</td>
<td>([-])</td>
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<td>Sediment diameter of uniform material ( \sigma_R &lt; 1.2 )</td>
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<tr>
<td>( d_i )</td>
<td>Sediment diameter of grain size distribution corresponding to ( i% ) finer</td>
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<tr>
<td>( D_\star )</td>
<td>Particle parameter as defined by van Rijn (1984), ( D_\star = d_{50}/d_R )</td>
<td>([\text{m}])</td>
</tr>
<tr>
<td>( F )</td>
<td>Froude number</td>
<td>([-])</td>
</tr>
<tr>
<td>( g )</td>
<td>Acceleration of gravity</td>
<td>([\text{m/s}^2])</td>
</tr>
<tr>
<td>( g' )</td>
<td>Submerged specific gravity</td>
<td>([\text{m/s}^2])</td>
</tr>
<tr>
<td>( h )</td>
<td>Water surface level</td>
<td>([\text{m}])</td>
</tr>
<tr>
<td>( h_b )</td>
<td>Flow depth</td>
<td>([\text{m}])</td>
</tr>
<tr>
<td>( h_c )</td>
<td>Critical flow depth</td>
<td>([\text{m}])</td>
</tr>
<tr>
<td>( h_{cap} )</td>
<td>Capillary rise</td>
<td>([\text{m}])</td>
</tr>
<tr>
<td>( h_{c,W} )</td>
<td>Critical flow depth for triangular weir of ( S_p = 2 )</td>
<td>([\text{m}])</td>
</tr>
<tr>
<td>( h_o )</td>
<td>Headwater surface level above channel floor</td>
<td>([\text{m}])</td>
</tr>
<tr>
<td>( h_{0,0} )</td>
<td>Headwater surface level above channel floor at breach start</td>
<td>([\text{m}])</td>
</tr>
<tr>
<td>( h_{US} )</td>
<td>Distance of ultrasonic probe from water surface</td>
<td>([\text{m}])</td>
</tr>
<tr>
<td>Notation</td>
<td>Description</td>
<td>Unit</td>
</tr>
<tr>
<td>----------</td>
<td>------------------------------------------------------------------------------</td>
<td>-------</td>
</tr>
<tr>
<td>$h_y$</td>
<td>Transverse profile of water surface level above channel floor</td>
<td>[m]</td>
</tr>
<tr>
<td>$I_S$</td>
<td>Channel bed slope</td>
<td>[m]</td>
</tr>
<tr>
<td>$k$</td>
<td>Fit parameter for prototypes in peak breach discharge equation</td>
<td>[-]</td>
</tr>
<tr>
<td>$L$</td>
<td>Length scale</td>
<td>[m]</td>
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<tr>
<td>$L_{dr}$</td>
<td>Length of drainage</td>
<td>[m]</td>
</tr>
<tr>
<td>$L_D$</td>
<td>Dike length</td>
<td>[m]</td>
</tr>
<tr>
<td>$L_f$</td>
<td>Downstream platform length</td>
<td>[m]</td>
</tr>
<tr>
<td>$L_K$</td>
<td>Crest length</td>
<td>[m]</td>
</tr>
<tr>
<td>$L_m$</td>
<td>Scaling length in model</td>
<td>[m]</td>
</tr>
<tr>
<td>$L_p$</td>
<td>Scaling length in prototype</td>
<td>[m]</td>
</tr>
<tr>
<td>$n$</td>
<td>Parameter in powerfunction approximation</td>
<td>[-]</td>
</tr>
<tr>
<td>$p$</td>
<td>Parameter in powerfunction approximation</td>
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</tr>
<tr>
<td>$Q_c$</td>
<td>Critical discharge</td>
<td>[m³/s]</td>
</tr>
<tr>
<td>$Q_b$</td>
<td>Breach discharge</td>
<td>[m³/s]</td>
</tr>
<tr>
<td>$Q_d$</td>
<td>Downstream river outflow discharge (fluvial dike breach tests)</td>
<td>[m³/s]</td>
</tr>
<tr>
<td>$Q_{dr}$</td>
<td>Drainage discharge</td>
<td>[m³/s]</td>
</tr>
<tr>
<td>$Q_M$</td>
<td>Maximum breach discharge in full-model test</td>
<td>[m³/s]</td>
</tr>
<tr>
<td>$Q_o$</td>
<td>Inflow discharge</td>
<td>[m³/s]</td>
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<tr>
<td>$Q_R$</td>
<td>Reservoir discharge</td>
<td>[m³/s]</td>
</tr>
<tr>
<td>$Q_s$</td>
<td>Seepage outflow discharge</td>
<td>[m³/s]</td>
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<tr>
<td>$R$</td>
<td>Reynolds number</td>
<td>[-]</td>
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<tr>
<td>$R_h$</td>
<td>Hydrafic radius</td>
<td>[m]</td>
</tr>
<tr>
<td>$R^2$</td>
<td>Coefficient of determination</td>
<td>[-]</td>
</tr>
<tr>
<td>$S_d$</td>
<td>Downstream dike slope V:H = 1:$S_d$</td>
<td>[-]</td>
</tr>
<tr>
<td>$S_o$</td>
<td>Upstream dike slope V:H = 1:$S_o$</td>
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<tr>
<td>$S_p$</td>
<td>Transverse pilot channel slope V:H = 1:$S_p$</td>
<td>[-]</td>
</tr>
<tr>
<td>$S_y$</td>
<td>Transverse water level slope $S_y = \Delta h_y / \Delta y$</td>
<td>[-]</td>
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</tbody>
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Notation

\( t \) Time \[ \text{s} \]
\( t_M \) Time when peak breach discharge \( Q_M \) occurs (time-to-peak) \[ \text{s} \]
\( t_p \) Time when headwater level reaches base of pilot channel \[ \text{s} \]
\( t_0 \) Time origin at breach start \[ \text{s} \]
\( t^* \) Dimensionless time by Pickert et al. (2011) [-]
\( T \) Dimensionless time [-]
\( T_{2D} \) Dimensionless time by Schmocker (2011) for 2D dike breaches [-]
\( u_{*cr} \) Critical bed-shear velocity according to Shields (1936) \[ \text{m/s} \]
\( v \) Flow velocity \[ \text{m/s} \]
\( v_s \) Settling velocity of a sand or silt particle \[ \text{m/s} \]
\( V_E \) Eroded sediment volume \[ \text{m}^3 \]
\( V_R \) Reservoir water volume \[ \text{m}^3 \]
\( V_W \) Cumulated breach discharge \[ \text{m}^3 \]
\( V_0 \) Available sediment volume influencing erosion process \[ \text{m}^3 \]
\( w \) Dike height \[ \text{m} \]
\( w_{	ext{mobile}} \) Maximum scour for mobile sediment bed below dike \[ \text{m} \]
\( w_p \) Pilot channel notch height \[ \text{m} \]
\( w_{p,0} \) Pilot channel sediment height at glass wall \[ \text{m} \]
\( W \) Weber number [-]
\( x \) Streamwise coordinate \[ \text{m} \]
\( x_D \) Distance of upstream dike toe from upstream channel wall \[ \text{m} \]
\( x_{dr} \) Distance of drainage from upstream dike toe \[ \text{m} \]
\( X \) Dimensionless streamwise coordinate [-]
\( y \) Transverse coordinate \[ \text{m} \]
\( y_{hg} \) Transverse coordinate of hourglass shape relative to \( y_{min} \) \[ \text{m} \]
\( y_{hg,d} = y_{max} - y_{min} \) \[ \text{m} \]
\( y_{max} \) Largest transverse water surface width of hourglass shape \[ \text{m} \]
<table>
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<th>Description</th>
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<tr>
<td>$y_{\text{min}}$</td>
<td>Smallest transverse water surface width of hourglass shape [m]</td>
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<tr>
<td>$Y$</td>
<td>Dimensionless transverse coordinate [-]</td>
</tr>
<tr>
<td>$z$</td>
<td>Vertical coordinate [m]</td>
</tr>
<tr>
<td>$z_b$</td>
<td>Vertical coordinate of sediment surface [m]</td>
</tr>
<tr>
<td>$z_M$</td>
<td>Maximum dike height along central streamwise breach profile ($y = 0$) [m]</td>
</tr>
<tr>
<td>$z_{Me}$</td>
<td>Maximum equilibrium dike height [m]</td>
</tr>
<tr>
<td>$Z$</td>
<td>Dimensionless vertical coordinate [-]</td>
</tr>
<tr>
<td>$Z_M$</td>
<td>Dimensionless maximum dike height with $Z_M = z_M/w$ [-]</td>
</tr>
<tr>
<td>$Z_{Me}$</td>
<td>Dimensionless maximum equilibrium dike height [-]</td>
</tr>
</tbody>
</table>

**Greek symbols**

<table>
<thead>
<tr>
<th>Greek symbol</th>
<th>Description</th>
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<tbody>
<tr>
<td>$\alpha$</td>
<td>Angle [°]</td>
</tr>
<tr>
<td>$\varepsilon$</td>
<td>Porosity [-]</td>
</tr>
<tr>
<td>$\theta$</td>
<td>Angle of repose [°]</td>
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<tr>
<td>$\theta_{cr}$</td>
<td>Critical mobility parameter [-]</td>
</tr>
<tr>
<td>$\theta_s$</td>
<td>Submerged angle of repose [°]</td>
</tr>
<tr>
<td>$\lambda$</td>
<td>Scaling factor [-]</td>
</tr>
<tr>
<td>$\lambda_F$</td>
<td>Froude scaling factor [-]</td>
</tr>
<tr>
<td>$\mu$</td>
<td>Dike shape parameter with $\mu = (3/7)[(1/2)(S_o+S_d)+L_k/w]$ [-]</td>
</tr>
<tr>
<td>$\nu$</td>
<td>Kinematic viscosity [m²/s]</td>
</tr>
<tr>
<td>$\sigma$</td>
<td>Surface tension of fluid [kg/s²]</td>
</tr>
<tr>
<td>$\sigma_d$</td>
<td>Standard deviation from bundle adjustment method (Luhmann 2013) [mm]</td>
</tr>
<tr>
<td>$\sigma_g$</td>
<td>Geometric standard deviation of grain size distribution [-]</td>
</tr>
<tr>
<td>$\rho$</td>
<td>Fluid density [kg/m³]</td>
</tr>
<tr>
<td>$\rho_s$</td>
<td>Solid particle density [kg/m³]</td>
</tr>
<tr>
<td>$\tau_b$</td>
<td>Bed shear stress [N/m²]</td>
</tr>
<tr>
<td>$\phi$</td>
<td>Natural angle of repose [°]</td>
</tr>
<tr>
<td>$\phi_s$</td>
<td>Submerged angle of repose [°]</td>
</tr>
</tbody>
</table>
Subscripts

\( b \quad \text{Breach, bed} \)
\( d \quad \text{Design, downstream} \)
\( dr \quad \text{Drainage} \)
\( e \quad \text{Energy} \)
\( M \quad \text{Maximum} \)
\( o \quad \text{Approach flow, head, upstream} \)
\( p \quad \text{Pilot channel} \)
\( ref \quad \text{Reference} \)
\( R \quad \text{Reservoir} \)
\( y \quad \text{Along the transverse coordinate} \)
References


VAW (2011). Breschenbildung an Dämmen kleiner Stauanlagen im Kanton Zürich – Numerische Simulation zur Beurteilung des Breschenabflusses bei progressiven Dammbrüchen. VAW-Bericht 4289, ETH Zurich, Zürich, Switzerland (in German, unpublished).


