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Control of Wastewater Collector Tunnels

Implementation of real-time control for the central collector tunnel in the city of Graz

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Abstract

Key words:

Combined sewer overflow, real time control, model predictive control, integrated modelling, effective flushing of sewers

Nowadays the protection of our ecosystem and the responsible use of our natural resources are of high importance to us. Therefore, the conservation or the re-establishment of a good chemical and ecological state for all surface water bodies is a high priority for every local sewer operator. In a city like Graz that mainly uses a combined sewer system where rainwater and municipal sewage come together in one pipe, an untreated overflow of that polluted water into a receiving water body should be prevented by all means.

This thesis deals with the management of the newly constructed collector tunnel in Graz and the possible control actions that can be taken to raise the efficiency of the system using an integrated model that combines a rainfall model, a conceptual model for the sewer system of the city and a detailed model of the collector tunnel.

The work focuses on two main points concerning the central storage tunnel. The first one is to maximize the cleaning efficiency by flushing the sewers to reduce sediments in the tunnel. The newly developed flushing schemes show clear advantages over the currently used strategy. The second point is developing a global real-time control strategy to minimize combined sewer overflows during storm events. Two control schemes were developed and compared with each other, a rule based real-time control system and a model predictive control system. Both proved to be significantly better in handling storm events than the currently used local control system.

Kurzfassung

Stichwörter:

Mischwasserüberlauf, Echtzeitkontrolle, modellprädiktive Kontrolle, integrierte Modellierung, effektive Schwallspülung

Auch oder gerade in der heutigen Zeit sind der Umweltschutz und der sorgsame Umgang mit den natürlichen Ressourcen von größter Wichtigkeit. Deswegen hat die Erhaltung bzw. Wiederherstellung eines guten chemischen und ökologischen Zustandes sämtlicher Oberflächengewässer eine hohe Priorität für Abwassernetzbetreiber. In einer Stadt wie Graz, die zum Großteil eine Mischwasserkanalisation umgesetzt hat, in der Regenwasser und Schmutzwasser in einem gemeinsamen Kanal abgeleitet werden, sollte ein unbehandelter Mischwasserüberlauf in angeschlossene Wasserläufe auf jeden Fall vermieden werden.

Die vorliegende Masterarbeit handelt von der Betreibung des neuen zentralen Speicherkanals in Graz und dem Einsatz von Kontrollregelungen zur Effizienzsteigerung des kompletten Systems. Zur Umsetzung dieser beiden Ziele wird ein integriertes Modell angewendet, das sich aus einem Regenmodell, einem konzeptionellen Modell für das Abwassersystem von Graz und einem hochaufgelöstem Modell für den zentralen Speicherkanal zusammensetzt.

Die Arbeit zielt auf zwei Hauptpunkte betreffend den Speicherkanal ab. Im ersten Punkt wird versucht die maximale Reinigungsleistung mithilfe von Schwallspülungen zu erreichen, um die Sedimentablagerung im Tunnel zu reduzieren. Die neu entwickelten Spülprogramme zeigen eine klare Verbesserung gegenüber dem momentan verwendeten Spülszenario. Im zweiten Punkt der Arbeit wird versucht eine globale Echtzeitkontrollstrategie zu entwickeln, um Mischwasserüberläufe während starken Regenereignissen zu minimieren. Es wurden zwei Kontrollansätze umgesetzt und miteinander verglichen, ein regelbasiertes Echtzeitkontrollsystem und eine modellprädiktive Kontrollstrategie. Beide Varianten haben signifikante Verbesserungen gegenüber dem momentan verwendeten lokal gesteuerten System gezeigt.

Table of Contents

List of Abbreviations

BOD	Biochemical oxygen demand
COD	Chemical oxygen demand
CSO	Combined sewer overflow
CSS	Combined sewer system
e.g.	For example
MPC	Model predictive control
OEWAV	Austrian water and waste management association
PC	Personal Computer
RTC	Real time control
SSS	Separate sewer system
TOC	Total organic carbon
TSS	Total suspended solids
WWTP	Wastewater treatment plant
ZSK	Central collector tunnel

1 Introduction

This chapter is separated into four main parts. At first there is an introduction of the topic followed by the motivation behind this thesis. After that the goals of this work are listed to give a general overview of which topics will be covered later on. Finally some of the fundamentals will be explained to create a solid base of knowledge to start from.

Graz, Austria applies a combined sewer system (CSS), where domestic and industrial wastewater is discharged together with stormwater in one combined system. Some areas, mostly situated in the outer regions of the city, are covered by a separate sewer system, where domestic / industrial wastewater and stormwater are drained in two separated systems. Separate systems are used in rural areas as well where stormwater is treated on sight which means that stormwater pipes are not necessary. The CSS is, in terms of overall volume transported to the wastewater treatment plant (WWTP), of much more importance. At the WWTP the wastewater is first treated and then discharged into a nearby recipient (in this case the river Mur). However, WWTPs are not built to handle such large amounts of water – their hydraulic capacity is mostly limited to two times the maximum dry weather flow. So in the case of a significant storm event, combined sewer overflows (CSOs) occur along the system. In Graz, 37 overflow structures discharge directly into the river Mur. That means that untreated but diluted wastewater enters the receiving water body, which could lead to interferences that can be a problem for the environment. In the year 2000 the EU published the water framework directive (WFD; EC, 2000) that requires EU member countries to follow higher standards to protect their open water bodies by continuously monitoring them and, in case of a disturbance of their chemical state, to pinpoint the source. For a system like the one in Graz the most likely cause of such a problem usually is CSO discharge.

If CSO discharge is the problem, the simplest way to improve the situation would be to extend the available storage volume in the sewer system, but for reasons of high costs, operational problems, lack of space and difficulties with land ownership situations, this is not always possible or the best solution. So in order to reach this goal, another option is to use the existing facilities in the system to their full potential by controlling the occurring wastewater flow within the system. Various strategies exist to realize this objective, all of which are combined within the method of real time control (RTC).

1.1 Motivation

The future challenge for the drainage system of Graz is to reduce discharge to the receiving water and increase the treated volume at the WWTP. The main goal is to fulfill the requirements of the new state of the art for CSOs in Austria, the OEWAV Guideline 19 (OEWAV, 2007b). In the past decade, the achievement of this goal was not realistic because of the high investment costs for the city and the local sewer operator. This situation changed however in 2009, when a new hydropower plant in the area south of the city was planned. It was discovered that some of the 37 CSO outlets were going to be flooded because of the rising backwater level in the river, and therefore a solution had to be found. In a partnership between the city of Graz and the company Energie Steiermark (operator of the hydropower plant), a central collector tunnel (later called ZSK) was planned to collect the discharged volume from the affected CSO structures. The ZSK was planned to be implemented in two stages. The first section with a length of 3.2 km was built alongside the already existing hydropower plant and is already in operation. The second stage with a length of 5.0 km will be built alongside another hydropower plant further up the river in the next couple of years and will be connected to the first section of the ZSK.

A significant rise of the total storage volume of around 91 000 m^3 (22 000 m^3 in stage one) and state of the art controllable weirs and orifices provide the opportunity to effectively install a control system to pursue the objective of minimizing the overall discharged pollution load from the drainage system to the receiving water.

1.2 Goals

Primarily this master's thesis focuses on creating efficient strategies to control the newly built first section of the ZSK storage tunnel with a length of about 3.2 km in order to maximize its retention capacity in case of a storm event. On the other hand, the discharged volume from the system to the receiving water should be minimized. The developed strategies are then tested with an integrated and calibrated model of the whole sewer system of the city and of the ZSK including control elements. As model input, artificial rainfall events with different return periods typical for the region are used. A side topic of this thesis is to investigate possible flushing strategies for the ZSK after a storage event to minimize sediment deposits along the tunnel.

The work is divided into the following sub-goals:

- Update, adjustment and calibration of the existing sewer model
- Definition of the optimization parameters to evaluate emptying and flushing strategies for the ZSK
- Development, simulation and evaluation of different emptying and flushing scenarios in the ZSK
- Definition of representative storm events to test the hydraulic behavior of the ZSK
- Development of efficient RTC strategies for storage, emptying and flushing of the ZSK
- Definition of validation parameters for RTC strategies
- Comparison of the developed RTC strategies with the non-controlled system
- Development of a demonstrative way to present the results

1.3 Fundamentals

This subchapter is separated into eight parts to introduce some basic knowledge of the topic.

First, urban drainage is explained, focusing on the problems of combined sewer overflows. After that, options to reduce overflow volume are introduced. With this foundation, the next part explains real time control and the various possible strategies that can be implemented, along with some case studies and the state of science in this field of research. This is followed by an introduction of sedimentation and deposition in storage tunnels. Finally the basics of modelling in sewer systems are explained.

1.3.1 Urban drainage

Urban drainage is the backbone of a healthy and modern city. The applied systems have been introduced and have evolved over the last centuries from simply dumping wastewater onto the streets into a complex system of underground sewers that collect and guide wastewater to treatment facilities almost unnoticeable to the public, where it is treated and then released into a receiving water body. The great challenge is to keep these systems working efficiently and to keep them optimized with affordable measures. In drainage systems, two main hydraulic flow conditions exist. During dry weather periods, dry weather flow occurs with a typical diurnal pattern. It contains domestic and industrial wastewater and some sewer infiltration water. This wastewater is highly polluted and usually undergoes a mechanical and biological treatment in a WWTP. During a storm event, additional stormwater accumulates in the form of runoff from the surface to the connected drainage system. This water is usually less polluted due to residues on sealed surfaces. In practice, two main approaches in the drainage of wastewater exist – combined sewer system (CSS) and separate sewer system (SSS) (for further information refer to Butler and Davies (2000)).

1.3.1.1 Combined sewer system versus separate sewer system

In a SSS stormwater is strictly separated from other wastewater sources, which results in two different treatment cycles. The advantage of this system is that the WWTP cannot be overloaded and the stormwater can be handled separately. Also, the dry weather flow can be channeled into much smaller sewer pipes, whereas the stormwater pipes are significantly bigger.

In a CSS the dry weather flow and the stormwater runoff are guided in the same sewer pipes. This results in larger profiles for sewers and also in the necessity of combined sewer overflow structures, because during a heavy rain storm "*the system can discharge from overflow structures into a recipient such as a stream, river, lake or sea, if the capacity of the system is exceeded"* (Mollerup *et al.*, 2012). However separate sewer overflow structures exist in large SSSs too, which can be overflowed during heavy storm events. Furthermore according to the current state of the art in Austria WWTP are designed to treat double the amount of the maximum dry weather flow of their connected area as described in the OEWAV Guideline 19 OEWAV, 2009). So in a heavy storm event, untreated wastewater enters the receiving water body.

There are also combinations of separated and combined sewer systems in place, but in Graz the combined system (70% of the overall sewer system) makes up for most of the targeted problems in this thesis (read chapter 2.1.1.1 for more information about the sewer system of Graz).

Figure 1-1 summarizes the differences of the drainage process of both systems. The upper three levels are the same in both approaches. The differences appear when the water is collected in the sewer pipes. Whereas in the SSS the storm water is handled and ultimately discharged into the receiving water body separately from the sanitary sewer flow, in the CSS both flows are mixed and channeled together towards the WWTP. However the overload of the combined sewer pipes is discharged preferably into a storm tank or another storage facility. Else the discharge directly flows into the receiving water body.

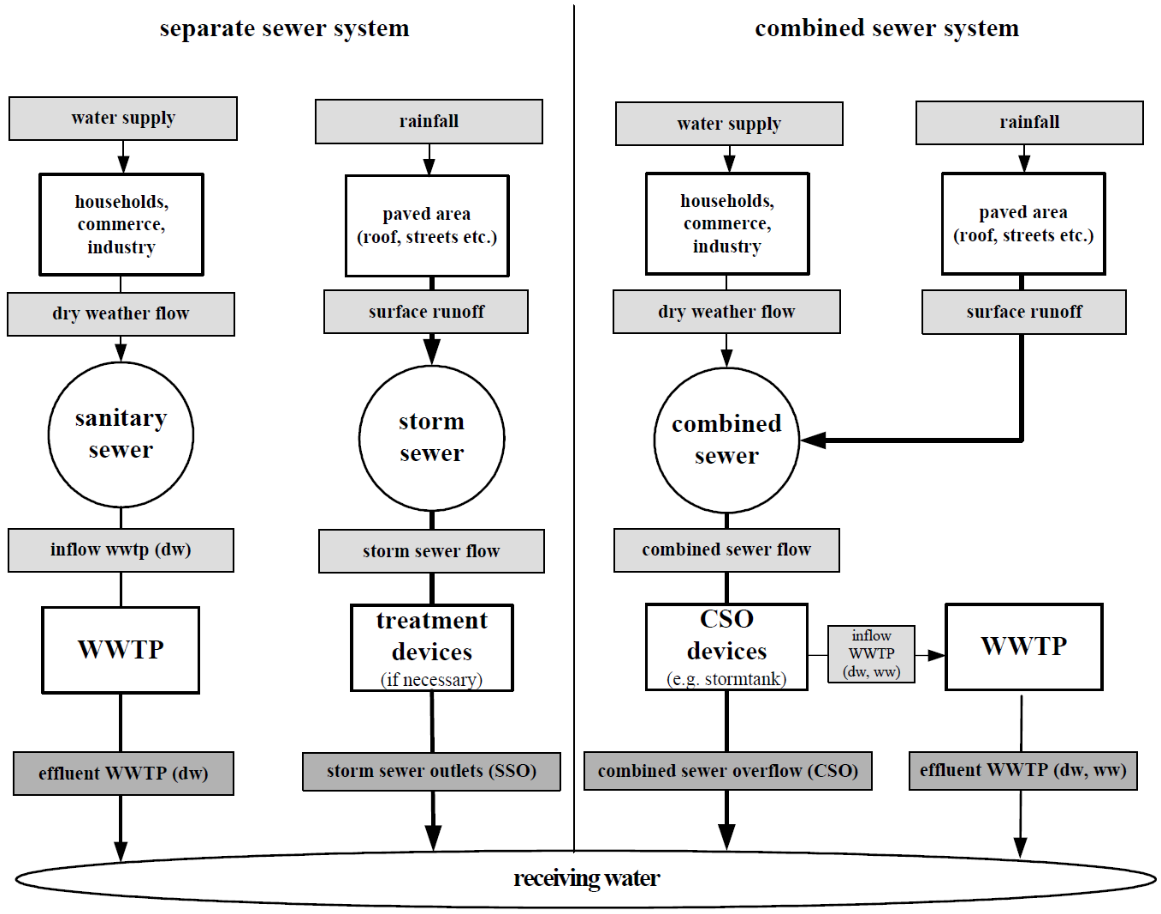


Figure 1-1: Comparison between separate and combined sewer system (Welker, 2008)

1.3.1.2 Combined sewer overflows (CSOs)

CSOs are unavoidable events in a CSS. That is because the pipe diameters used in the system are limited due to the fact that the space needed and the high costs of pipes that could transport the combined wastewater flow towards the WWTP are often economically not justifiable enough to implement them. So the flow that overloads the system needs to be discharged with a CSO structure. In the past, that was accepted as a necessary evil. Although “*since the EU water framework directive came into force in 2000, wastewater systems (sewer system and wastewater treatment plants) in Europe have been put under pressure to reduce the number of combined sewer overflows (CSOs) from the system to protect the aquatic environment”* (Mollerup *et al.*, 2012) (EC, 2000). Even though in case of the Mur the good chemical and ecological status of the river is not threatened by the CSOs because it is a big enough receiving stream, the public opinion on the matter shifted and therefore any improvement concerning the CSO volume is preferable. The overflow structures can be set up in different ways. There can be a simple fixed weir crest (more information in US-EPA (1999)) that controls the ongoing flow towards the WWTP, or different forms of overflow basins (more information in ATV-DVWK (2001);

DWA (2013a)) for various purposes, starting with inducing sedimentation to partly treat the overflowing water, up to completely storing the overflow event until there is extra capacity in the WWTP.

1.3.1.3 Legislative context in Austria

The OEWAV published the OEWAV Guideline 19 document (OEWAV, 2007b) as the Austrian standard for designing CSOs that, though not legally binding, states that the total system efficiency of CSS is determined over a minimum required efficiency of the CSOs in the system. Instead of describing the state of the art with constructional restrictions, the guideline introduces the objective to transport a specific ratio of the pollutants contained in the runoff to the WWTP. This guideline currently is not state of the art and therefore not legally binding. However there are various ways to proclaim it as the new state of the art. If it is commonly used throughout different communities, it would happen automatically. Furthermore it can be named state of the art in any newly issued permit concerning the matter. And of course it can be stated as state of the art in national law. If one of these events were to happen, the document that was formerly advisory would become obligatory. An English description of the OEWAV Guideline 19 can be found in Kleidorfer and Rauch (2011).

1.3.2 Options to meet the legislative requirements

To reach the goals of the OEWAV Guideline 19 document, different strategies can be applied.

1.3.2.1 Development and extension of stormwater management

The first and probably the most logical way to reach the goals of the guideline document would be to prevent the stormwater to enter the sewer system in the first place. In cities, pervious areas are scarce. Streets and buildings seal up the surface and result in much higher peaks and a greater volume of runoff during a storm event because water is transported quickly over artificial channels such as streets or stormwater collectors. So exchanging impervious areas with pervious areas again or infiltrating the occurring rainwater with an artificial facility like absorbing wells into the groundwater would help lower that runoff peak and therefore decrease the stress in the recipients. Currently there are programs and guidelines that encourage these steps for new constructions. However the older buildings and facilities still make up for a significant rise in runoff. Also, runoff from streets, copper roofs, etc. has to be treated because of their contamination with heavy metals, oil or other micro pollutants. More information on this topic can be found in Butler and Davies (2000); OEWAV (2003); DWA (2007).

1.3.2.2 Increase of storage volume

Another way to deal with CSOs would be to build additional storage basins and collector tunnels on the surface or below ground to collect potential storm events and drain the overflow water for treatment to the WWTP when there is extra capacity available. Although this option would get rid of the problem, the high costs, the immense use of space by these facilities and the possibility of odor nuisance for the public are significant disadvantages of this solution. More details on this topic can be found in DWA (2013b).

1.3.2.3 Increase of WWTP capacity

Another option to reduce the CSOs of a city would be to resize the WWTP so that it is capable of dealing with bigger amounts of runoff. This method is usually combined with creating more storage volume in the sewer system or replacing small sewer pipes with bigger ones to transport the additional wastewater to the WWTP. The downside of this approach is that during dry weather flow, big parts of the plant are unused. Also, the cost and needed space of such projects are significant. Another way to increase the work load of a WWTP is with dynamic adjustment of the maximum treatment capacity according to the current sludge situation in the second clarifier of the plant, meaning that from a normal capacity of for example double the maximum dry weather flow, the capacity can be risen for short periods of time to handle a bigger inflow (Seggelke *et al.*, 2013). This strategy can also reduce the overflow peak during storm events. Nevertheless, at first additional sewer pipe capacity needs to be implemented.

1.3.2.4 Implementation of control strategies in urban drainage systems

If a city's sewer system operator applies control strategies to use the maximum possible storage capacity available, the amount of needed storage volume or additional treatment capacity decreases. Alone it might not be enough to just control the available storage facilities, but redirection of runoff peaks to areas that still have additional storage capacity available can reduce the problem substantially.

Generally two different forms of control exist, which are described in the following two sections. The description refers especially to the application in drainage systems.

Local control strategy

In this form a regulator like an orifice or a moveable weir is controlled by a locally applied trigger reacting to e.g. the filling status of a storage basin or the flow in a pipe. What is important here is that the control action taken is dependent only on a local state. The depth in another facility nearby is not taken into account. More in Schilling (1990).

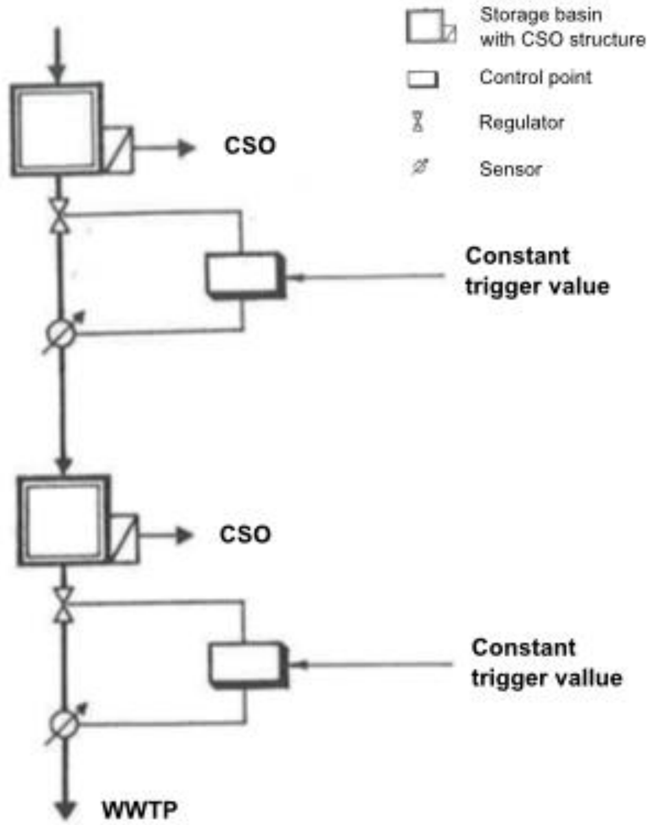


Figure 1-2: Local control scheme in urban drainage systems (Schilling, 1990 modified)

Global control strategy

Global control, instead of local control, takes the bigger picture into consideration. All the sensors in a system send their information to one central location, where it is computed. With this information, control decisions can be made, considering the state of the whole system. This means for example, if a storage basin is almost full and another one downstream still has additional storage capacity available, a regulator from the first basin can be triggered to empty part of the basin into the other storage facility before an overflow occurs.

However this behavior can only be achieved when the necessary equipment is installed. A communication system is needed to transfer the information to the central location and to trigger regulators. A fast computer system has to be installed to handle the information arriving from the different sensor locations, to process and to monitor them so that the staff can make an expert decision on how to set the system to react to the current conditions (or in more advanced systems let the system decide how to react to the situation with the staff just monitoring the current situation). The more sophisticated a system gets and the less decisions are taken from trained staff, the more backup strategies and redundancies need to be implemented to ensure that the system does not suffer a complete failure. That means that in case of a multiple failure the system still needs to at least fulfill its primary goal of draining its connected area. In addition strategies have to be figured out to deal with different situations. The central location has to be capable of processing the incoming data and presenting it, so that in a manually controlled system, trained staff can overview the data and make the necessary decisions. More in Schilling (1990).

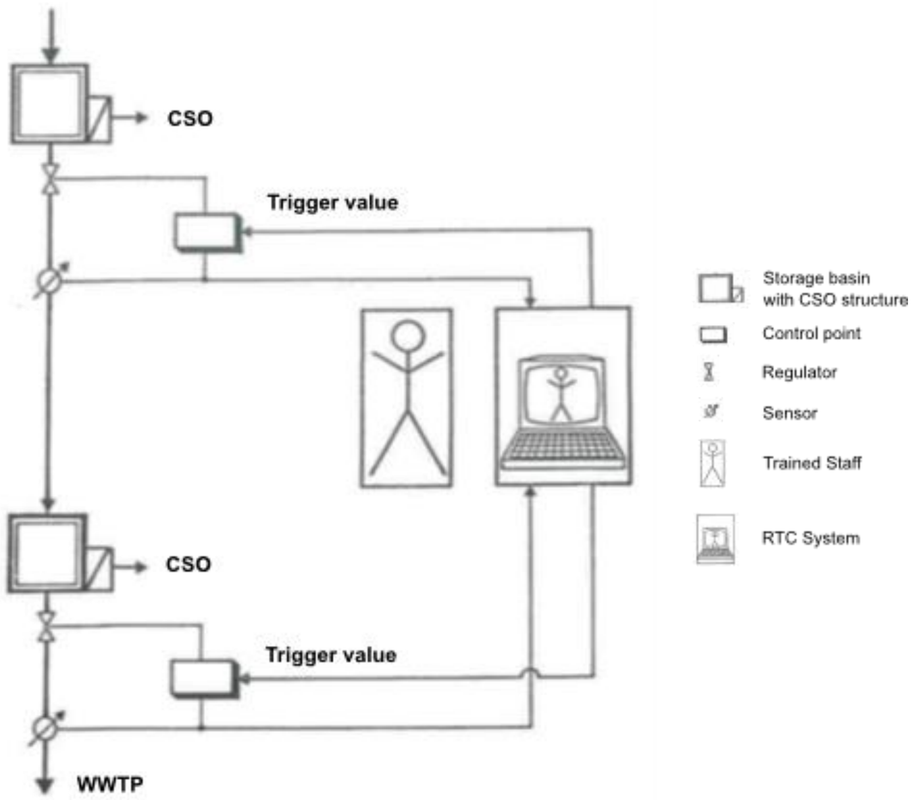


Figure 1-3: Global control scheme in urban drainage systems (Schilling, 1990 modified)

1.3.3 Real time control (RTC)

Real time control (RTC) is the application of global control in urban drainage systems. It presents the possibility to automatically control the drainage system. The staff still has the possibility to interact with the system or even intercept control decisions. Nevertheless, RTC should be capable of handling different scenarios on its own. Furthermore, the worst-case scenario of a well-developed RTC system should never be worse than the behavior of the system prior to the installation of RTC, which means a non-controlled system.

To find a strategy and evaluate it, trial and error can be an option, although in times of modern computers and sophisticated modelling tools, this is generally not necessary anymore. So nowadays a detailed model of the area to be controlled is set up and calibrated. Then different test scenarios are worked out and simulated without controlling the system to set a reference point with which any future strategies can be compared. With this model, different control strategies can be tested against each other to find the optimal one to apply in the actual system.

More information on RTC can be found in Schilling (1990); Hou and Ricker (1992); Schilling (1996); Colas *et al.* (2004); Schütze *et al.* (2004); Campisano *et al.* (2013); Beeneken *et al.* (2013).

RTC can be applied in different ways, which are described in the following subchapters. Figure 1-4 gives an overview for these subchapters.

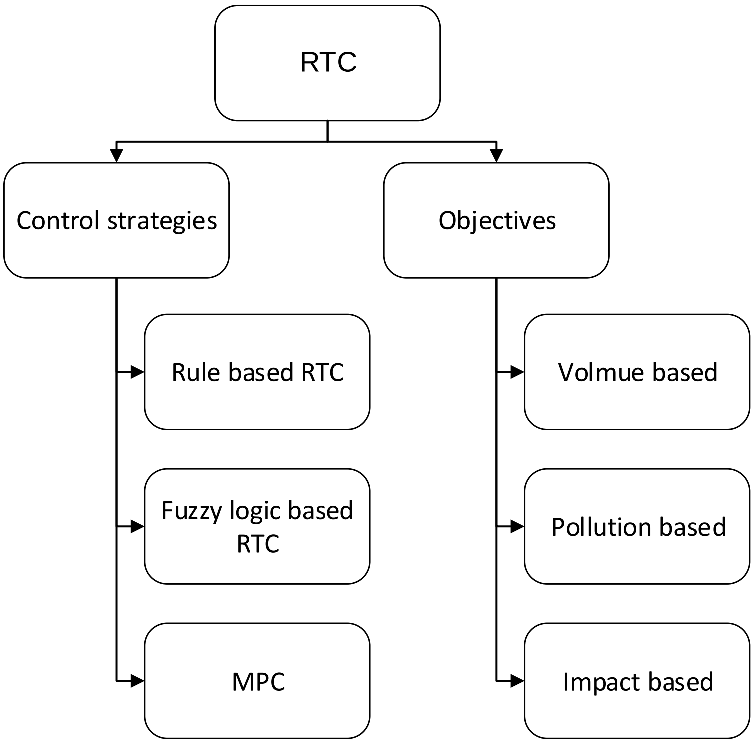


Figure 1-4: Overview for RTC control and optimization strategies

1.3.3.1 Rule based RTC

The control strategy of rule based RTC works with predefined rules to control the drainage system. See Figure 1-5 for the construction of such rules. These rules take various states of the system into account and trigger regulators like orifices, moveable weirs or pumps. The priorities or the order of how the rules are executed are predefined to guarantee that locations with a higher priority get preferential treatment. A weak point of this strategy is that it acts rather statically. So it might work perfectly fine during a normal storm event, but for a bigger storm, the set points (e.g. which water level or which flow triggers a regulator) of the rules might not apply and the RTC system could become obsolete. Furthermore an RTC strategy in a complex system can end in a lot of rules, which makes it difficult to modify the strategy or even to understand it for somebody who is not familiar with it. Further information can be found in Borsanyi *et al.* (2008).

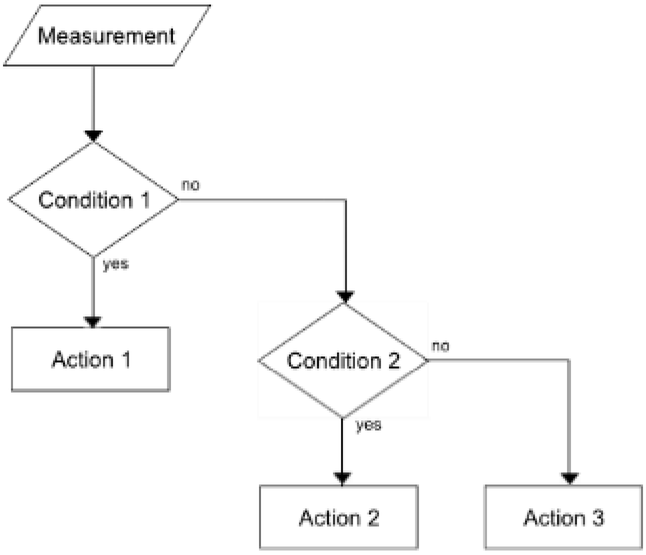
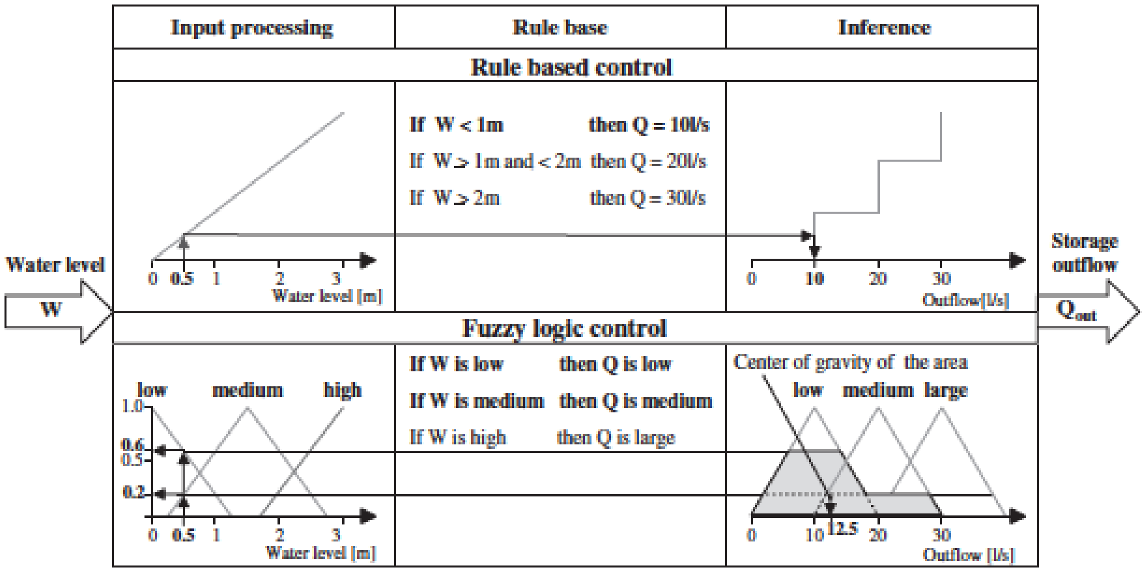


Figure 1-5: Rule structure example for rule based RTC

1.3.3.2 Fuzzy Logic based RTC

Fuzzy logic based RTC works essentially the same way as rule based control. The difference though, is the formulation of the rules. This scheme works with functions instead of exact set points. So a complicated set of rules can be put together into one function. Because these functions can be a bit complicated to set up for an outsider, fuzzy logic based RTC sometimes is rejected as a solution. In Figure 1-6 a traditional rule based scenario is compared to a fuzzy logic set up. The rule based RTC uses an input value that falls in a predefined range and triggers a static output value. In between these ranges the system does not react to changes. For example in Figure 1-6 a water level of 0.5 m triggers a storage outflow of 10 l/s. In a fuzzy logic based RTC strategy the input value is interpreted with a function. The interpretation is then used to assign an output value matching the current input with another function. In Figure 1-6 the input value of a water level of 0.5 m results in an interpretation of the water level as low and medium, each with their distinct ratios. These ratios are used to produce an area with the output function. The actual output value is generated by calculating the center of gravity of this area and results in a storage outflow of 12.5 l/s. Such a strategy is often used in WWTPs so far, although due to the higher effort involved in setting up the rules in a sewer system, they are often left aside for a simpler rule based scenario. However the more complex a system is the more complicated a rule based RTC system gets, whereas a fuzzy logic based system can present itself more clearly. More information can be found in Hou and Ricker (1992); Klepiszewski and Schmitt (2002).



With: W … water level (m); Q … flow (l/s); Q_{out} … outflow (l/s)

Figure 1-6: Rule based RTC strategy compared to fuzzy logic based RTC strategy (Klepiszewski & Schmitt, 2002)

1.3.3.3 Model predictive Control (MPC)

Model predictive control (MPC) takes sewer system control one step further. Here, an algorithm takes the current state of the system and information about the surrounding area as input and uses that information to find the best possible actions to be taken to work towards an objective function e.g. the lowest possible CSO volume. This strategy was, until recently, hardly ever implemented because it needs a vast amount of computing capacity to be able to find a result quickly. With today's processors and the use of multithreading and parallelization of computer programs, these problems moved to the background, but there are still limitations to utilizing MPC in a system. This means that trying to find the perfect solution for multiple parameters often results in an exponential rise of computing time in comparison to simple problems. Also, when it comes to looking into different parameters such as for example total suspended solids (TSS), total organic carbon (TOC), sedimentation and so on, the complexity of the problem gets too complicated for a simple linear program. This is where genetic algorithms and other methods to find the best possible solution to a problem come into play.

More detailed information on MPC can be found in Pleau *et al.* (2005); Ocampo-Martinez (2010); Fradet *et al.* (2011).

1.3.4 Objectives of RTC

There are different objectives of RTC that will be introduced in the next three subchapters.

1.3.4.1 Volume based RTC

Volume based RTC aims to minimize the overflow volume of CSOs. This approach suggests that the pollution produced from runoff directly correlates with the amount of overflow. It is the easiest approach to implement because the necessary sensors consist of flow measurement and water level sensors, which are less sensitive to wastewater than for example sensors that detect pollution. More information can be found in Fradet *et al.* (2011); Seggelke *et al.* (2013).

1.3.4.2 Pollution based RTC

Most RTC systems aim to minimize CSO volume because it seems to be the most obvious way to reduce stress for the receiving water body. Nevertheless, new approaches work in the direction of directly measuring the pollution of the current runoff to determine if an overflow is of high risk for the recipient. The problem with this approach is that sensors detecting pollution often have problems like obstruction or even total failure because of debris. So if such a system is set up, redundancy has to be a clear focus to ensure its stability, which can be quite cost intensive. More information can be found in Hoppe *et al.* (2011).

1.3.4.3 Impact based RTC

Impact based RTC takes the state of the recipient into consideration, so as to decide if it can take an overflow in its current condition or not. The direct measurement of the receiving water body however is not suggestible because of the large range that needs to be measured to produce the necessary result and therefore the long timespan that would be needed to actually get a measurement result. That is because ammoniac for example is measured directly after the CSO structure, whereas BOD (biochemical oxygen demand) has to be measured long after the effluent enters the recipient. Therefore calibrated river quality models are used to determine the effects of the effluent on the receiving water body. More information can be found in Langeveld *et al.* (2013).

1.3.5 State of RTC in scientific literature

The development of RTC goes in different directions. The following two subchapters should give a glimpse of the current state of science.

1.3.5.1 Literature on fundamental research on RTC

Currently there are a lot of ongoing case studies to improve RTC. More reliable and cheaper measurement systems for real time pollution measurement allow more options in the direction of pollution based RTC. Faster processing speed of computers makes it possible to find better solutions for more complex problems and also lays the base for more detailed and more accurate models which makes a case study in the early planning phase much more attractive (Schütze *et al.*, 2004; Campisano *et al.*, 2013).

1.3.5.2 Integrated RTC

Another approach that will be of higher importance is integrated control. Here not only the sewer system is taken into consideration, but the whole urban drainage system, even with rain prediction and the state of the receiving water body. The challenges in this direction are currently the interfaces between the different models that are used to model the various parts of an urban drainage system. That is because the models use different parameters and are not always compatible with each other (read chapter 2.1.4.1 for more details). There are already projects working in that direction (Erbe, 2004; Seggelke *et al.*, 2008; Seggelke *et al.*, 2013), but there is still a lot of work to be done to come to a satisfying and applicable solution.

1.3.6 State of RTC in practical implementation

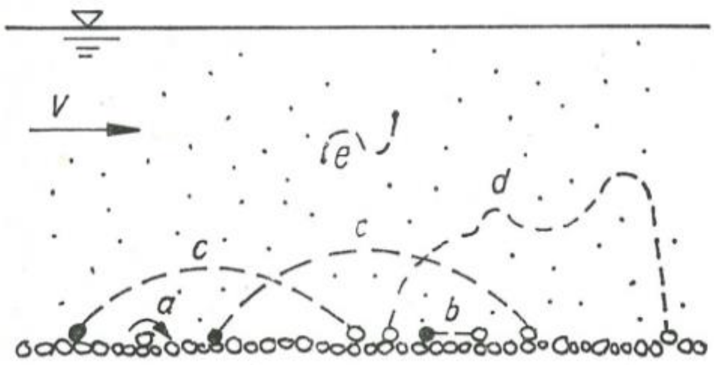
There are some implementations that more or less act as poster projects in RTC. One of them is the system that is in place in Quebec, Canada. They use a global optimal control strategy (a form of MPC) to control their drainage system. The MPC system is capable of reacting to most of the scenarios and is learning with every event by using the information gained during the occurred storm. Even a total failure of the WWTP in a dry weather period could be controlled by the system because of the integration of the WWTP capacity into the system. Also, in the case of Quebec it was shown that significant cost savings could be achieved by installing an RTC system instead of increasing storage volume. More information about the RTC system in Quebec can be found in Pleau *et al.* (2005); Pleau *et al.* (2001); Fradet *et al.* (2011).

Another example of a future orientated project is the system in place in Wuppertal, Germany. There, due to limited space and a lack of options, a rule based RTC strategy with the objective to minimize the pollution of the receiving river, the Wupper, was set up to measure the current state of the stormwater runoff to determine when to channel it to the WWTP and when it is safe to channel it to the receiving river. With that system in place, the stress on the ecosystem of the Wupper was largely reduced. More information in Hoppe *et al.* (2011).

There are many more systems in place in other locations, but as the focus of this chapter is merely to give an introduction to the topic, they won't all be listed. Some more interesting examples can be found in the papers Seggelke *et al.* (2013) and Langeveld *et al.* (2013).

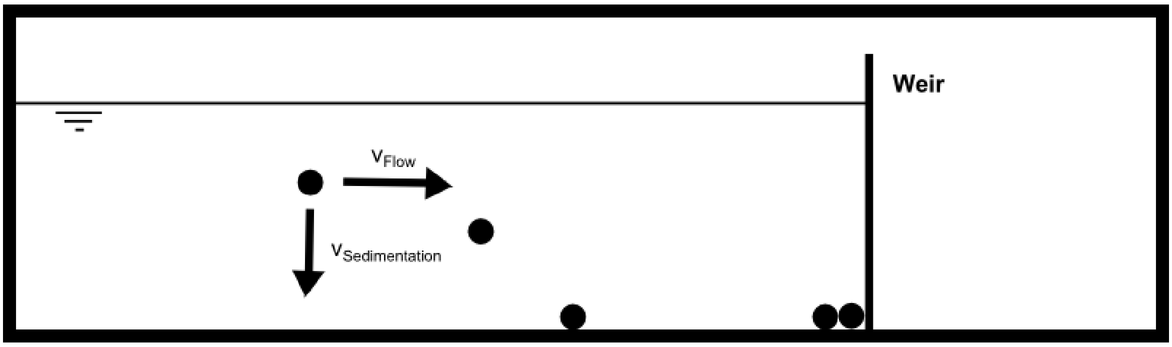
1.3.7 Sedimentation and deposition in storage tunnels

One of the common problems in any sewer network is the formation of deposits of sediments. Whereas in normal sewer pipes the constant flow prevents intense deposits (see different kinds of particle movement in Figure 1-7) and a mechanical cleaning in defined intervals is enough, significant problems arise in collector tunnels and storage basins where polluted wastewater containing a high amount of sediments is stored over a longer period of time. Especially right in front of weirs, these deposits can accumulate intensely (see Figure 1-8). That is because these areas are the ones where the sedimentation speed is higher than the flow speed most of the time (because the flow velocity basically slows to zero). So the particles float down to the floor and form deposits. Figure 1-8 shows the principal of sedimentation on one of the weirs in the tunnel as an example. The wastewater enters the tunnel completely mixed, then the sediments slowly sink with a vertical velocity component, due to the low horizontal speed component in the tunnel produced by the backwater effect of the weir, and form sediments in the area in front of the weir.



With: a … rolling; b … sliding; c … saltation (jump and roll); d … saltation under influence of cross fluctuations; e … suspension (whereas: movements a to d … rubble; e … suspended particles)

Figure 1-7: Variation of movement of sedimentation particles (Bollrich, 1989)



With: v_{Flow} … horizontal velocity component; $v_{Sedimentation}$ … vertical velocity component

Figure 1-8: Velocity components of sedimentation process in front of a weir

Problems of deposits in drainage systems:

Consequences of deposits in sewers can be seen in Geib *et al.*, 2007:

- Decrease of the sewer cross section
- Less retention volume of the sewer network during storm events and therefore a higher occurrence of CSO events
- Higher pollution due to CSO events during storms
- Higher operating costs for cleaning measures
- Higher pipe roughness
- Higher risk of biogenic corrosion due to acid sulfur
- Odor due to the formation of hydrogen sulfide
- Health risks for the operating staff

Possible measures to reduce deposits in sewer systems:

To counteract deposits in collectors there are various options like flushing or mechanical cleaning, which can be automated or manually executed. In the case of the ZSK, facilities were installed to use flush waves to clean the collector after a storage event. There is also the possibility of a manual mechanical cleaning with a cleaning vehicle that can be driven through the tunnel, although this measure should not be used very often due to the high operation costs of such an endeavor. Read chapter 5.4.3 in Golger (2014) for more details on cost comparison between mechanical cleaning and the use of low pressure flushing waves. More information about possible measures against deposits in sewer pipes, storage basins and collector tunnels can be found in Dettmar (2005).

Connection between bottom shear stress and wave velocity

To remove formed deposits from the sewer invert after a storage event in the ZSK, the parameter bottom shear stress describes the ability of the tunnel to clean itself with the use of flushing waves. Because the bottom shear stress cannot be measured directly, this work uses the flow velocity as a surrogate parameter to describe it. Equation 1-1 is used to transform the average flow velocity into the bottom shear stress.

$$\tau_0 = \rho * \frac{\lambda}{8} * v_m^2$$

Equation 1-1

With τ_0: shear stress (N/m^2), ρ: density of the wastewater (kg/m^3), λ: resistance coefficient of the friction in pipes (-) and v_m: average flow speed (m/s)

The resistance coefficient λ is calculated in Equation 1-2.

$$\lambda = \frac{2g * d_{hy}}{k_{st}^2 * \left(\frac{d_{hy}}{4}\right)^{\frac{4}{3}}}$$

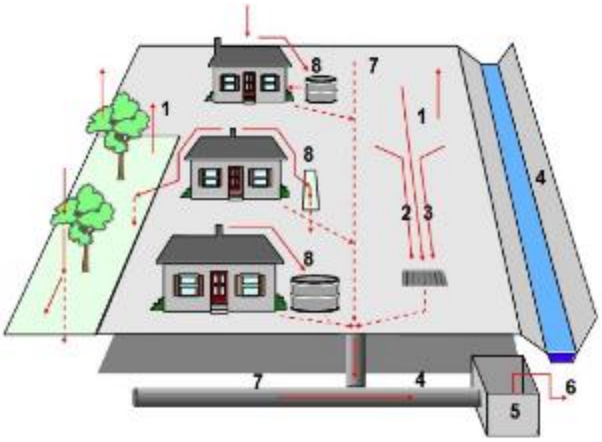
Equation 1-2

With λ: resistance coefficient of the friction in pipes (-), g: gravitational acceleration (m/s^2), d_{hy}: hydraulic diameter (m) and k_{st}: coefficient of roughness (m$^{1/3}$/s)

More details of this process can be found in Golger (2014).

1.3.8 Basics of modeling in sewer systems

Generally rainfall-runoff modeling is separated into two different parts: Processes happening on the surface (evaporation, generation of runoff, concentration of runoff and more specific processes) and processes happening in the sewer system (transformation of runoff, separation of runoff, storage of runoff, overflows, calculation of dry weather runoff and concentration of pollution). The processes on the surface are not of any relevance for this thesis, so they will not be explained further. However, the processes happening in the sewer system will be explained later. Figure 1-9 shows an overview of the processes treated in rainfall-runoff modeling.



1. Evaporation
2. Generation of runoff
3. Concentration of runoff
4. Transformation of runoff
5. Separation and storage of runoff
6. Overflows
7. Calculation of the dry weather runoff and the concentration of the pollution
8. More specific processes (e.g. infiltration)

Figure 1-9: Concepts for rainfall-runoff modeling (Muschalla, 2008 modified)

The procedures happening in the sewer system are summarized in the transformation of runoff. Hereby the results from the surface runoff calculations represent the inflows to the various parts of the system. The boundary conditions for these processes are attributes like geometry and runoff specific characteristics of the sewer system including special constructions like weirs or orifices. In general, two effects describe a runoff wave: translation (propagation delay) and retention (damping). Together they form the runoff transformation. Figure 1-10 shows the principle of a wave deforming over time.

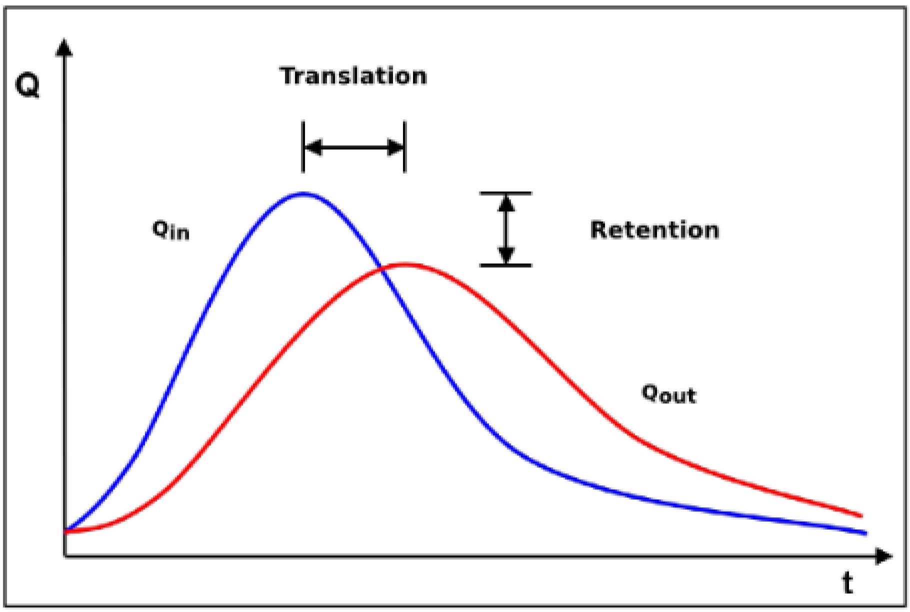


Figure 1-10: Runoff transformation (Muschalla, 2008 modified)

These effects need to be described for the implementation of a model. Therefore two different model approaches are used: conceptual and hydrodynamic models.

In hydrodynamic transport models, a detailed description of the runoff processes is performed with consideration of every physical process involved. Whereas in conceptual transport models, the runoff processes are described by an empiric transfer function. Table 1-1 shows the main advantages and disadvantages of the two approaches.

Table 1-1: Differences between conceptual and hydrodynamic models (Klawitter & Ostrowski, 2006 modified)

Conceptual model	Hydrodynamic model
Short computing times (big dt)	Long computing times (small dt)
Little data management	Large data management
Long-term simulations	Barely suitable for real-time predictions
Easy to use	Needs experienced user
No consideration of backwater effects	Considers backwater effects
Only mass balance at nodes	Flow calculation dependent on time and location
Smoothing of single processes	Separation of flow processes between surface and sewer
Less congruency between nature and model	More congruency between nature and model

1.3.8.1 Closer look at 1D hydrodynamic modeling of transportation processes in sewer systems

The base of 1D hydrodynamic models is the De-Saint-Venant-equation-system. The models can be used with the assumption that any velocity components across the general flow direction are negligible. So a channel is interpreted as a pipe with a flat fluid surface whose profile can only change gradually. Figure 1-11 shows the heads relevant for the energy equation that is the base of the De-Saint-Venant-equation-system.

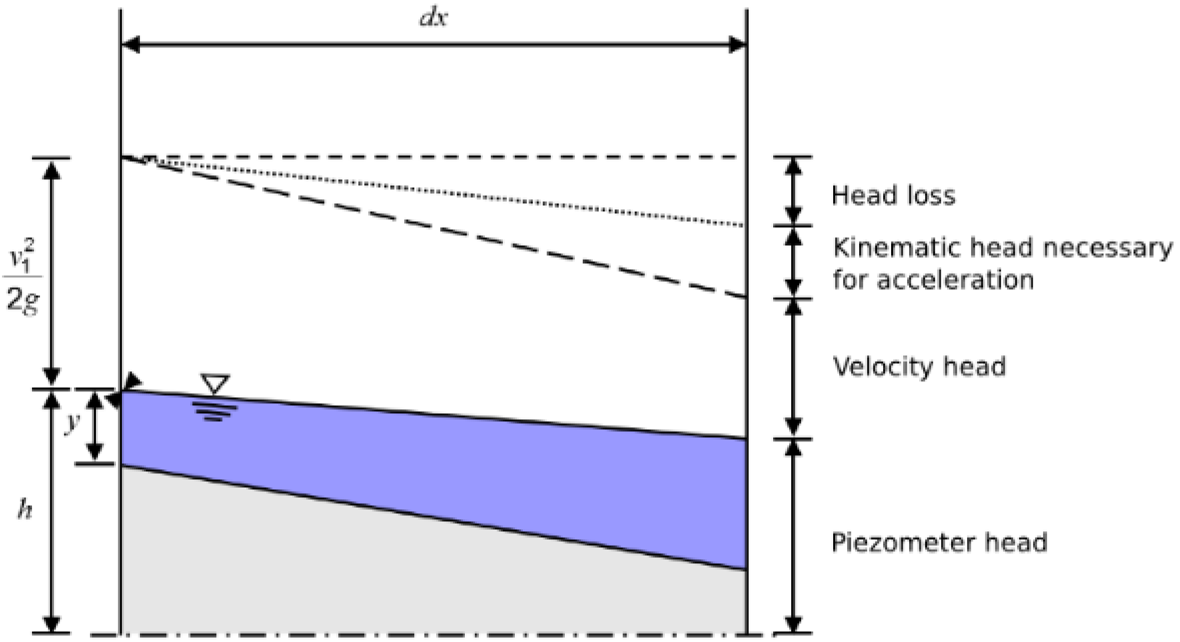


Figure 1-11: Schematic of the components of energy equation (Maniak, 2005 modified)

Figure 1-12 shows the actual equation system separated into the different parts that a model can focus on.

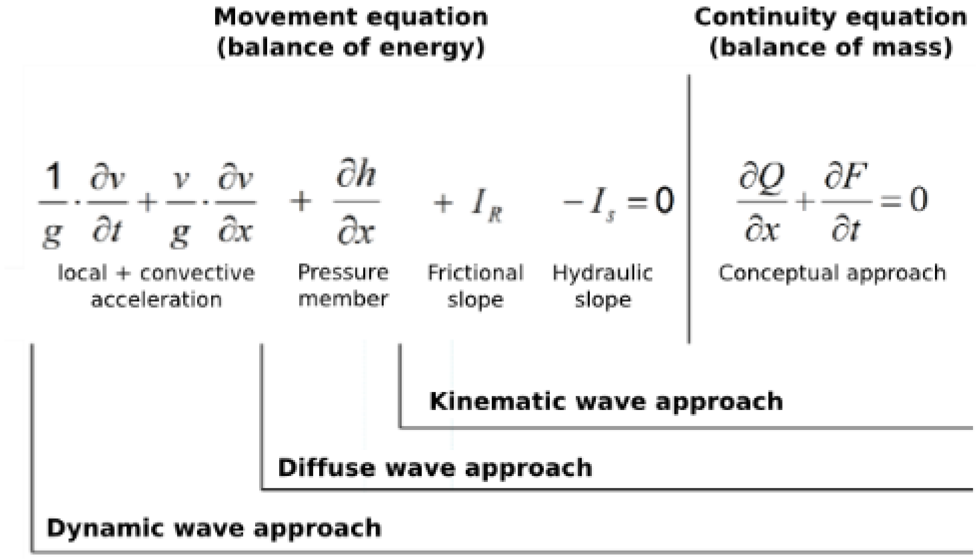


Figure 1-12: De-Saint-Venant-equation-system (Dyck & Peschke, 1995 modified)

Kinematic wave approach

Besides the continuity equation, the frictional and the hydraulic slope are taken into account. It should only be used for steep systems with no occurrence of backwater effects.

Diffuse wave approach

This approach also includes the pressure member in its calculation, which means that backwater effects are considered. However, effects of inertia are not taken into account.

Dynamic wave approach

This approach includes the whole equation system. Therefore all physical processes can be simulated.

Limitations of 1D hydrodynamic models

- The energy loss of overflowing water, when it hits the invert is not factored in the calculation. That means that turbulence is not simulated in these models.
- Surface shear is not considered in the calculation.
- Horizontal velocity components are ignored.
- Other physical phases like air and water are not included in the calculation of current, for example to start a simulation with a dry surface.

Characteristics of 1D hydrodynamic models

- Integration in horizontal direction (flow direction)
- The De-Saint-Venant-equation-system is the basis for these models. They describe the unsteady flows with average depth and width variables.
- Effects of turbulence, dissipation, shear and secondary flows are only considered in the energy line slope I_E.
- No use of turbulence models.

2 Methodology

This chapter consists of three parts. First the general Methodology is shown in an overview. After that the materials and the model setup are introduced, which is followed by the description of the RTC modeling scenarios.

The methodology of this work is separated into seven steps:

- Represent the current state of the ZSK
    - To establish the current state of the ZSK and the possibilities that it offers, an accurate model has to be built to be able to monitor the behavior of different strategies under different boundary conditions. The main adjustments were to correct the geometry (invert elevation, profile) and the roughness of the ZSK. In addition an average loss coefficient had to be applied to match the reality and the attributes of the weirs had to be modified to simulate their actual behavior. It is also necessary to find out about the currently installed measurement equipment and the actions that can be taken from a central point of operation to take control measures over the ZSK.
- Find the best emptying and flushing scenario after a storm event
    - To find the best flushing scenario, first it is necessary to reproduce the strategy of Holding Graz currently used to get a basic scenario to compare the new scenarios to. With the given possibilities to control the ZSK, the best requirements for an optimal cleaning effect for the tunnel will be attempted.
- Analyze and discuss the results of the found flushing scenarios
    - After generating different strategies with different initial statuses, the results are plotted and compared to see the advantages and disadvantages of each approach.
- Generate demonstrative test scenarios
    - To model different test scenarios of the ZSK, it is necessary to create demonstrative weather scenarios strong enough to see how the whole system reacts under peak conditions. If the available data of past events does not produce such conditions, an artificial storm event will be created.
- Find the best control strategy to handle a big variety of storm events
    - To find control strategies, primarily it is necessary to set up a reference scenario of the currently used strategy to use as a comparison. After that, different approaches are used to gain the best possible results to fulfill the requirements of a feasible approach.

- Analyze and discuss the results of the RTC strategies
    - After setting up the found strategies to control the ZSK, they will be run within different conditions and their results are plotted and analyzed to show the benefits and drawbacks of each approach.
- Set up a control strategy for storm events and the control actions used to empty and flush the ZSK afterwards
    - With the found solutions for both problems, a control strategy is set up that could be applied by Holding Graz to maximize the effectiveness of the infrastructure of the ZSK.

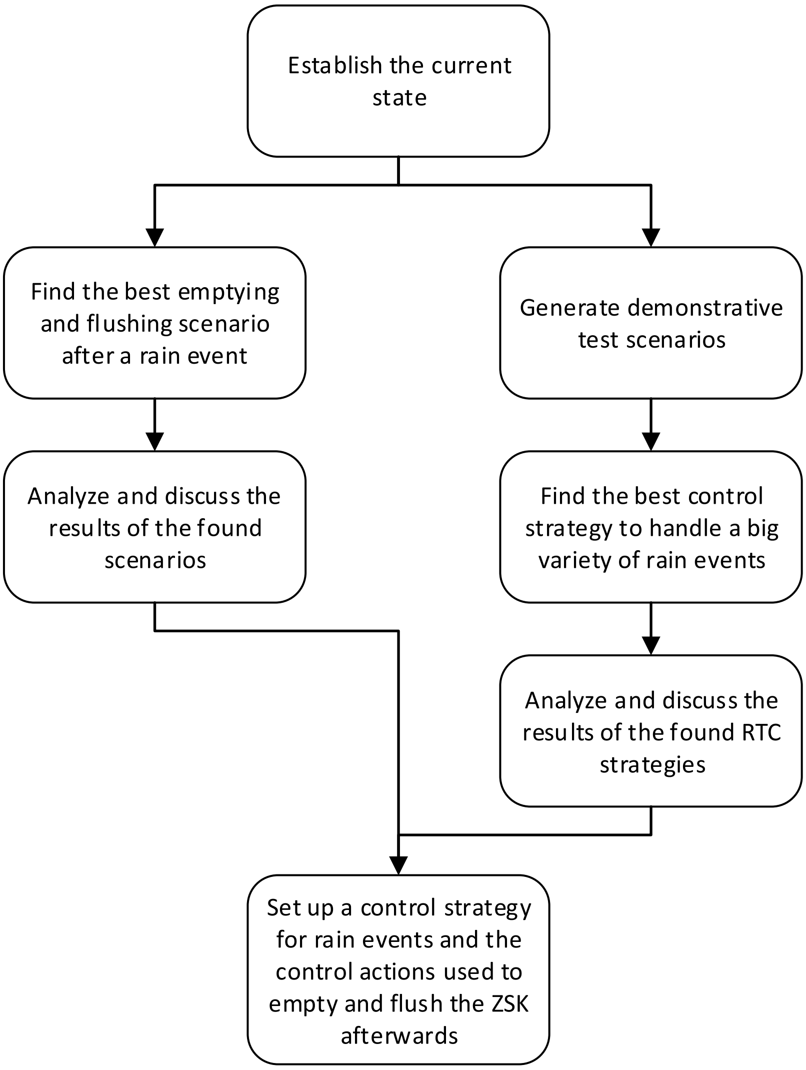


Figure 2-1: Methodology to find a control strategy for storm events and the actions taken after it

2.1 Materials and model setup

The following chapter introduces the simulation models used in this work and shows how the interface problem of the integrated model was handled. It also deals with the different challenges that needed to be overcome to set up a model exact enough to produce suitable data output. Finally, the resulting model with its calibration results is presented.

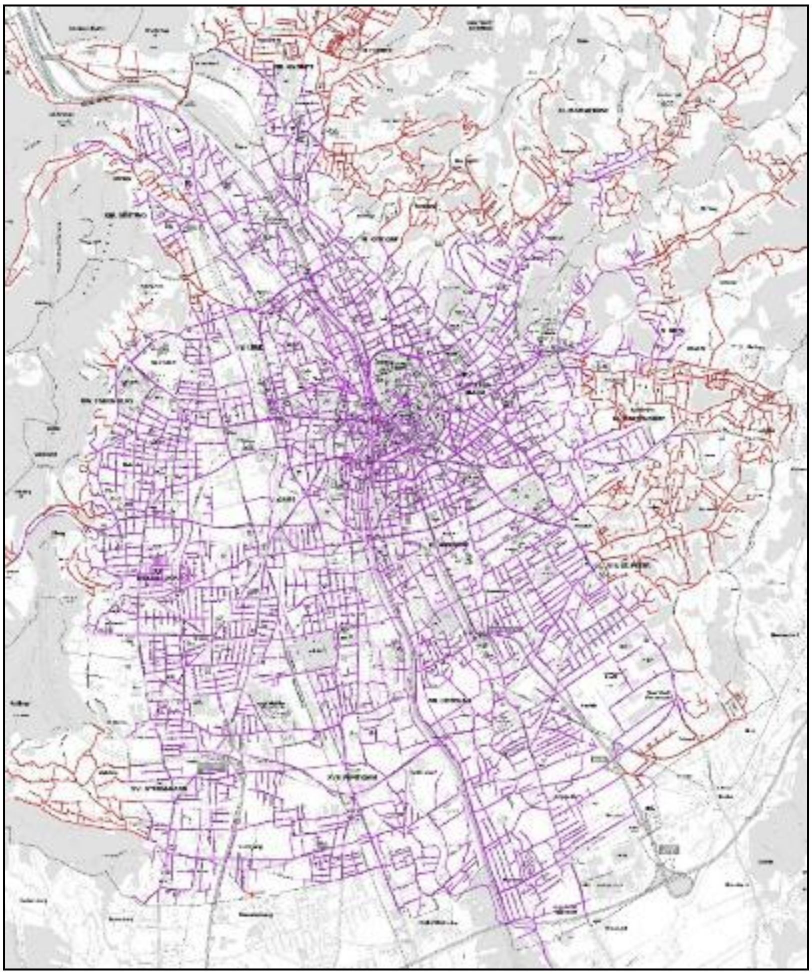
To simulate a full storm event, starting with rainfall and ending with inflow into the WWTP or respective outflow to the river, one model is not enough. So a conceptual runoff model was used to simulate the whole sewer system of the city of Graz during a rainfall event. The outputs of the conceptual model are the CSO discharges and the inflow into the main collector to the WWTP. These pathways are used as input for a more detailed hydrodynamic runoff model representing the ZSK and the last part of the main collector also including the CSO basin before the WWTP.

2.1.1 Description of the case study area

Graz is the second largest city in Austria with about 270 000 inhabitants in 2014. It lies in the south of the country at the river Mur (mean flow of 120 m^3/s) that starts in the Austrian Alps and enters the river Drave on the border of Croatia and Hungary.

2.1.1.1 Urban drainage system of Graz

The urban drainage system of Graz has a sewer network with a span of 854 km., 70% of which are set up as CSS. In Figure 2-2 purple signals CSS and red signals SSS.



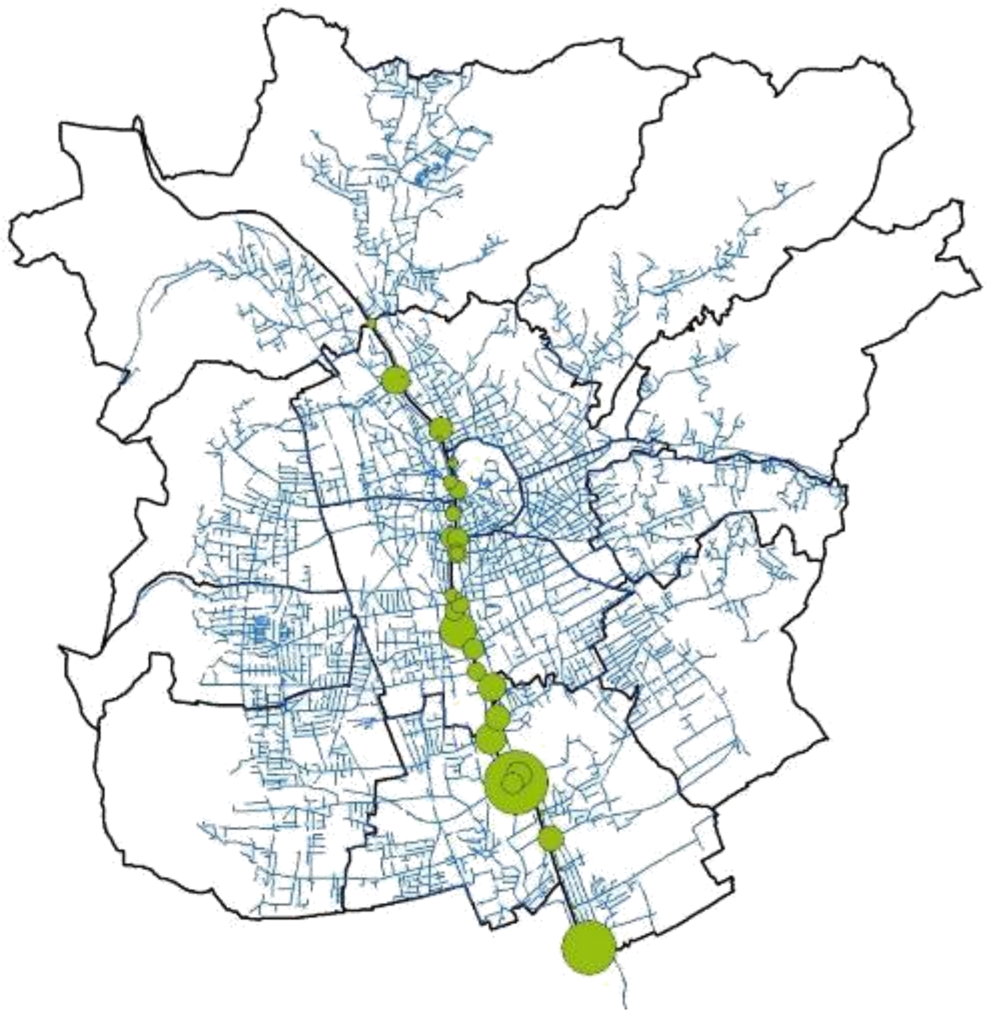
With: red … SSS; purple … CSS

Figure 2-2: Spatial distribution of the drainage system of Graz (Land-Steiermark, 2010)

Basic data on the sewer system of Graz (Land-Steiermark, 2010):

- 854 km sewers
    - ~ 577 km combined sewers
    - ~ 226 km sanitary sewers
    - ~ 51 km stormwater sewers
- 8 wastewater pumping stations
- 9 stormwater pumping stations
- 1 CSO basin measuring 12 000 m^3
- Collectors measuring ~ 20 000 m^3
- 37 CSO structures alongside the Mur
- 1 WWTP designed for 500 000 population equivalents (PE)

The 37 CSO structures alongside the Mur are shown in Figure 2-3. The size of the spots represents their significance in terms of discharged biochemical oxygen demand (BOD) load per year.



The dot size signals their BOD loads per year

Figure 2-3: CSO structures alongside the river Mur (Holding-Graz, 2013)

In terms of BOD there is a total amount of approximately 870 t/year entering the Mur. 660 t of which can be directly traced back to CSO events. That makes up for 76% of pollution just from CSO overflows with no means of retention or pre-treatment.

2.1.1.2 Adaption of the Graz drainage system

With two hydropower plants planned south of Graz, specifically in Gössendorf and Puntigam, a synergy project was created. To produce enough height difference to effectively obtain energy in a hydropower plant in a river, the river needs to be retained which results in backwater. The beginning of this backwater effect reaches up the stream whereas the distance varies with the decline of the river and other factors of its surroundings. Because of the backwater threatening to flood some of the CSO structures and the need for the city to upgrade their CSO handling strategy plans, came the idea to build a collector tunnel. This central storage tunnel (termed as ZSK), follows the Mur and redirects the CSO overflows downstream of the plants and at the same time stores the runoff water with the help of moveable weirs separating the tunnel into storage cascades. The companies planning the hydropower plant were forced to do this because the city of Graz and the sewer operators have the permits to discharge water into the Mur and the hydropower plant would prevent them from doing so by raising the water level of the Mur.

2.1.1.3 Development of the central storage tunnel (ZSK) of Graz

In 2012 the first part of the ZSK was finished and in 2013 it was connected to the WWTP in Gössendorf. Figure 2-4 shows the already constructed and planned implementation of the project.

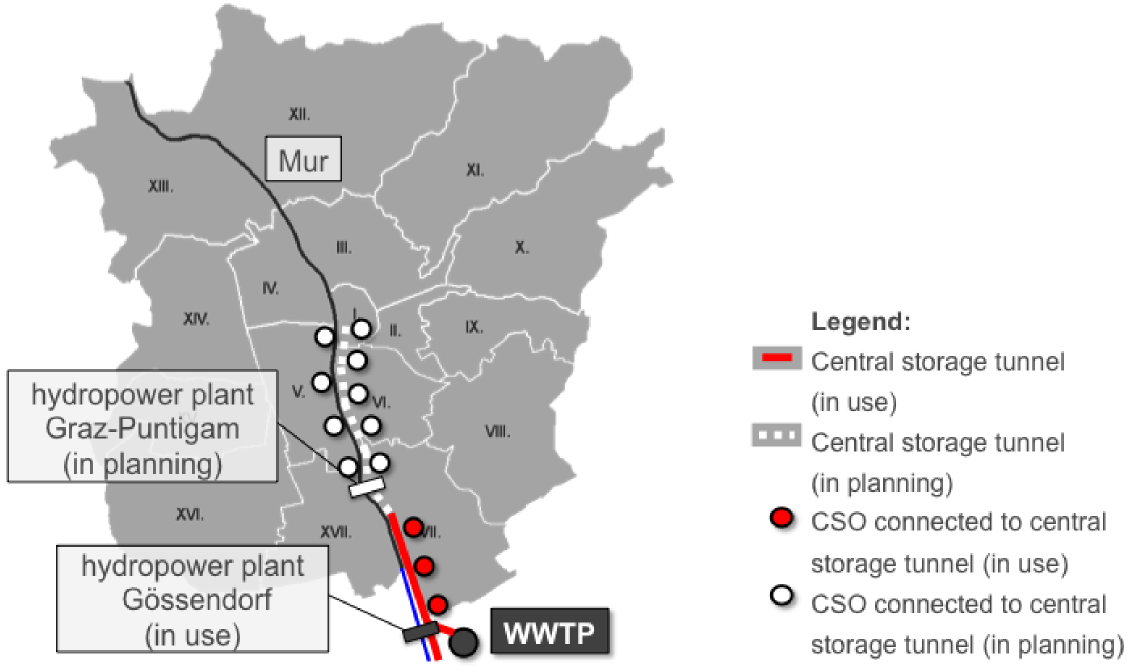


Figure 2-4: Location of the ZSK in Graz with the locations of the hydropower plants and the affected CSO structures (Golger, 2014 modified)

The red line symbolizes the already constructed part of the ZSK, with the red circles showing the CSO structures covered by the first phase of the project. The dotted white line and the white circles show the area covered by the second stage of the project.

Together both parts of the ZSK are going to hold a volume of 91 000 m³, which means that together with the CSO basin already in place at the WWTP that holds 12 000 m³, a total storage volume of 103 000 m³ will be reached. With that volume, two thirds of the yearly BOD load could first be stored and then treated in the WWTP (Holding-Graz, 2013). This system would enable Graz to fulfill the requirements of the OEWAV Guideline 19 (OEWAV, 2007b).

2.1.1.4 Investigation area in this study

Currently a storage volume of 22 000 m³ of the already existing section of the ZSK and 12 000 m³ at the CSO basin at the WWTP is in place. Figure 2-5 shows the area that will be considered in this thesis.

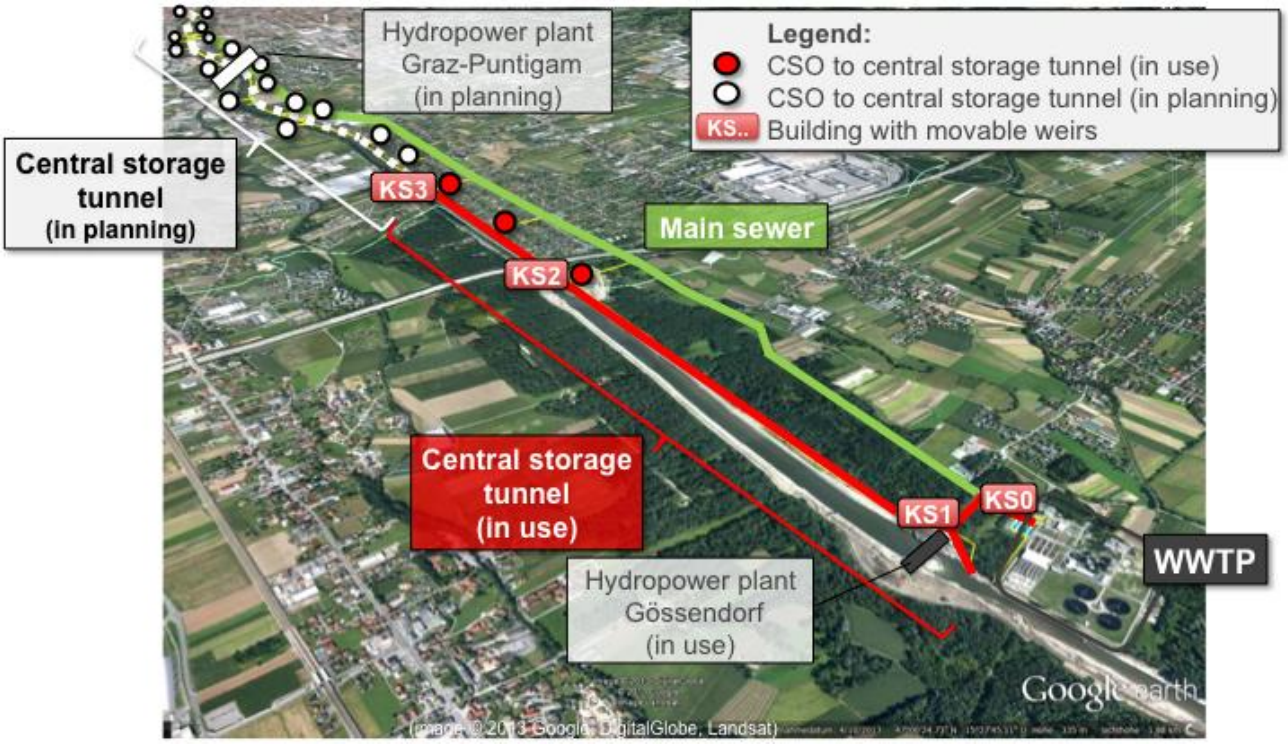


Figure 2-5: Considered area of the thesis (image © 2013 Google, DigitalGlobe)

KS 0 to 3 mark the movable weirs and orifices that will be used to control the ZSK. Each weir has at least one sensor to measure the water depth installed. The KS0 structure is also equipped with a flow measuring system. The weir KS3 separates the collector tunnel from a flushing chamber that is connected to the river Mur and can be filled with river water to flush the whole tunnel. The chamber holds approximately 400 m³ and can be filled in about 4.5 minutes. To regulate the flushing chamber, an orifice was put into place.

The current overflow structures that monitor the only possible overflows in the projected area are shown in Figure 2-6.

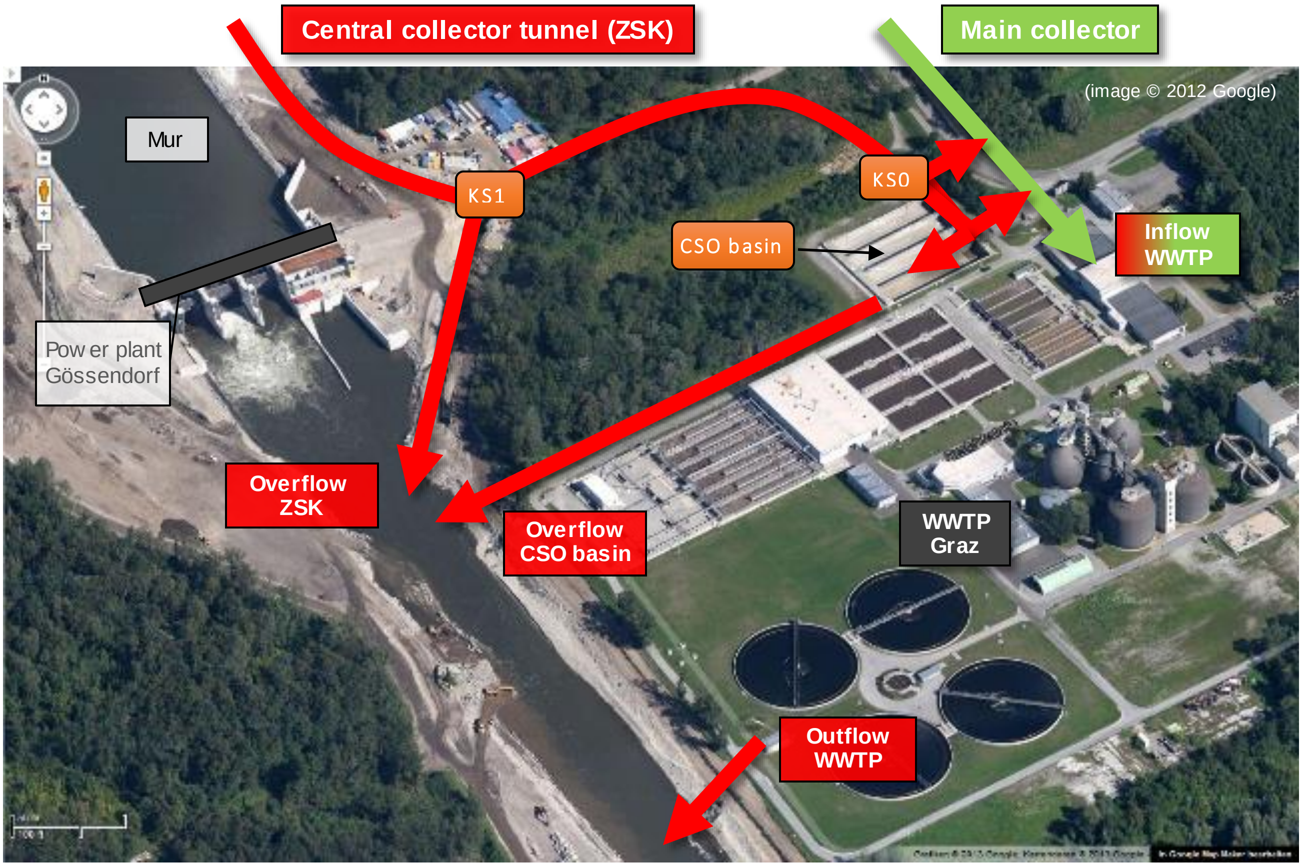


Figure 2-6: **Overflow structures for the projected area (image © 2013 Google, DigitalGlobe)**

During heavy storm events, the ZSK can overflow directly into the Mur. To empty it after a storm event, down to a depth of 3 m at the lowest point of the ZSK, the water can flow towards the main collector gravitationally. The rest of the stored stormwater is emptied into the CSO basin where it is pumped towards the WWTP with two screw pumps at a total rate of 0.48 m³/s.

2.1.2 The functionality of a moveable weir

The weirs installed in the ZSK (ASA Technik GmbH) are basically vertical walls that can be sunk into the ground until they vanish completely leaving the full cross section for the water to flow through. They are constructed as stainless steel coated armored concrete plates with hydraulic jacks to control them. In the already built section of the ZSK, there are two of these weirs installed (KS2 and KS3). After the whole collector tunnel is finished, eight of these weirs are going to be operated in the whole facility.

Table 2-1 gives an overview of the main attributes for the mounted weirs.

Table 2-1: Attributes of the installed weirs from the company ASA (http://wp.asatechnik.de/kaskadenwehre/, 2014-10-21)

Dimensions	Width 3.2 m; height 3.8 m
Weir speed	7 – 9 m/min
Weir construction	Site-mixed concrete or precast concrete component
Sensors	Water level before and after the weir, current weir setting
Power supply	5 – 9 kW/h (depending on the size of the weir

- The moveable weirs in the ZSK can be used to …
  - … use the volume created by the cascades as CSO storage.
  - … flush the collector tunnel.
  - … reduce the runoff peaks for the WWTP.
  - … change the condition of the stored stormwater (decantation).

Figure 2-7 explains the functionality of a moveable weir. Illustrations one and two show the normal operating process enacted during a storm event. The weir is raised while the stormwater flows in. The water level in front of the weir rises until the cascade is filled up. If more stormwater comes, it will simply overflow to the next cascade. Illustrations three and four demonstrate how the flushing process works to remove deposits. With the accumulated stormwater behind it, the weir is lowered and a wave starts to form that flushes the cascade(s) below the weir.

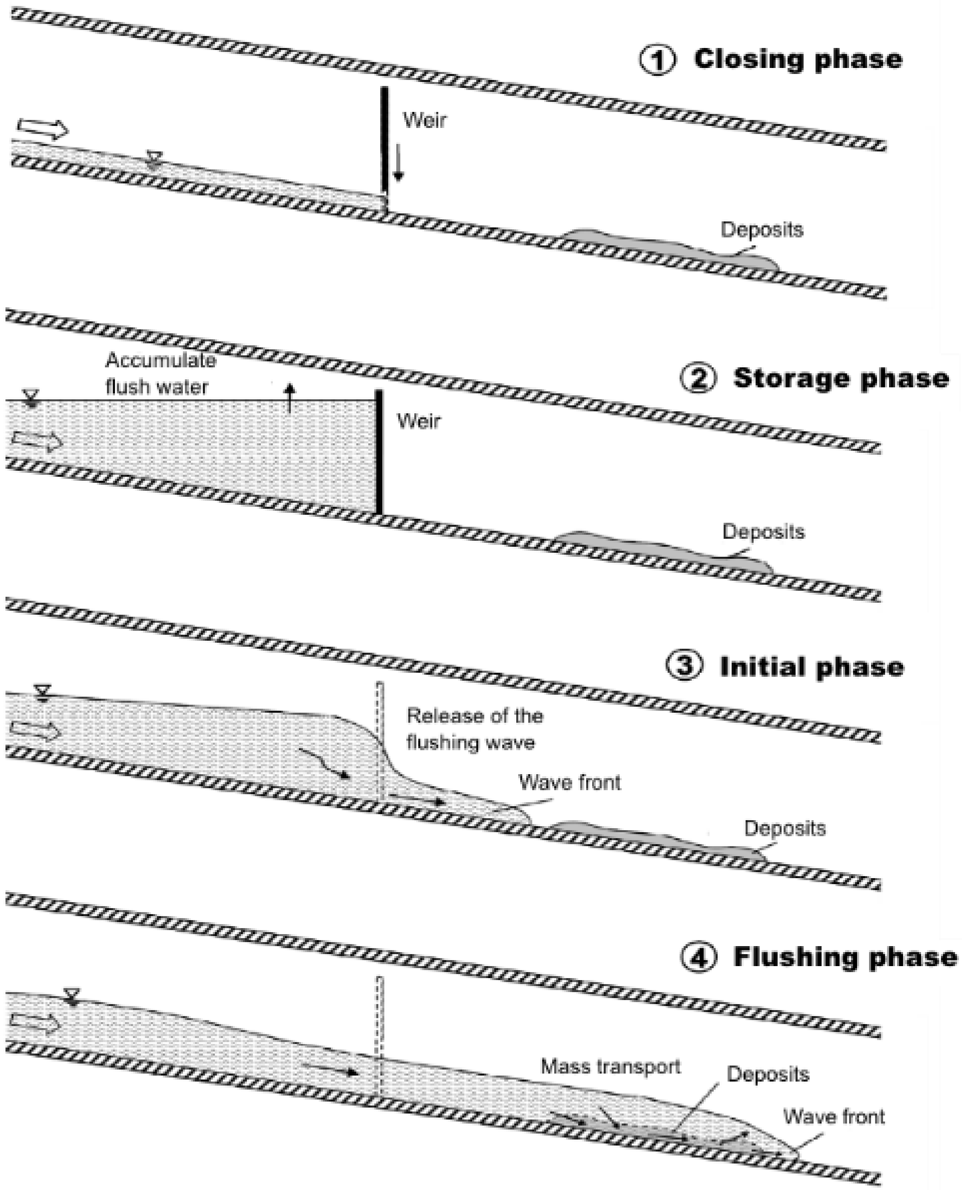


Figure 2-7: Functionality of a moveable weir (Dettmar, 2005 modified)

2.1.3 Description of the integrated model setup

In the model the process from rainfall to runoff flowing either into the WWTP or into the river is simulated (see Figure 2-8). To do this, a conceptual runoff model was used as a grey-box model to simulate the behavior of the sewer system of Graz. This model uses the rain input data of a single pluviograph. As an end result, the model delivers the overflows of the various CSO structures and the inflow into the main collector that flows towards the WWTP. This whole approach is called boundary relocation and is used to lessen the computation time of the simulation. It cuts elements and areas free of the total system and replaces them with a grey-box model as long as it produces correct results. Figure 2-9 shows an example of such a boundary relocation. The dashed rectangles represent the areas, which can be replaced by a surrogate model. More information on this approach can be found in Vanrolleghem *et al.* (2005).

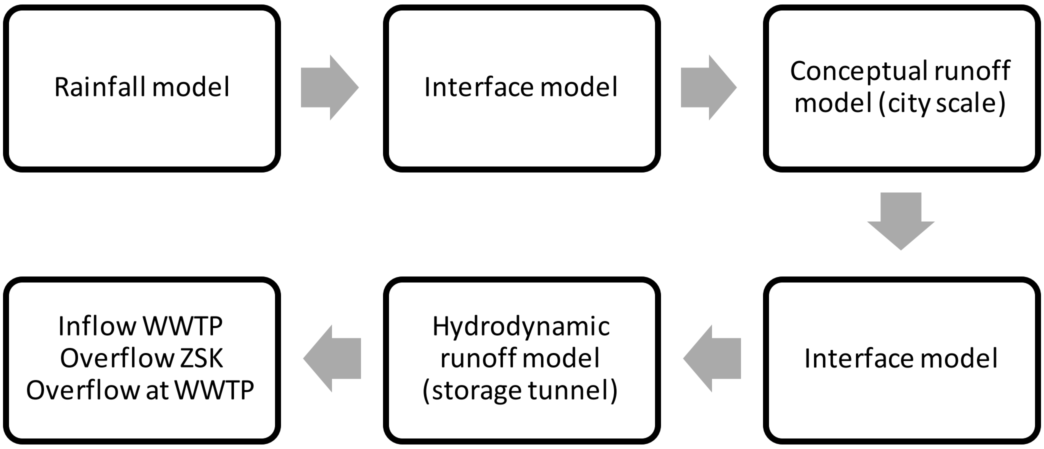
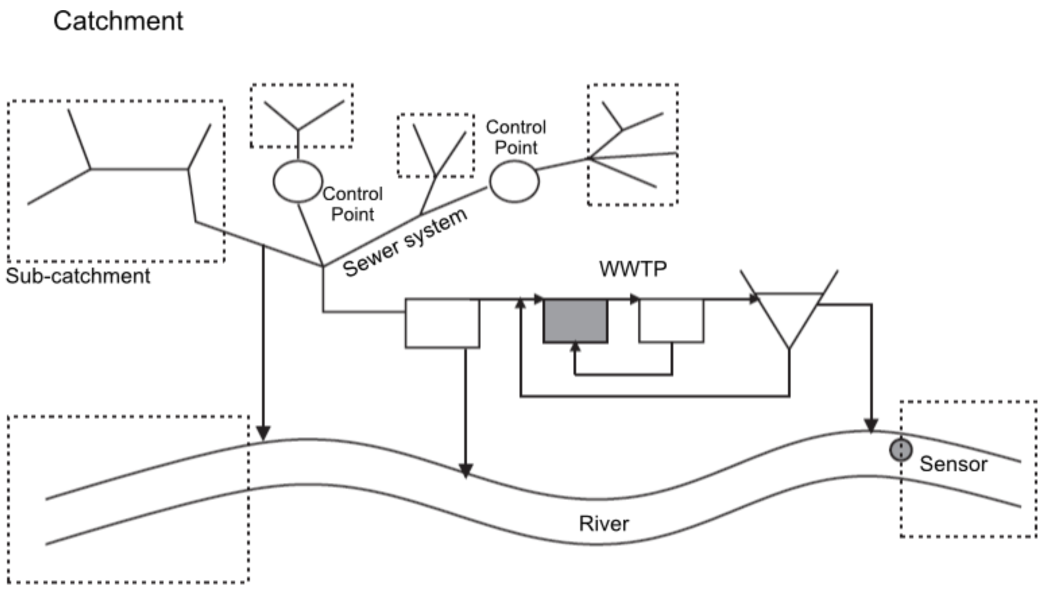


Figure 2-8: Integrated simulation process



The dashed rectangles represent the areas that can be surrogated by a faster model

Figure 2-9: Example for boundary relocation (Vanrolleghem *et al.*, 2005 modified)

2.1.3.1 Rainfall model

To actually test an RTC strategy, a rainfall has to be simulated strong enough to actually trigger a CSO. Otherwise the comparison to a reference scenario without RTC does not make much sense.

At first an attempt to find a rainfall event from actual records provided by pluviographs was made. The provided data, however, only contained events in the timespan of 1989 to 2006 (OEWAV, 2007a). Only in the last couple of years has the intensity of storms risen significantly in Graz. Therefore it was decided to produce artificial Euler type II rains based on normal rainfall amounts for the area of Graz. Originally return periods of 1, 2, 5, 10, 20, 30 and 50 years with 90-minute durations were to be considered. But after deciding that the WWTP's treatment capacity will be assumed to constantly be at its maximum rate of 3 m^3/s, only the return periods of 20, 30 and 50 years produced overflows, so the storm events with shorter return periods were not examined further. Also it needs to be mentioned here that rain storms with longer durations and smaller return periods would also produce overflows. The intense storms used here are merely chosen to produce CSOs in a short period of time.

To generate an artificial Euler type II rain, the guideline DWA-A-118 from DWA (2006) was used. First the rain level curves were supplied by the eHYD platform, a GIS system with high-resolution rainfall and runoff data (http://ehyd.gv.at/). Then the differences between every 5-minute step were calculated. An Euler type II rain has its peak after 0.3 times the total rain duration. To obtain that amount, the differences only needed to be reordered to create the Euler rain.

Table 2-2: Creating an Euler type II rain for Graz

	Rain level curve			Differences			Euler type II		
Return period	**20**	**30**	**50**	**20**	**30**	**50**	**20**	**30**	**50**
Duration [min]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]
5	17.50	18.70	20.20	17.50	18.70	20.20	5.50	6.10	6.80
10	26.10	28.10	30.70	8.60	9.40	10.50	8.60	9.40	10.50
15	31.60	34.20	37.50	5.50	6.10	6.80	17.50	18.70	20.20
20	35.70	38.60	42.40	4.10	4.40	4.90	4.10	4.40	4.90
25	38.75	42.00	46.10	3.05	3.40	3.70	3.45	3.65	4.05
30	41.80	45.40	49.80	3.05	3.40	3.70	3.05	3.40	3.70
35	43.53	47.23	51.83	1.73	1.83	2.03	3.05	3.40	3.70
40	46.98	50.88	55.88	3.45	3.65	4.05	2.15	2.30	2.50
45	48.70	52.70	57.90	1.73	1.83	2.03	1.73	1.83	2.03
50	49.78	53.85	59.15	1.08	1.15	1.25	1.73	1.83	2.03
55	51.93	56.15	61.65	2.15	2.30	2.50	1.08	1.15	1.25
60	53.00	57.30	62.90	1.08	1.15	1.25	1.08	1.15	1.25
65	53.88	58.25	63.88	0.88	0.95	0.98	0.88	0.95	0.98
70	54.77	59.20	64.87	0.88	0.95	0.98	0.88	0.95	0.98
75	55.65	60.15	65.85	0.88	0.95	0.98	0.88	0.95	0.98
80	56.53	61.10	66.83	0.88	0.95	0.98	0.88	0.95	0.98
85	57.42	62.05	67.82	0.88	0.95	0.98	0.88	0.95	0.98
90	58.30	63.00	68.80	0.88	0.95	0.98	0.88	0.95	0.98

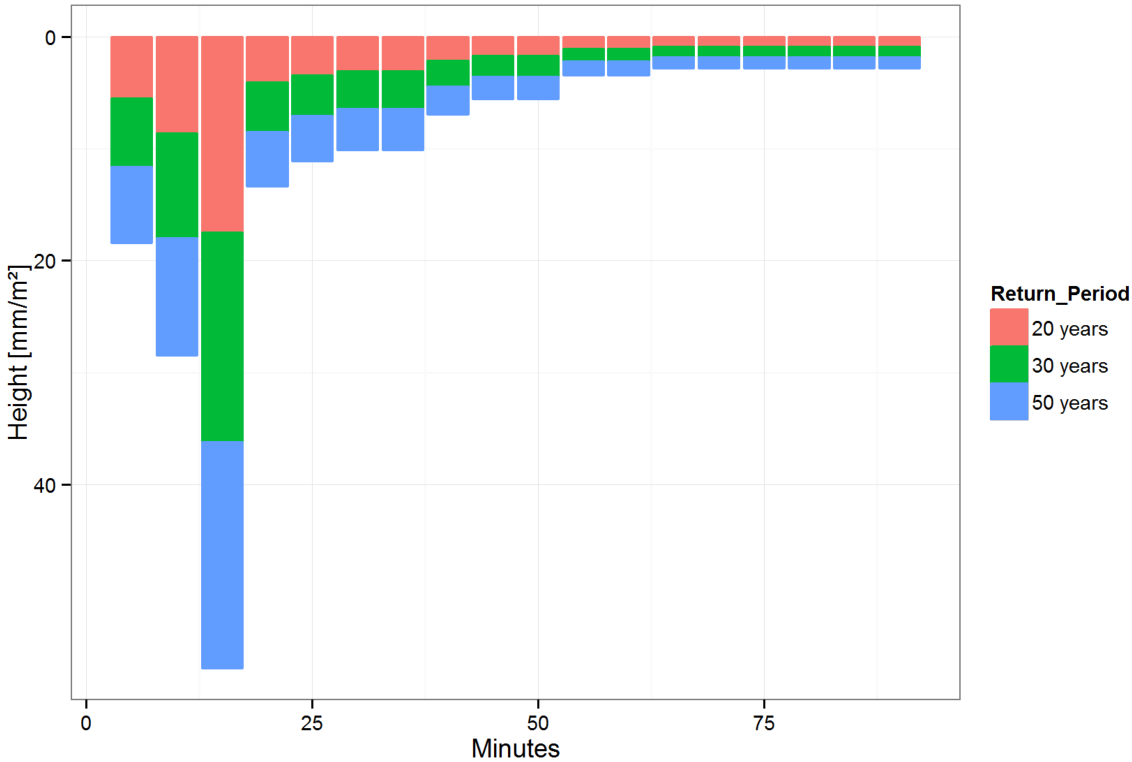


Figure 2-10: Resulting Euler type II rains

With this rainfall, three input files were generated for the conceptual KOSIM model, which was developed in the still ongoing project from the institute of urban water management of Graz University of Technology called iZSK. The KOSIM simulation then produced outputs for the CSO facilities that discharged into the collector tunnel or respectively into the main collector. Those output files were later translated into time series files, which were then used as input files for the SWMM simulation. More about that process can be found above in chapter 2.1.1.

2.1.3.2 Conceptual runoff model

KOSIM is a long-term simulation tool capable of simulating catchments and storage basins. It is a modeling tool from the company ITWH (http://www.itwh.de/) and it is implemented as a conceptual model, which fits the needs of this work. The benefit of this model is the short computing time for a large area with a complex sewer system. Even though the conceptual approach only incorporates the continuity equation, the results are sufficient for the requirements of this simulation. The model was supplied by the institute of urban water management and landscape water engineering of Graz University of Technology.

2.1.3.3 Hydrodynamic runoff model

SWMM 5 is a modeling tool from the United States Environmental Protection Agency (http://www.epa.gov/) and is essentially a one-dimensional hydrodynamic modeling tool. The tool supplies a variety of regulators that can be set up to a high detailed model (see Table 2-3). Also the software is an open source project, which allows for some alterations in its code if needed. It is able to dynamically simulate rainfall-runoff scenarios in primarily urban areas for surfaces and sewers. Outputs are generated as time series for each node and link of the modeled system, delivering quantity and quality parameters. There are various tools to present the data SWMM produces. One of which is PCSWMM, which combines GIS functionality with the SWMM5 engine and is developed from CHI (http://www.chiwater.com/). PCSWMM is used in this thesis for its ability to quickly plot different variations of data to give a quick overview of each created scenario.

Table 2-3: Regulators in SWMM and their usage

Type	Usage
Orifice	Openings in walls, storage facilities or control gates
Weir	Along the side of a channel, within a storage unit
Outlet	Controls outflow from storage units
Pump	Not a traditional regulator but can be used alike

2.1.4 Challenges of the integrated model setup

As with every simulation, various challenges come up that need to be tackled before precise enough results can be achieved. The following subchapters list the main challenges that were encountered during the implementation of the model in this thesis.

2.1.4.1 Integration problem

There are three different forms of integrations when it comes to an integrated model (Laniak *et al.*, 2013).

Model-wise integration

The problem definition model-wise is to find the right model that produces the correct output so that the connected model gets the correct input data to continue with the simulation. So in the case of the situation at hand, the conceptual runoff model needs to produce output representing the runoff in the form of flow so that the data can be used as input for the hydrodynamic runoff model.

Semantic integration

In some cases the output of one model resembles the correct form of data but needs to be modified semantically to fit the input requirements of the connected model. For example, the chemical oxygen demand (COD) is only considered as a single parameter in sewer models. In WWTP models however it needs to be separated into different fractions like slowly and quickly degradable COD. This transformation is considered semantic integration.

Technical integration

Technical integration means that the output data has the right dimension and is semantically correct. Only the format of the data needs to be adjusted. In this work the problem is the following. As the output files of the conceptual model are not compatible with the input files that the hydrodynamic runoff model needs, a script was implemented in R (R Core Team, 2013) that translates one into the other.

2.1.4.2 Time sensitive movement of a weir using PID-controllers in SWMM5

Most weirs in urban drainage have a fixed weir crest. Even with a weir flap, its movement is not of importance because in the overall result it does not make a significant difference. The moveable weirs that were installed in the ZSK are up to 3.8 m high and sink into the floor at a speed of 7 to 9 m/min when they are opened. For detailed modeling of the flushing scenarios, this movement is important enough that is has to be implemented in the model. There are three ways to do so:

- Time series

  With the option of control rules it is possible to manipulate the setting of a weir. That means that a time series can be created that represents the movement of the weir over time with a graph. The downside of that option is that the time series is either bound to the clock time of the simulation (12:15 pm) or the simulation time (e.g. 1:54 h after the simulation's start). That means that the weir control is not variable enough to use it in RTC. The syntax for the necessary control can be found in Figure 2-11.

```
RULE time_series
IF NODE node1 DEPTH >= 1.0
THEN WEIR weir1 SETTING = TIMESERIES setWeir1
```

Figure 2-11: Syntax for a control rule featuring a time series

- Time to open/close

  The regulator orifice in SWMM has an option that is called "time to open/close". With that option, as the name already suggests, it sets the time for the regulator to fully open. This type of functionality doesn't yet exist for a weir. However, with a little effort, it is possible to adapt the core functions of SWMM to add that functionality. The disadvantage of this approach is that whenever a SWMM update is installed, this process has to be repeated.

- PID controller

  It is also possible in SWMM to implement a PID controller within the control rules. PID stands for proportional, integral, derivative, which are the three different tuning parameters for this controller. In a control loop, these three parameters converge together towards a predefined set point. The process produces an output that is compared to the desired result, which results in an error e(t). With e(t) and the parameters set for the PID controller ahead of time, a new input value for the process is generated that produces a new output and so on to continuously minimize e(t). The process is shown in Figure 2-12. Such a PID controller can be used to simulate time sensitive movement of a weir. Within a control rule a water level is defined that has to be achieved by setting the weir. With every time step of the simulation the weir setting is adjusted to minimize the error produced by the last setting. The correct parameters for the PID controller result in the correct time sensitive movement of the weir.

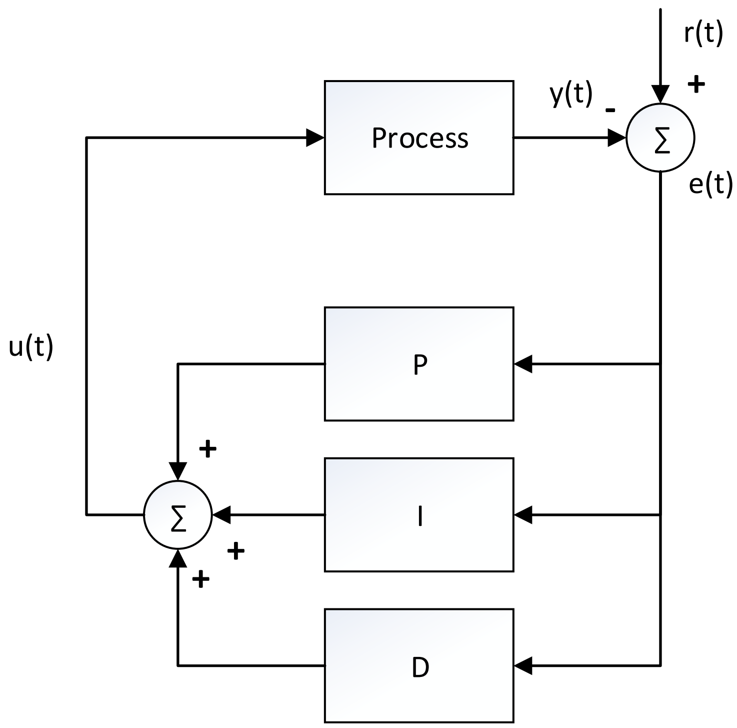


Figure 2-12: Process scheme of a PID controller; u(t) … controller output; y(t) … process output; r(t) … target value; e(t) … error

An example of the syntax of such a control rule is shown in Figure 2-13

```
IF PUMP pump1 STATUS = ON
AND NODE node1 DEPTH <= 1.0
THEN WEIR weir1 SETTING = PID 0.01 -0.03 -0.05
```

Figure 2-13: Syntax for a control rule featuring PID

This solution is independent of the simulation time, so it was used in the model.

To see if the PID control works accurately, two simulations were set up for evaluation. In the first one, a weir was controlled with a time series, and in the second one, a PID controller was used. In order to analyze the scenarios, the produced wave height (depth) before and after the weir was compared. The following figures show the comparison between PID and time series control of a single wave (Figure 4-3) and a wave sequence of seven waves (Figure 4-4).

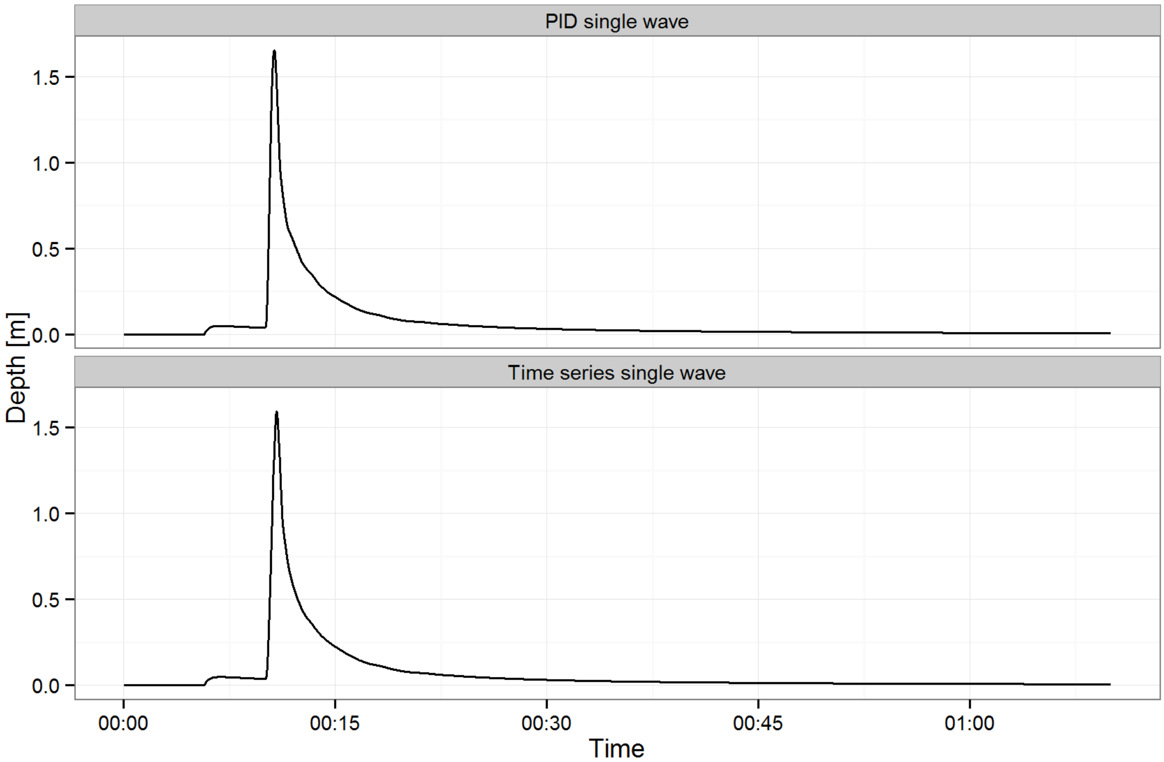


Figure 2-14: Comparison between PID and time series control of a single wave

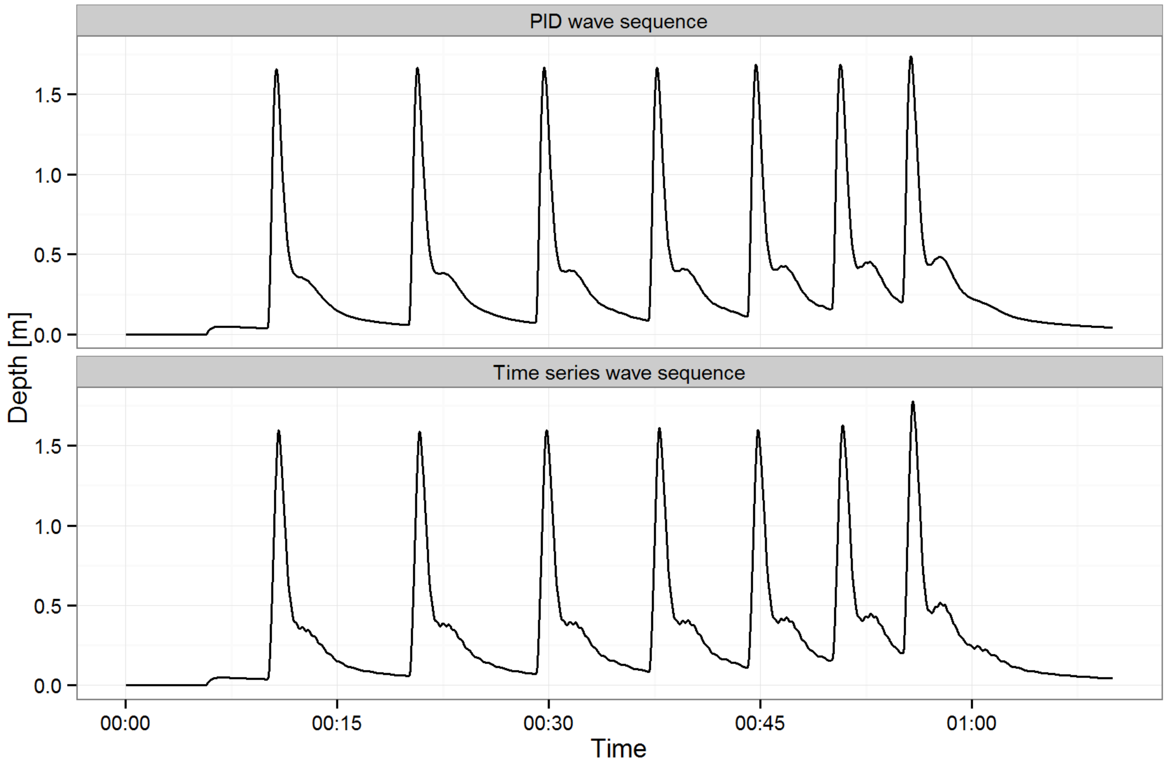


Figure 2-15: Comparison between PID and time series control of a wave sequence

It can be seen that, except for some minor differences in the wave's stability, the outcome of both control approaches is the same. So using PID controllers to control a weir appears to be an efficient way to simulate the weir's movement.

2.1.4.3 Modeling a loss free weir in SWMM

In SWMM flow over a weir is calculated with Poleni's equation:

$$Q = \frac{2}{3} * \mu * \sqrt{2 * g} * B * h^{\frac{3}{2}}$$ **Equation 2-1**

With Q: flow (m³/s), µ: weir coefficient, g: gravity (m/s²), B: weir crest width (m) and h: weir head (m)

In the software tool, the weir coefficient is represented by the discharge coefficient, which is calculated as follows:

$$dCoeff = \frac{2}{3} * \mu * \sqrt{2 * g}$$ **Equation 2-2**

With µ: weir coefficient and g: gravity (m/s²)

The problem with this approach is that in the case of the ZSK, the weir doesn't produce any losses, because it completely sinks into the invert. So to compensate for this error, two actions were taken. The first one was to find the discharge coefficient producing a minimum local loss. To evaluate the different values of dCoeff, the water level before, at and after the weir where compared. The optimal value produces the least difference between those three water levels. Table 2-4 shows the results of that procedure.

Table 2-4: Finding the optimal discharge coefficient for a minimal local loss

		Q			Depth		
dCoeff	**μ**	**Before**	**Weir**	**After**	**Before**	**Weir**	**After**
2.95	1.00	1.00	1.00	1.00	1.35	0.29	0.32
2.00	0.68	1.00	1.00	1.00	0.39	0.29	0.30
1.00	0.34	0.90	0.90	0.90	0.50	0.27	0.28
4.00	1.35	1.00	1.00	1.00	0.33	0.30	0.30
4.50	1.52	1.04	1.02	1.02	0.32	0.30	0.30
5.00	1.69	1.07	1.05	1.05	0.32	0.30	0.31

A dCoeff of five produced the least possible loss without making the model unstable. The flow and the water level before and after the weir show that there are only minimal differences after changing the coefficient.

A second measure was taken during calibration. As the average loss coefficients were adjusted for every link to match the measured speed of the actual ZSK (read chapter 2.1.5.5 for more details), the found loss coefficient was not applied to the links right after a weir to lessen the effect of Poleni's equation even further. That means that not applying the loss coefficient to those links counteracts the slight backwater effect before a weir.

After putting these two measures into place and considering the fact that for the flushing scenarios, the loss at the weir is not a determining factor for the flow at the end of the tunnel, the gained result was decided to be the best possible option.

2.1.4.4 Quick adjustment of input files for SWMM

To find the best strategies for RTC, emptying and flushing the collector tunnel, lots of scenarios needed to be simulated. To run these scenarios, it was best to start SWMM without its graphical user interface and to manipulate its input files externally with automated scripts. These scripts were written in R.

2.1.4.5 Quick assessment of the results

To assess all the different scenarios, PCSWMM offers a plotting functionality to analyze time series. However, it wasn't used because it produced inconsistent plots and was not as variable as needed. Therefore the evaluation was also done with the scripting tool R with the package ggplot2 that produces clear plots to quickly analyze the data (Wickham, 2009). This also offered the possibility to run and evaluate every scenario automatically. The produced scripts became very complex in the end, but ultimately creating and using those automated scripts was simpler than analyzing every single scenario separately.

2.1.5 Structure of the hydrodynamic runoff model of the ZSK

A basic hydrodynamic runoff model was supplied by Holding Graz. It simulated the whole ZSK with the second planning stage already in place. This model was then modified and calibrated with measurement data from a previous project that was conducted in early 2014.

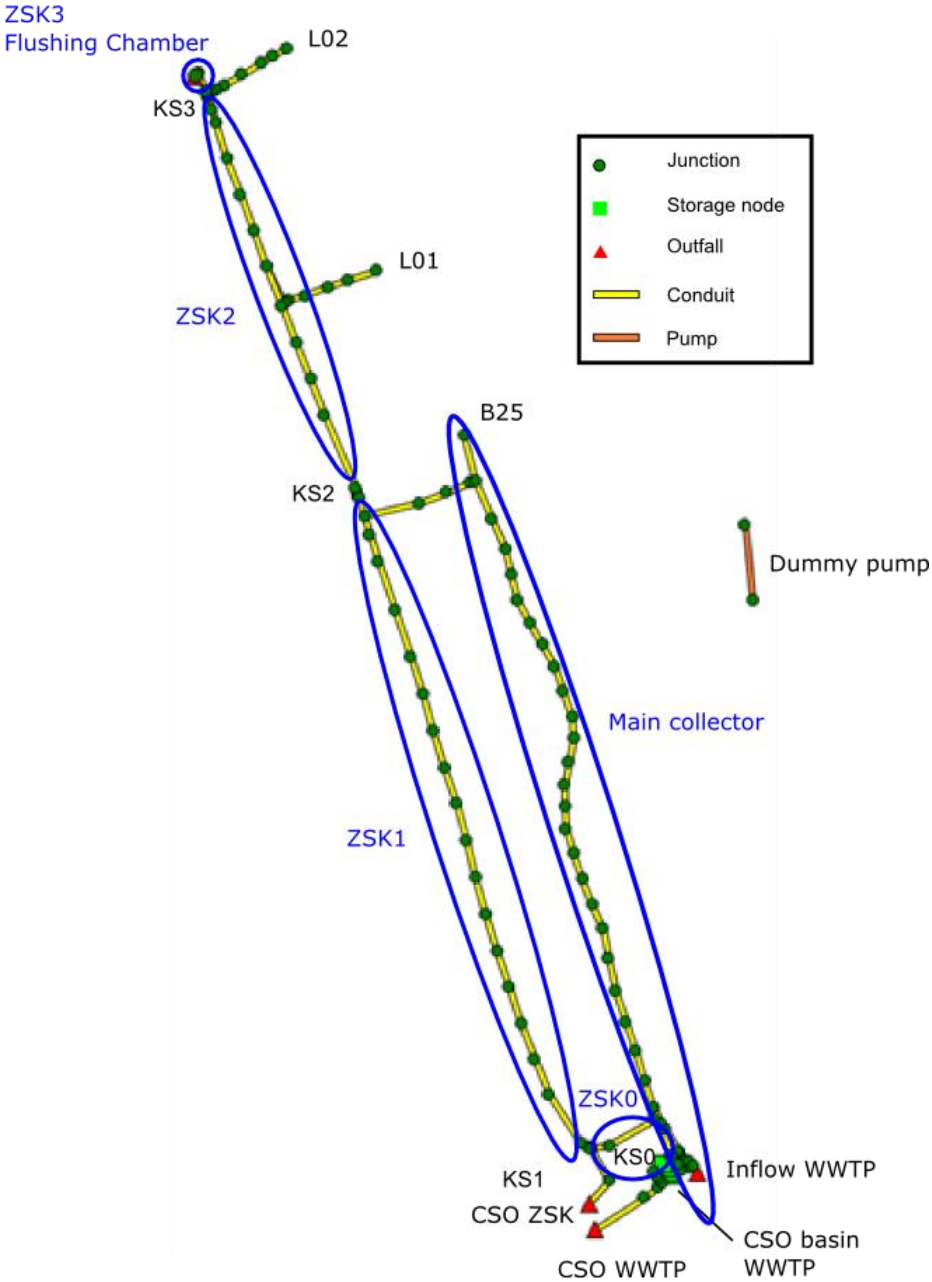


Figure 2-16: Map view of the model of the ZSK implemented PCSWMM

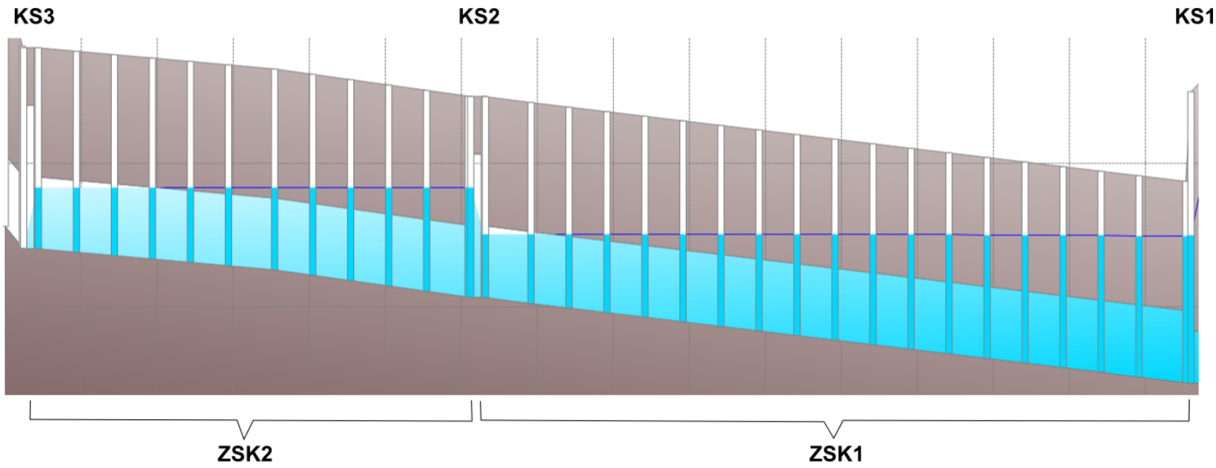


Figure 2-17: Section view with ZSK1 and ZSK2

The top left outfall node, as seen in Figure 2-16, indicates the river Mur. The water level is set to a fixed height of 3 m above the node to ensure constant flow into the flushing chamber when needed. To simulate the filling of this chamber, a pump was put into place that controls the inflow. The pump that can be seen on the right has no connection to the actual facility whatsoever. It acts solely as a switch that can remember a state independent of the current simulation, which is needed for the setup of the control rules. ZSK1 is the lower cascade of the storage tunnel between KS1 and KS2. ZSK2 is the upper cascade between KS2 and KS3. These names will be used for these sections throughout the rest of the thesis. Figure 2-17 shows the profile plot for ZSK1 and ZSK2.

The following changes were applied to the basic hydrodynamic runoff model:

- Irrelevant parts of the model (the not yet constructed facilities) were removed.
- The names of every link and node were standardized to simplify their handling in the scripts produced later.
- The invert elevations of the nodes were adjusted to fit the latest measurements submitted by a geodesist.
- The weir offsets of the sinkable weirs were set to zero.
- The discharge coefficient was adjusted as mentioned in chapter 2.1.4.3.
- The lengths of the conduits were adjusted so that they are equidistant with a length of approximately 100 m and to match the total length of the ZSK submitted by the geodesist.
- The size of the flushing chamber was adjusted to 400 m³.
- For the simulations to flush the ZSK, flap gates were installed for the conduits L_L02_A, L_L01_A and L_B25_A that channel the inflows from the sewer system (these flap gates were removed for the simulation of the RTC strategies)

- According to the documents supplied by Holding Graz, the orifices in the area of the K0 were given a time to open/close of 0.06 h.
- The profile of the weir chambers was adjusted to 320 x 500 cm.
- The roughness was set to 0.0107, which is equivalent to a k_{st} of 93.46 to match the characteristics of smooth and even concrete. This value was chosen during the calibration (read chapter 2.1.5.5 for more details)
- The average loss coefficient was set to 1.6 (read chapter 2.1.5.5 for more details).
- The depth of the CSO basin at the WWTP had to be adjusted to enable a possible overflow into the Mur.

2.1.5.1 Structure of the distribution building KS0

Figure 2-19 is a schematic of the KS0 structure, which is essential for the RTC control strategies during a storm event. There are two orifices and a weir (look at Figure 2-20 and Figure 2-21 for a more detailed view). The first orifice (K0_EINL) separates the ZSK from the WWTP. When opened, it gravitationally empties the ZSK into the main collector over a fixed weir (see Figure 2-18). This process works as long as the water depth in KS1 is above 3 m and K0_MUEB is closed. After falling below that limit the second orifice (K0_MUEB) facing towards the CSO basin is opened. The water then flows into the basin where two screw pumps transport the water to the main collector. The flow in the main collector is regulated by the facility B25 that cuts off the runoff that is too much for the lower part of the main collector to intake and channels it to the ZSK. The orifice K0_EINL is equipped with a sensor measuring flow and depth, so it can be adjusted to regulate the flow. During the emptying of the ZSK currently a fixed flow of 0.6 m^3 is applied so that the capacity of the WWTP is not maxed out. K0_DR is a fixed weir to stop water from the main collector from entering KS0. It can also be used to regulate the gravitational flow from KS0 to the main collector. The weir ARA_MUEB regulates the inflow into the WWTP down to 3 m^3/s, which equals double the maximum dry weather flow that the WWTP was designed for.

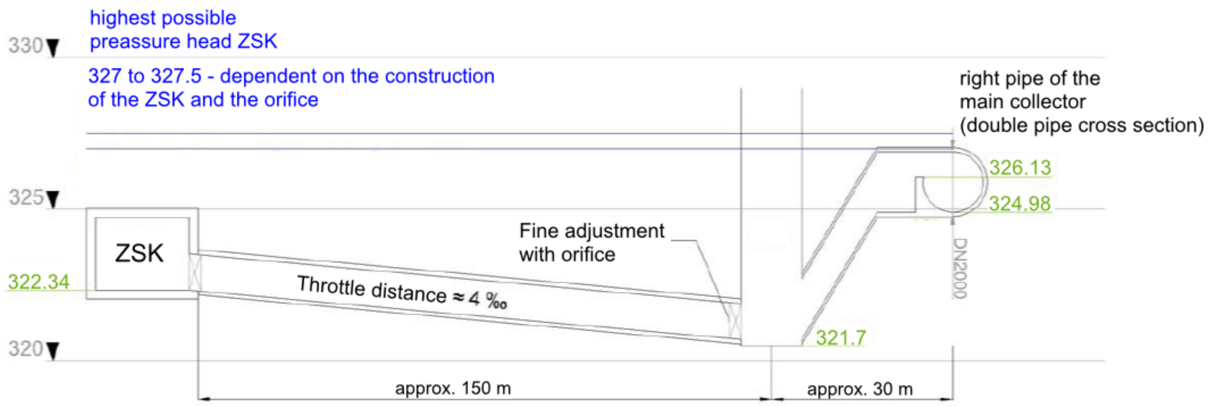


Figure 2-18: **Section view of KS0 for gravitational emptying (Institute for urban water management, 2007 modified)**

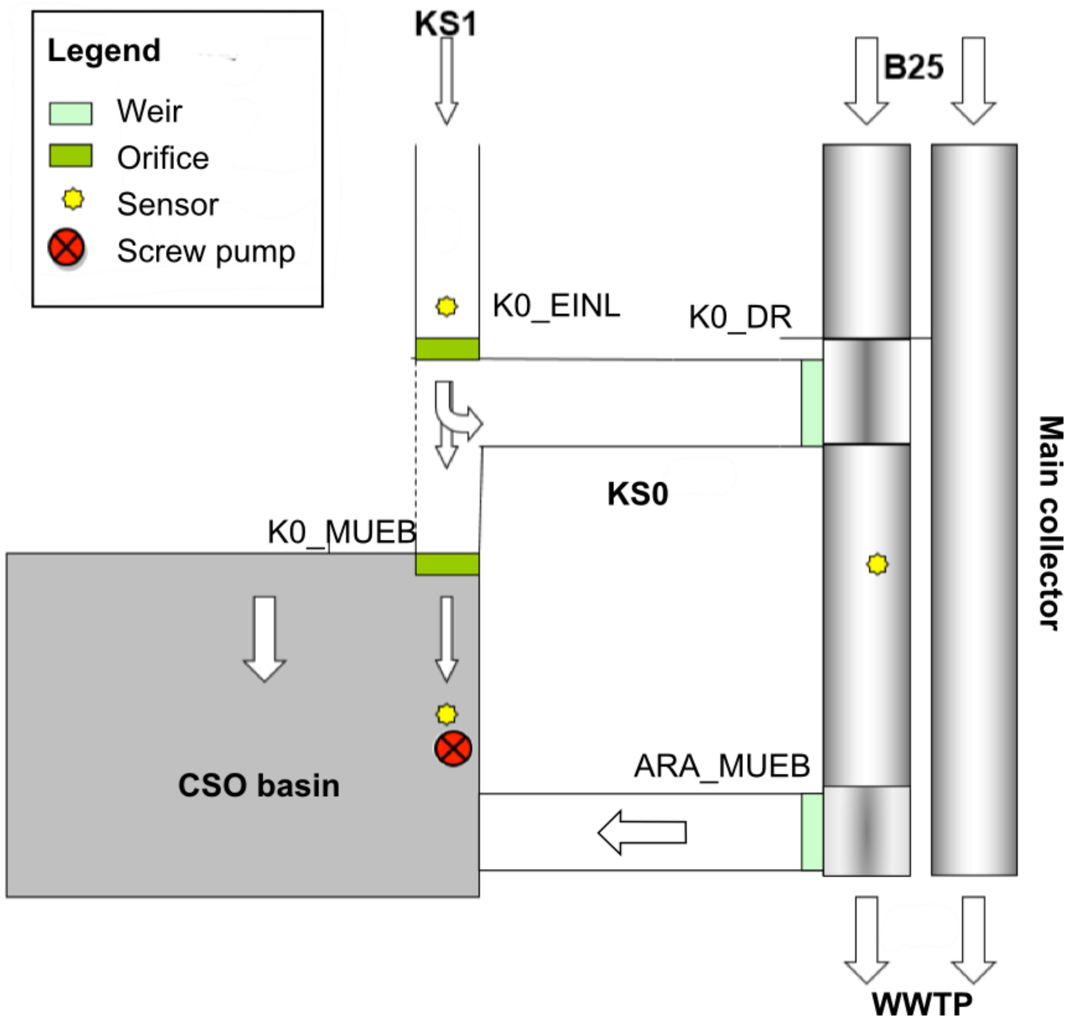


Figure 2-19: **Schematic of the KS0 structure (Holding-Graz, 2013 modified)**

Figure 2-20 represents how the lowest section of the ZSK was implemented in SWMM. The overflows from ZSK and WWTP are directly discharging into the Mur. The storage basin is implemented with a series of weirs and orifices to represent the four separate basins and to fill them equally during a storm event.

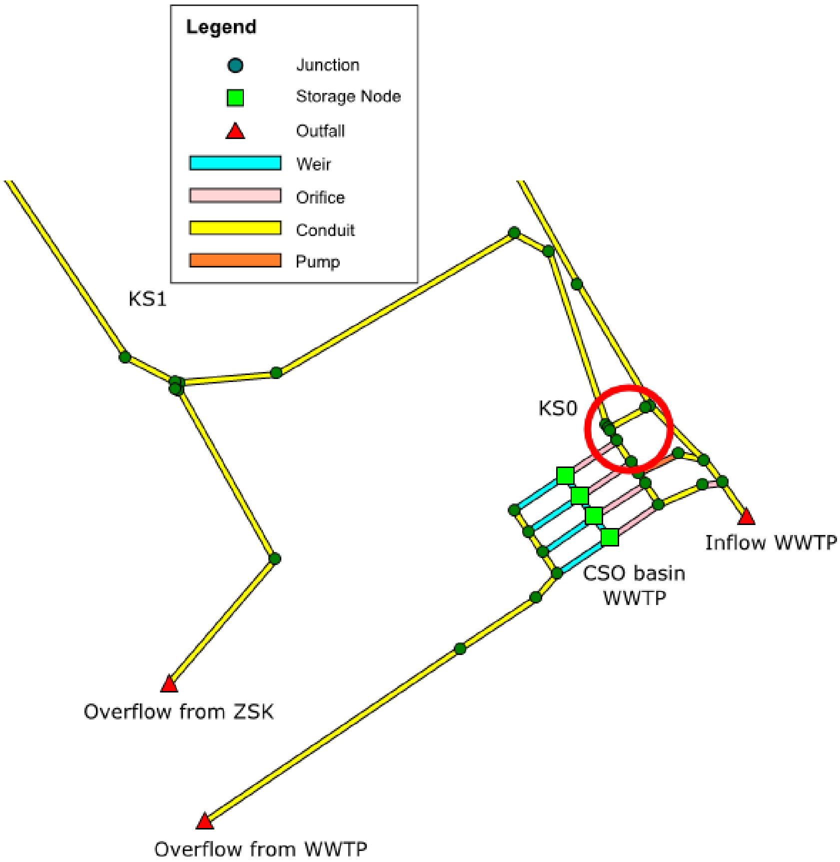


Figure 2-20: End section of the ZSK implemented in SWMM

Figure 2-21 shows the three regulators that form KS0 more detailed. During a storm event K0_MUEB is closed and K0_EINL is used for all control actions.

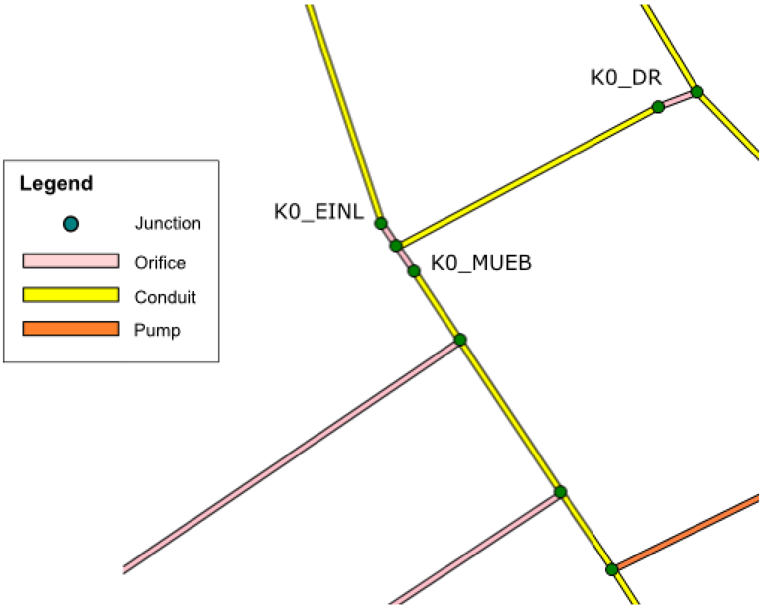


Figure 2-21: Implementation of KS0

2.1.5.2 Structure of the weir building KS1

KS1 is the facility at the bottom of ZSK1. It contains three weirs and two orifices. On the left in Figure 2-22 two weirs with flaps can be seen. These weirs have a height of 5.5 m with their flaps up and ensure, that during a massive storm event with a return period of 50 years and a long duration, enough water can be discharged into the Mur to prevent flooding in the upper parts of the ZSK or the sewer system. Another weir is situated on top of the orifice SB03 for the same reason. Its weir crest however is fixed and has a height of 5.15 m. The orifice SB03 can be opened for revisions in KS0 to directly discharge the runoff into the Mur. The last orifice on the bottom of Figure 2-22 points towards KS0 and is only closed for revisions in KS0. The sensor measures the water level in KS1.

Figure 2-23 shows the implementation of KS1 in PCSWMM. K1_O1 and K1_O2 resemble the weirs with the flaps, K1_O3 the one with the fixed weir crest. K1_MUR implements the orifice towards the Mur and K1_ARA the one towards the WWTP. The sharp bends of the weirs and orifices were only implemented to make it easier to distinguish every single regulator.

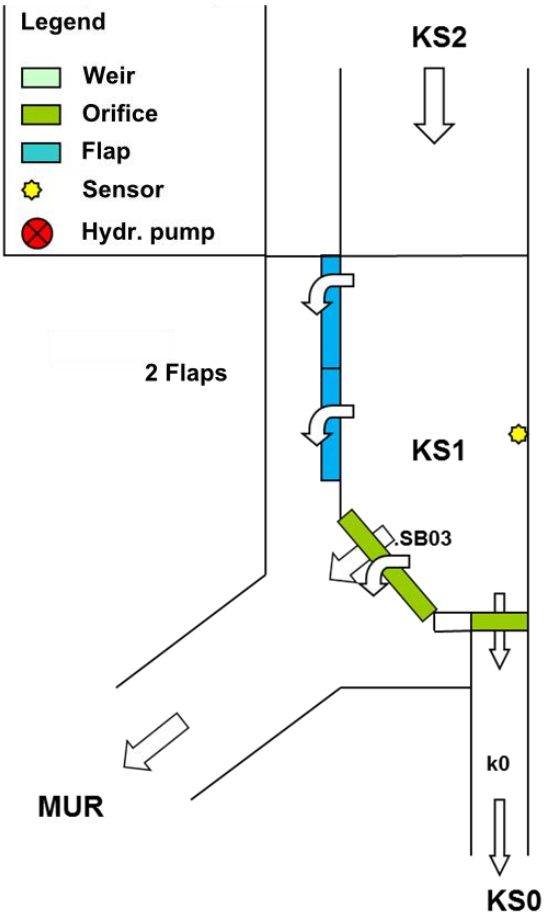


Figure 2-22: Schematic of KS1 structure (Holding-Graz, 2013 modified)

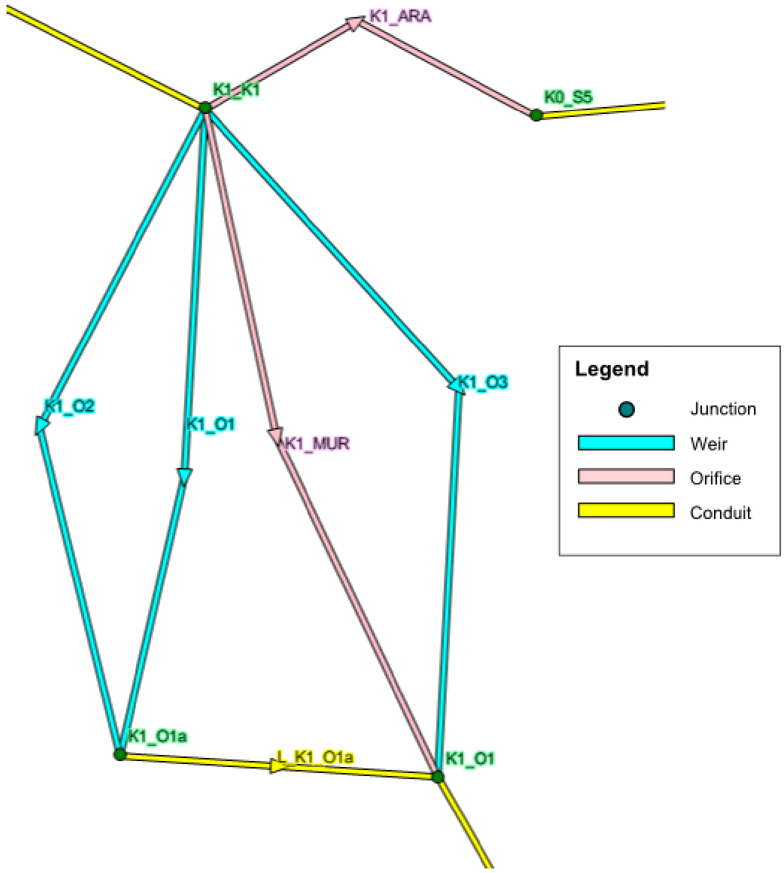


Figure 2-23: KS1 implemented in PCSWMM

2.1.5.3 Structure of the weir building KS2

Figure 2-24 shows a schematic of KS2. The weir is one of the moveable weirs described in chapter 0 with a weir height of 3.8 m. In case of a total fail of the weir, opening the two orifices on the right side of the schematic can surpass it. The sensors in front of the weir measure the flow velocity and the water depth and the sensor downstream of the weir measures the water depth.

Figure 2-25 shows the PCSWMM implementation of KS2. It can be seen that the surpassing structure was not implemented. That is because the weirs cannot fail in the actual model setup and therefore the structure was not necessary. K2 resembles the moveable weir and can have settings from 0 to 3.8 m.

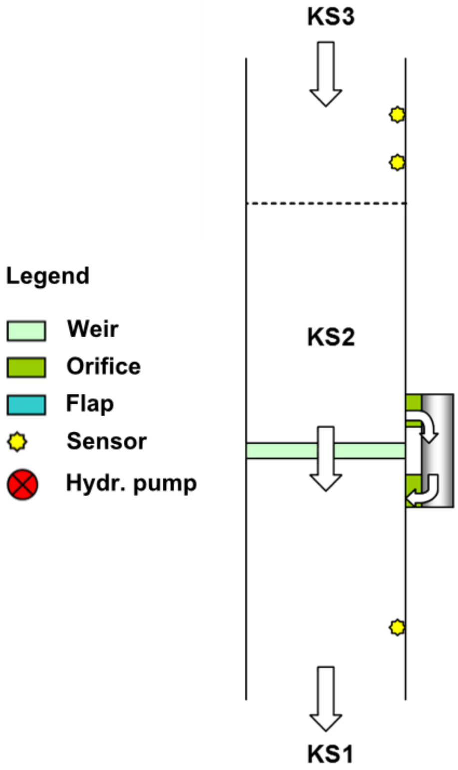


Figure 2-24: Schematic of KS2 structure (Holding-Graz, 2013 modified)

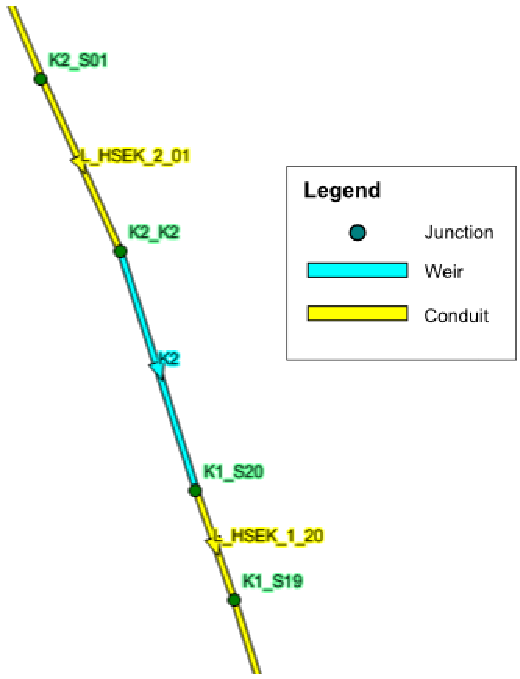


Figure 2-25: KS2 implemented in PCSWMM

2.1.5.4 Structure of the weir building KS3 including flushing chamber

Figure 2-26 shows a schematic of KS3. KSZ is representing the flushing chamber, which is separated from the Mur by an orifice. The sensors in this section are measuring the water depth. The weir and the surpassing structure are identical to the structure of KS1.

The implementation shown in Figure 2-27 is the same as the implementation of KS2 if it comes to the weir K3. The flushing chamber and the connection to the Mur are implemented differently than shown in the schematic in Figure 2-26. An outlet that is 3 m below water surface represents the Mur. That guarantees a constant flow from the Mur if needed. The pump P_S1_Spuelung regulates the filling of the flushing chamber with a maximum flow of 2 m³/s, which equals the flow of the actual facility. The conduits between pump and weir equal a volume of 400 m³ when filled, which is the same amount as the flushing chamber's capacity as constructed.

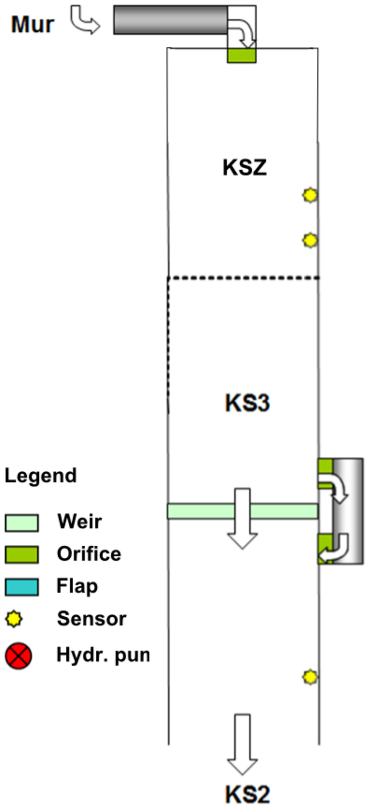


Figure 2-26: Schematic of KS3 structure (Holding-Graz, 2013 modified)

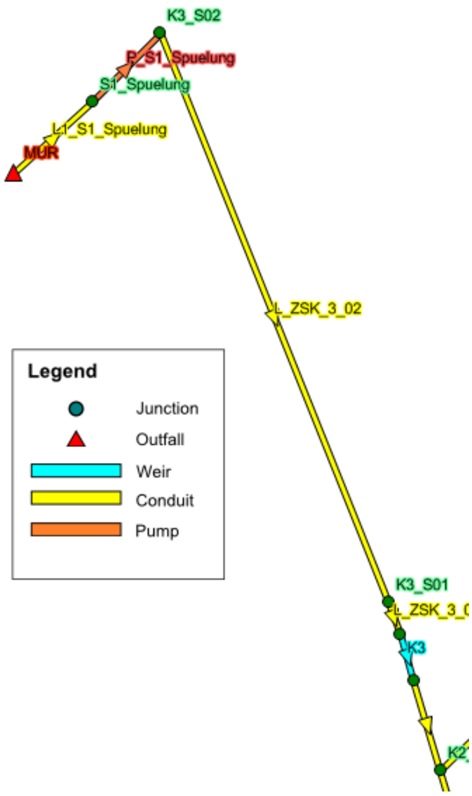


Figure 2-27: KS3 and flushing chamber implemented in PCSWMM

2.1.5.5 Calibration of the hydrodynamic runoff model

For the RTC control strategies during a storm event, an exact match of velocity and flow throughout the whole model of the ZSK is not essential. Nevertheless an adjustment as accurate as possible is required. Furthermore those factors are of significant importance when it comes to analyzing the flushing scenarios. Even though the scenarios' efficiencies are merely compared with each other to decide which one is to be used in the future, a better match to reality is always preferable. Also it was assumed that a wave's velocity is directly correlated to the cleaning capacity of a flushing wave (see chapter 1.3.7). That means that the calibration of the velocity is of higher importance than the calibration of the water depth.

In 2013, a master's thesis was conducted to examine the flushing mechanism of the ZSK. During that research, the depth and the velocity of flushing waves were measured at the weir KS2.

The following scenarios were used:

1. Flushing scenario 1 (single wave): The flushing chamber was filled completely. After the filling process, the orifice to the Mur was closed and the weir KS3 was lowered all the way to release a single wave towards the already opened weir KS2.

2. Flushing scenario 2 (multiple waves): The orifice to the Mur was opened to refill the chamber continuously. The weir KS3 was lowered with the following intervals in-between the lowering processes in minutes: 10-9-8-7-6-5-4. Again, the weir KS2 was already opened.

The measurement results gained from the single wave test from the thesis of 2013 were used to calibrate the model used in this thesis, whereas the multiple wave experiment was used to validate the calibration. With the alterations mentioned above already in place, the scenario of the research project was reproduced in the model and then simulated. To get more information on the conducted experiments read Golger (2014).

To match the values from the project, it was first tried to adjust the roughness of the tunnel. However, this only produced satisfying results with a roughness of $k_{st} = 53$ which equals the roughness of a gravel river basin. Considering that the tunnel was constructed with a smooth and even concrete surface, another approach had to be found. The roughness was corrected to its original value of $k_{st} = 93.46$. To reach the velocity of the measurement results, instead of the parameter used above, the average loss coefficient was adapted to produce the desired velocity loss effect. After running several simulations, while adjusting the loss coefficient, a value of 1.6 was found. Table 2-5 shows that the results of the calibration for a single wave with a deviation of only -2.96% for the wave's depth and a deviation of -0.54% for the wave's velocity proof an accurate calibration. The deviations of the multiple waves experiment are bigger, but considering the general uncertainty of a 1D hydrodynamic model, they are still in a plausible range.

Table 2-5: Absolute values for the calibration

Calibration scenario		Measurement	SWMM Model	Deviation
single wave	Depth	28,32	29,16	-2,96%
	Velocity	0,93	0,93	-0,54%
multiple waves	Depth	39,93	49,88	-24,90%
	Velocity	1,30	1,07	17,49%

The following figures show the time series of the comparison between measured values and the graphs produced by the model.

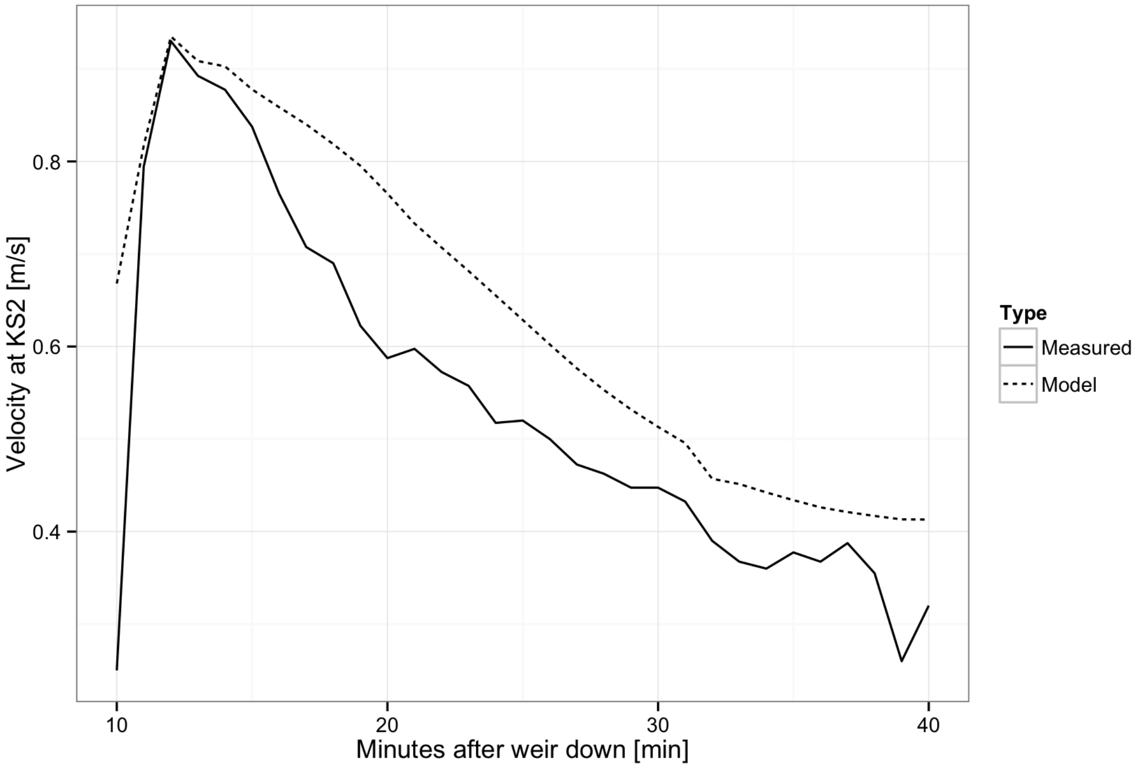


Figure 2-28: Calibration: single wave – velocity

In Figure 2-28 it can be seen that the velocity cannot be matched exactly to the falling branch of the curve. However, this result was the closest that could be achieved without further knowledge of what causes the losses. It is possible that even small amounts of residue on the floor of the tunnel produces disturbances. With a 1D hydrodynamic model, as was used in this thesis, those disturbances cannot be accounted for and therefore only an approximation to the real behavior can be achieved.

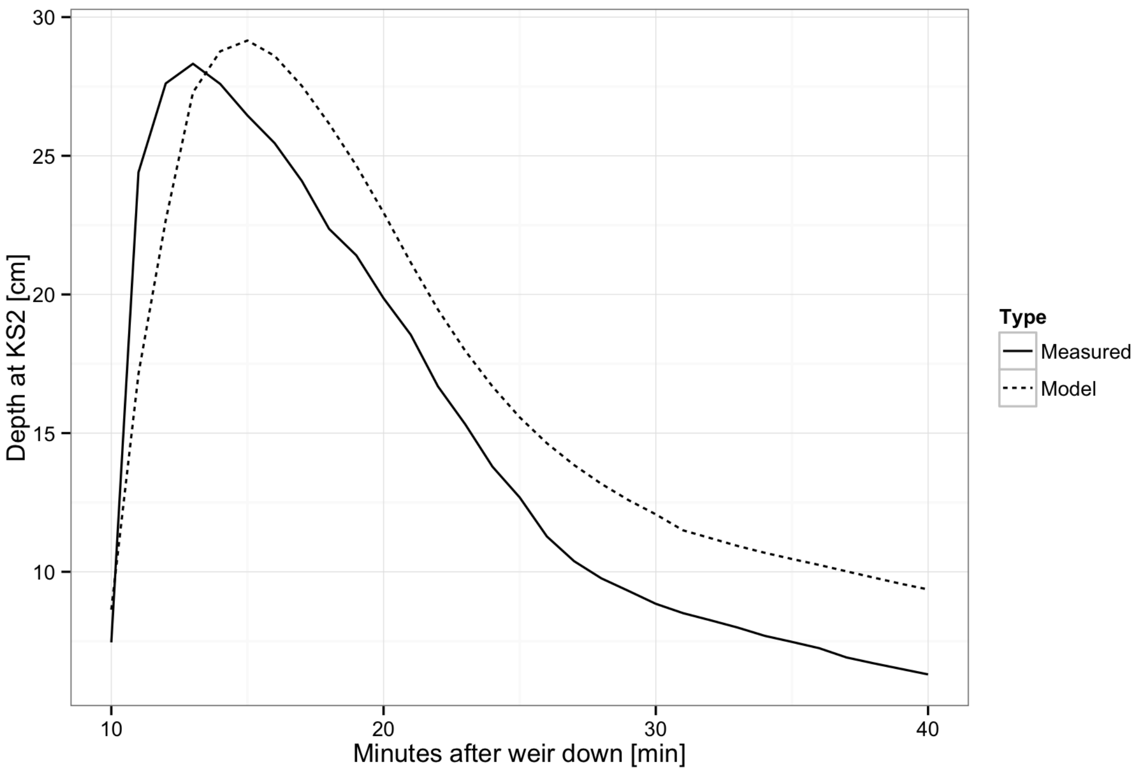


Figure 2-29: Calibration: single wave – depth

The wave peak also arrives a little bit later and again the falling branch is not quite matched. This might be caused by the effect of Poleni's equation being used in SWMM for calculating the flow over the weir. So what can be seen here is the backwater effect of the loss produced by the calculation approach. This is not a significant problem for a single wave, although accumulated, as seen later, the differences stand out more clearly.

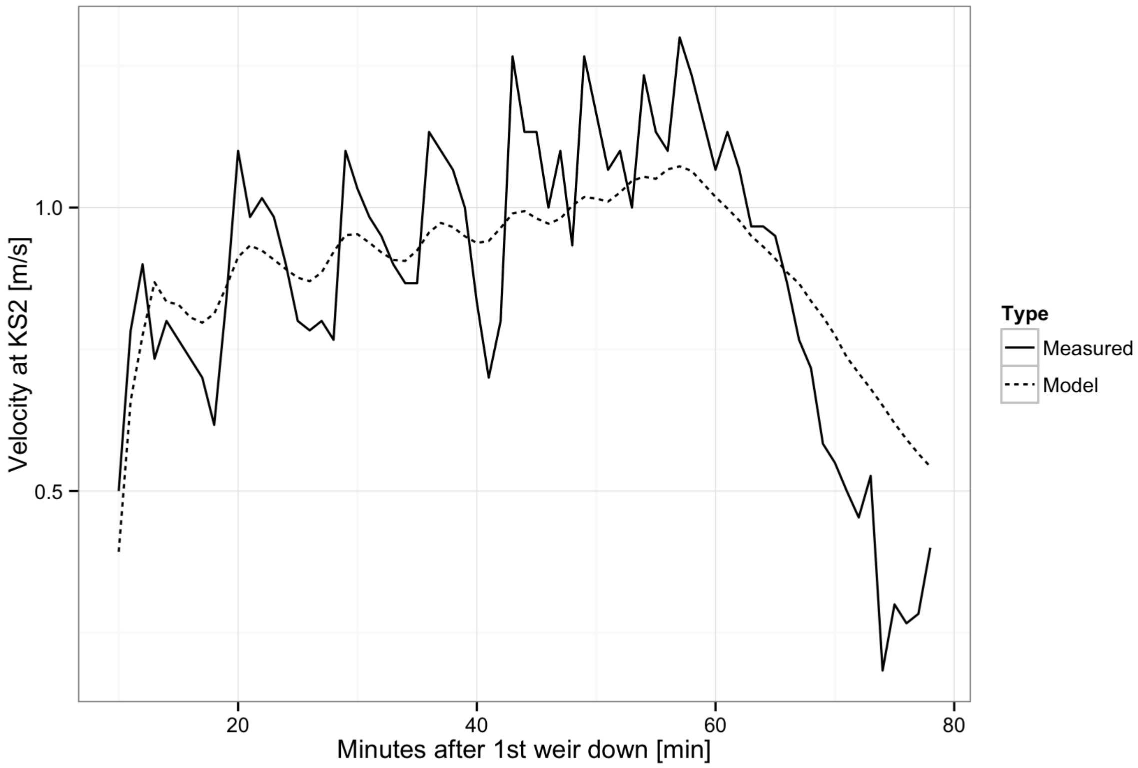


Figure 2-30: Calibration: multiple waves – velocity

In this scenario, the measurements clearly did not show the actual waves. Disturbances, foam or other effects may have produced these high amplitudes. The behavior of the model seems more realistic than the actual measurements. Nevertheless, the general trend is shown in Figure 2-30 and appears to be correct.

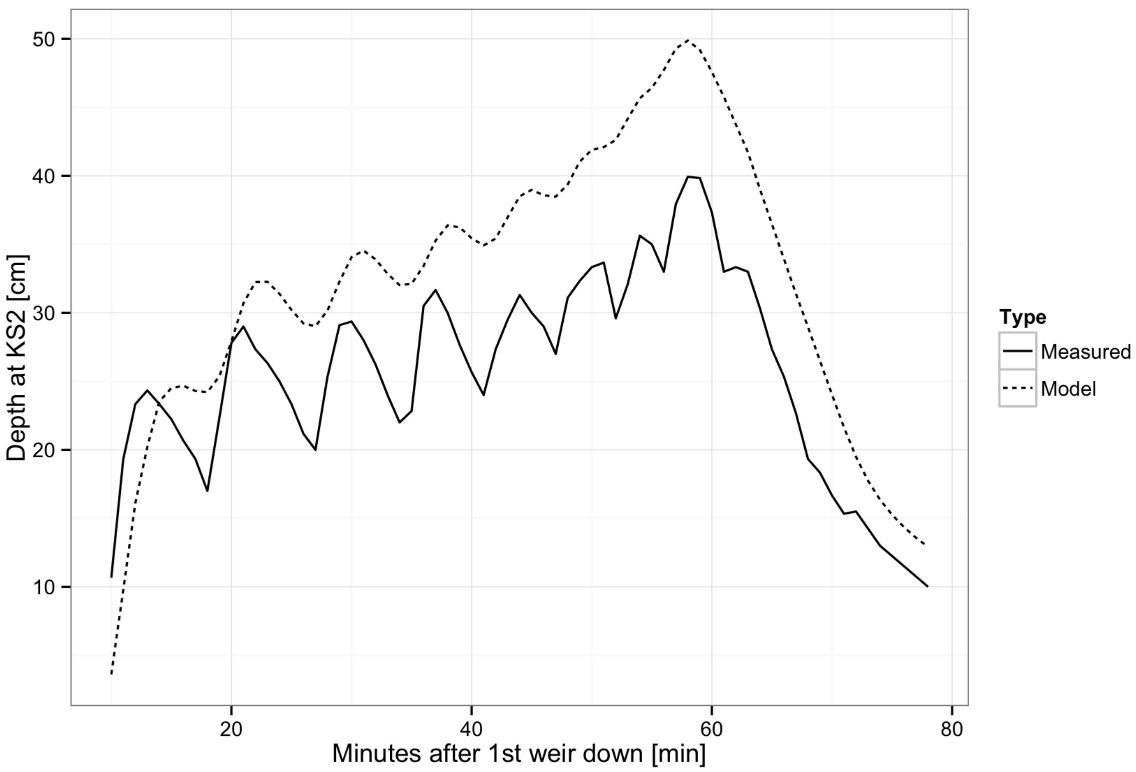


Figure 2-31: Calibration: multiple waves – depth

Here, the backwater effect already seen in the single wave scenario stands out more obviously.

2.1.5.6 Limitations of the hydrodynamic runoff model

As this model was implement as a 1D hydrodynamic model, processes like turbulence or the exact behavior of water flowing over the weir cannot be represented (as explained in chapter 1.3.8.1). That means that, for example, the situation right behind the weir is not correctly reproduced. To do that, a multidimensional hydrodynamic model in combination with a turbulence model would be needed.

Furthermore, the exact causes of the losses in the collector tunnel were unknown, so only an approximation could be made to match the measurements used for calibration.

And finally, the model can only be as accurate as the data used for calibration. Unfortunately, the research project used, only measured in one place, so the parameters used to approximate the model had to be applied for the whole collector tunnel even though it is not certain whether this assumption is correct or not.

2.2 Description of RTC modeling scenarios

This chapter defines the different strategies used to empty and flush the collector tunnel. It also covers the setup of the various strategies. Lastly, there is a description of how the MPC framework was validated and how it was initialized.

2.2.1 Emptying and flushing scenarios in the ZSK

The first task was to find an efficient strategy to empty and flush the collector tunnel after a storm event. After the model was modified and calibrated, the current strategy used by Holding Graz was reproduced to generate a reference scenario that the newly developed schemes could be compared to later. Then various initial situations were created with different initial depths set for the separate parts of the ZSK. Then, with the generated strategies, a way to efficiently compare the scenarios had to be found. Cumulative graphs were used to show the flow velocities over time, based on the assumption that the cleaning effect directly correlates with the velocity of a wave.

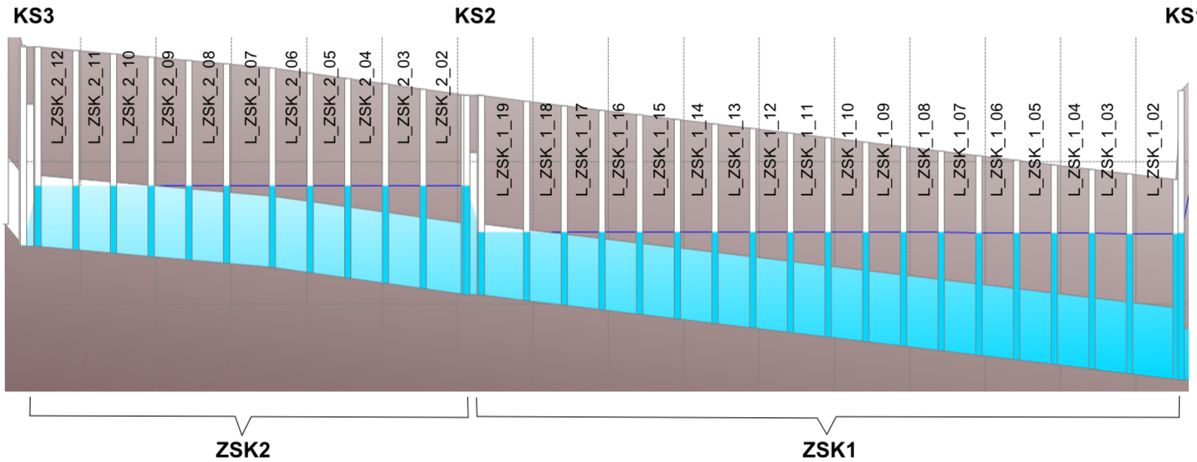


Figure 2-32: Section view of the filled ZSK with the section names used for the velocity distribution plots

2.2.1.1 Implementing initialization statuses for the ZSK

To produce different initial statuses for the collector tunnel, six stages were implemented to simulate the emptying scenarios and one for flushing scenarios:

Emptying:

- ZSK1 and ZSK2 are filled completely
- ZSK1 and ZSK2 are filled halfway
- ZSK1 is filled completely while ZSK2 is empty
- ZSK1 is filled halfway while ZSK2 is empty
- ZSK2 is filled completely while ZSK1 is empty
- ZSK2 is filled halfway while ZSK1 is empty

Flushing:

- ZSK1 and ZSK2 are empty

To implement these initial stages during the simulation there are two options. First, it is possible to simply fill up the collector tunnel at the beginning of each simulation from the inlet to the Mur and control the weirs so that after the needed time the initial depths are reached. The downside of this approach is that the exact time of the filling process is unknown and it would take a couple of control rules just to get to that state.

Therefore, the second solution was implemented. SWMM provides the option to set an initial depth to each node. Therefore, knowing the slope, the water level of each node could easily be calculated. With automated scripts those values were then simply replaced in the SWMM input file before the simulation was started. The result for the ZSK (from KS1 to KS3) being completely filled is shown in Figure 2-32.

2.2.1.2 Optimization parameters

As the model is implemented in SWMM, a 1D hydrodynamic simulation model, effects of turbulence or shear cannot be considered to determine the cleaning efficiency of each scenario. Therefore it was assumed that the velocity of a wave correlates with its cleaning efficiency. So to optimize the emptying and flushing strategies, the highest possible flow velocity had to be found. As described in chapter 1.3.7 the critical locations for deposits are in front of the weirs. So the scenarios were optimized to produce the highest possible velocities in front of KS1 and KS2. As already mentioned above, this velocity is presented with cumulative graphs. For better resolution, the model was separated into equidistant sections (see Figure 2-32), each of which produced one of the graphs to give an accurate enough picture of the velocity distribution over the whole tunnel.

2.2.1.3 Scenarios

In the process of finding efficient scenarios it was soon discovered that the bigger the wave is, the higher the velocity gets. The connection between the wave's depth and its velocity can also be observed when looking at Figure 2-30 and Figure 2-31. So the following three scenarios where implemented and then tested.

Emptying Scenarios

- Reference scenario

  This scenario is currently used by Holding Graz to empty the ZSK after a rainfall event. So far, the main goal is to continuously supply the WWTP with a constant flow of 0.6 m^3/s. First the ZSK1 is emptied down to 3 m at KS1. When this limit is reached, the weir KS2 is lowered continuously to produce a flow of 0.6 m^3/s into ZSK1. Because this flow cannot be guaranteed, there is also an upper limit of 5 m at KS1. When it is reached, KS2 is closed again. This process is continued until the whole ZSK is empty.

  - Reference scenario full (RF)
  - Reference scenario half full (RHF)
  - Reference scenario ZSK1 full ZSK 2 empty (RZSK1F)
  - Reference scenario ZSK1 half full ZSK 2 empty (RZSK1HF)
  - Reference scenario ZSK1 empty ZSK 2 full (RZSK2F)
  - Reference scenario ZSK1 empty ZSK 2 half full (RZSK2HF)

- Quick refill scenario

  This scenario guarantees the 0.6 m^3/s as well. Although instead of slowly lowering the weir KS2 to produce a steady flow, it is lowered quickly and only raised when the limit of 5 m at KS1 is reached again.

  - Quick refill scenario full (QF)
  - Quick refill scenario half full (QHF)
  - Quick refill scenario ZSK1 full ZSK 2 empty (QZSK1F)
  - Quick refill scenario ZSK1 half full ZSK 2 empty (QZSK1HF)
  - Quick refill scenario ZSK1 empty ZSK 2 full (QZSK2F)
  - Quick refill scenario ZSK1 empty ZSK 2 half full (QZSK2HF)

- Refill-on-empty scenario

  This scenario does not guarantee the 0.6 m^3/s throughout the whole emptying process. However it allows a constant flow towards the WWTP of 0.6 m^3/s while KS2 is up and the water in ZSK2 is still stored. When ZSK1 is emptied and the flow towards the WWTP falls below 0.59 m^3/s, KS2 is lowered and all the water stored in ZSK2 creates a massive wave to flush the lower ZSK1.

  - Refill-on-empty scenario full (RF)
  - Refill-on-empty scenario half full (RHF)
  - Refill-on-empty scenario ZSK1 full ZSK 2 empty (RZSK1F)
  - Refill-on-empty scenario ZSK1 half full ZSK 2 empty (RZSK1HF)
  - Refill-on-empty scenario ZSK1 empty ZSK 2 full (RZSK2F)
  - Refill-on-empty scenario ZSK1 empty ZSK 2 half full (RZSK2HF)

Flushing scenarios

The flushing chamber installed in the ZSK uses fresh river water from the Mur to flush the collector tunnel to reduce deposits left by a storage event. However the tunnel is emptied, some deposits will always stay behind. So it is necessary to flush the collector tunnel after emptying it. Nevertheless it should be kept in mind that the used water creates extra volume that needs to be treated in the WWTP. The water from the Mur contains sediments too, however the pollution of it is far less intense then in the stormwater runoff. The consequence of that is the dilution of the wastewater by the cleaner water from the river, which can result in worse treatment efficiency (decay rate in %) at the WWTP. That means the less water used for flushing the better. With that in mind the following scenarios were implemented and compared:

- One single wave (1W)
- Two consecutive waves (2W)
- Three consecutive waves (3W)
- Three consecutive waves with six-minute intervals (3W6M; currently used by Holding Graz)
- Three consecutive waves with eight- and six-minute intervals (3W8M6M)
- Three consecutive waves with ten- and six-minute intervals (3W10M6M)
- Two consecutive waves intercepted at KS2 and then released together (2WSt)
- Three consecutive waves intercepted at KS2 and then released together (3WSt)

- Four consecutive waves intercepted at KS2 and then released together (4WSt)
- Two consecutive waves intercepted at KS2, then released together and caught by a third wave right before KS1 (2WSt1W)

2.2.2 Control scenarios for RTC in the ZSK

The RTC strategy takes control over the ZSK 24 hours a day seven days a week. The danger of CSOs only occurs during the actual storm event and therefore is handled separately from emptying and flushing, which happens after a storm, in this thesis. The strategies themselves aim to minimize the total CSO volume during storm events.

2.2.2.1 Definition of optimization parameters

To find the optimal RTC strategy, the optimization parameters have to be set precisely. There are different ways to optimize those parameters as described in chapter 1.3.4. Also, regulators like weirs, orifices and pumps can be used more efficiently or be treated with more care to expand their lifespan. Throughout this project, the focus was set on minimizing the total CSO volume during a storm. More sophisticated approaches were not applied because it is only the first case study to show the potential for RTC in the system.

2.2.2.2 Description of the applied strategies

As described in chapter 1.3.3 there are different ways to implement RTC. In this thesis, rule based RTC and MPC were chosen to improve the system's efficiency.

Reference scenario

Currently the ZSK reacts to a storm event by raising all its weirs and storing as much water as possible. The runoff that exceeds the storage volume creates a CSO. After the storm ceases, the emptying process described in chapter 2.2.1.3 is initiated manually.

Rule based strategy

The rules in this strategy were found during various simulations and aim to minimize the CSO volume by routing as much runoff as possible to the WWTP so that during a storm, the WWTPs maximum treatment capacity can be activated. To do that, only the orifice K0_EINL has to be controlled. The water depth sensors at KS1 and the CSO basin at the WWTP deliver the information needed to enact the control rules. K0_EINL opens when the water level in the collector tunnel surpasses 4.79 m and closes again when the level there falls below that limit again or when the water level in the CSO basin surpasses 4.6 m. These limits guarantee that the storage volume of the collector tunnel and the basin are used most efficiently and that CSOs only occur when the full storage capacity of both facilities is already reached. In addition, the weir KS3 that separates the flushing chamber from the rest of the ZSK is initially lowered, so that it can hold some of the backwater stored in ZSK2.

MPC strategy

To implement the MPC strategy, a framework was needed to handle the optimization. The institute of urban water management and landscape water engineering of Graz University of Technology is currently developing such a framework. This framework is a working prototype and only had to be modified to fit the needs of this thesis. It is implemented in Python, an open source project and programming language (https://www.python.org/) and uses the DEAP toolbox, a framework implementing evolutionary computation (http://deap.reathedocs.org/en/latest/index.html) for optimization, which utilizes genetic algorithms to optimize problems regarding a fitness function. It also uses SCOOP, a distributed task module allowing concurrent parallel programming (https://code.google.com/p/scoop/) to parallelize the calculation process to reduce the computing time.

Because this thesis only focuses on the model implementation, the exact functionality of the used framework will not be discussed here.

Figure 2-33 shows an overview of the more important optimization methods. All of them have in common that they have an objective function. This objective function specifies the optimization parameters and can either be a single-objective function with only one ultimate goal to consider during the optimization or a multi-objective function that takes multiple goals into consideration. The two main groups of optimization algorithms are local and global optimization. The difference between them is that local optimization can only find a local optimum (see Figure 2-34), which means that it cannot consider the whole range of a function, whereas global optimization works with metaheuristic methods that can find a global optimum as well. A more detailed description of this topic can be found in Kirkpatrick *et al.* (1983); Goldberg and Richardson (1987); Mitchell *et al.* (1993); Großmann and Terno (1997).

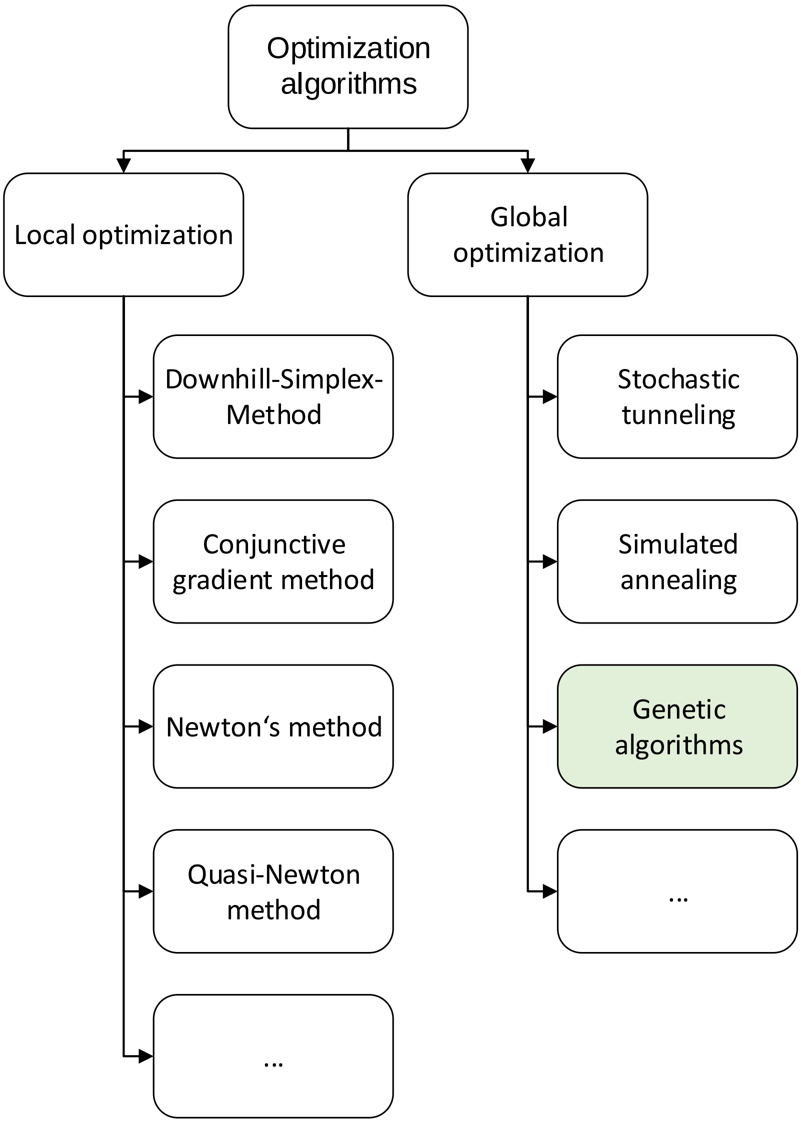


Figure 2-33: Overview over optimization methods (local and global)

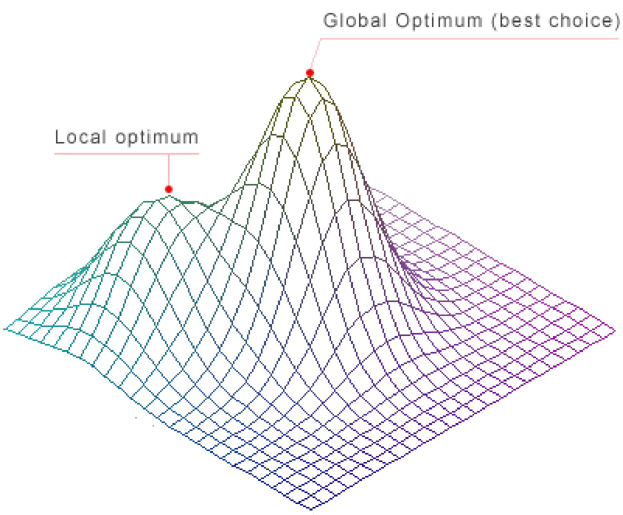


Figure 2-34: Description of a local and global optimum (Barcomb, 2012)

The method of genetic algorithms has been used to implement the applied MPC system and is therefore explained here in more detail to understand the general process.

Genetic algorithms

This process borrows from the theory of evolution and uses the processes of mating, mutation and selection to find an optimal solution for a problem following a predefined fitness function. It should be mentioned that this approach is metaheuristic. That means that every run of the algorithm usually delivers different outcomes for the same problem. Figure 2-35 gives an overview of the processes of genetic algorithms.

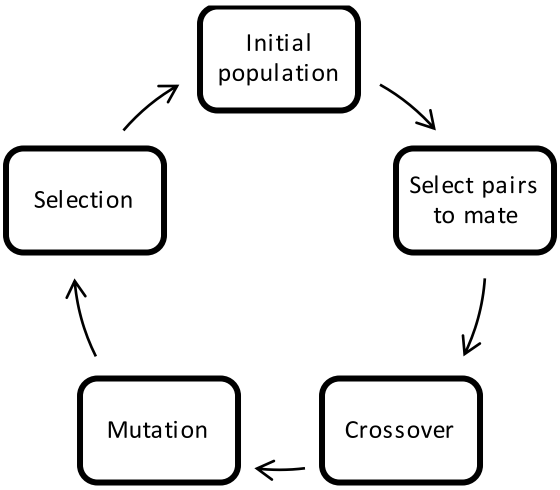


Figure 2-35: Overview of process loop in genetic algorithms

- Initial population

  In the beginning, a random population (initial values) is set up concurring to a population size. This reflects the first generation of solutions.

- Select pairs to mate

  Following a crossover probability, individuals are selected to mate.

- Crossover

  The parents mate and exchange their genes following an earlier chosen algorithm.

- Mutation

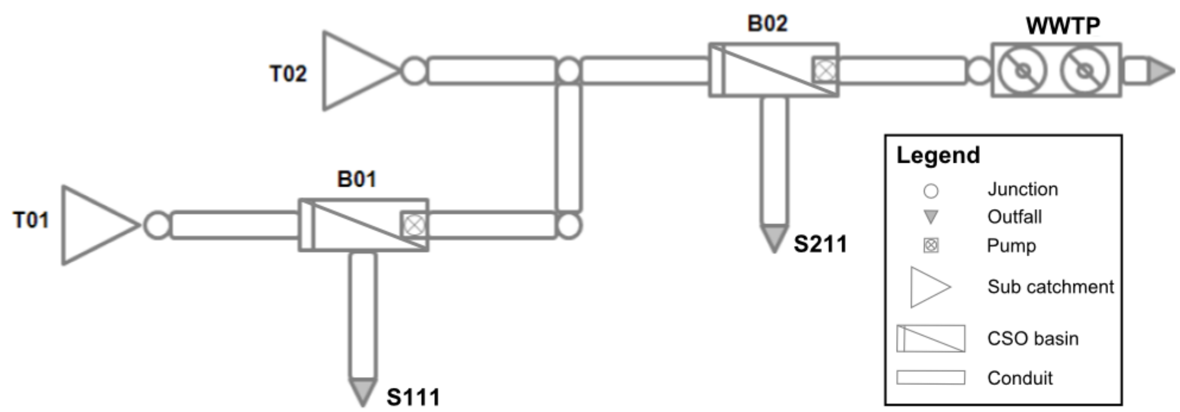
  To keep variety up, some solutions are chosen to mutate following a mutation probability.

- Selection

  All individuals are tested with the fitness function to find the best ones to enter the next generation.

This process is repeated for a predefined number of times or until the fitness function finds the best possible solution (e.g. no CSO). More detailed information can be found in Muschalla (2006).

However, the framework needed to be tested to make sure it delivered a valuable result for the given problem. To do so, the dissertation of Heusch (2011) was used. In his thesis, a simple network was created according to the DWA M180 guideline document (DWA, 2005; for more information on the DWA M180 guideline read Schütze *et al.* (2008)) as an experimental area to test an MPC strategy with the goal of minimizing the total overflow volume. The same network was used under the same conditions to run our MPC to compare the results afterwards. The time settings used for both implementations consist of evaluation (4h), prediction (4h) and control horizon (1h) and a control step of 10 min (further explanation in Heusch (2011)). The network used for the simulation is shown in Figure 2-36.



With: B01 & B02 … CSO basins to be used efficiently; S111 & S211 … outfalls for the optimization

Figure 2-36: Schematic of the M180 guideline network (Heusch, 2011 modified)

The results of the comparison show that the MPC system developed by Graz University of Technology works. The minor deviation of only 1.7% (calculated from the results in Table 2-6) from the MPC system used by Heusch can be explained due to the use of different algorithms, which are both heuristic. However, both implementations show an equally promising result compared to the uncontrolled reference scenario of about 40%. The MPC strategies also show big differences between the two overflow structures, which strengthens the theory that the used algorithms work differently.

Table 2-6: Comparison between the two MPC implementations

Outfall	No RTC	MPC Heusch	MPC Graz
S111	422.5 m³	60 m³	338.1 m³
S211	993.4 m³	783 m³	519.1 m³
Total	1415.9 m³	843 m³	857.2 m³

The time settings used in the ZSK simulation are basically the same except for the control step, which was reduced to 5 minutes. This was done because the inflow data also had a time interval of 5 minutes, so the resolution didn’t have to be downscaled.

In the genetic algorithm, the population size was set to 50 as well as the number of generations. Furthermore, the evolution was interrupted when the fitness function was fulfilled, meaning no CSO occurred for the current time step. More specifically the optimization, chosen for the MPC strategy, was to minimize the total CSO volume, the same as for the rule based RTC strategy.

3 Results and Discussion

This chapter presents the findings from the comparison between the emptying and flushing scenarios to the present reference strategy. Furthermore, it shows the results of the RTC strategies for storm events with three different return periods compared to the current control strategy.

3.1 Emptying and flushing scenarios in the ZSK

In order to show the results of the emptying scenarios only the graphs with full collector tunnels as their initial status will be shown (see chapter 2.2.1.3). Graphs for other initial statuses can be found in the appendix. The scenarios presented in this chapter are marked in green in Table 3-1. Also, the absolute results, representing the highest velocities achieved in every tested scenario, are listed in Table 3-2. During the entire evaluation, speeds of less than 0.1 m/s were ignored, as they didn’t have any impact on the cleaning efficiency.

Table 3-1: Overview over the emptying and flushing scenarios (marked scenarios will be presented in this chapter)

Goal	Description	Name
Emptying Scenarios	Reference scenario FULL	RF
	Reference scenario HALFFULL	RHF
	Reference scenario ZSK1 FULL	RZSK1F
	Reference scenario ZSK1 HALFFULL	RZSK1HF
	Reference scenario ZSK2 FULL	RZSK2F
	Reference scenario ZSK2 HALFFULL	RZSK2HF
	Quick refill scenario FULL	QF
	Quick refill scenario HALFFULL	QHF
	Quick refill scenario ZSK1 FULL	QZSK1F
	Quick refill scenario ZSK1 HALFFULL	QZSK1HF
	Quick refill scenario ZSK2 FULL	QZSK2F
	Quick refill scenario ZSK2 HALFFULL	QZSK2HF
	Refill-on-empty scenario FULL	EF
	Refill-on-empty scenario HALFFULL	EHF
	Refill-on-empty scenario ZSK1 FULL	EZSK1F
	Refill-on-empty scenario ZSK1 HALFFULL	EZSK1HF
	Refill-on-empty scenario ZSK2 FULL	EZSK2F
	Refill-on-empty scenario ZSK2 HALFFULL	EZSK2HF
Flushing Scenarios	1 single wave	1W
	2 consecutive waves	2W
	3 consecutive waves	3W
	3 consecutive waves with 6-minute intervals	3W6M
	3 consecutive waves with 8- and 6-minute intervals	3W8M6M
	3 consecutive waves with 10- and 6-minute intervals	3W10M6M
	2 consecutive waves Stored in ZSK2 and then released together	2WSt
	3 consecutive waves Stored in ZSK2 and then released together	3WSt
	4 consecutive waves Stored in ZSK2 and then released together	4WSt
	2 consecutive waves Stored in ZSK2, released together and caught by a 3rd right before K1	2WSt1W

3.1.1 Reference

As would be expected, the currently used scenario fails to produce velocities high enough to clean the tunnel because of the original goal to continuously supply the WWTP with the same flow instead of producing the highest possible flow velocity in the tunnel. This means that during emptying, no wave is generated to produce any cleaning effects. The next two pages show the plots for the reference scenario. Figure 3-1 shows the ZSK2. The highest velocity in this fragment occurs closest to the weir KS2, showing the overflow of the weir. Figure 3-2 represents ZSK1 of the tunnel. Here again the highest velocities happen close to the weir KS2, also produced by the weir overflow. It should be noted that the scale of the x-axis is different for both figures and for all the other velocity distributions due to the legibility. The significant areas in the tunnel are situated directly before the weirs because it is assumed that that is where the most deposits will remain after a storage event (see chapter 1.3.7). A velocity of 0.5 m/s right before the weir KS1 seems rather low compared to, for example, the results from the calibration experiment from chapter 2.1.5.5.

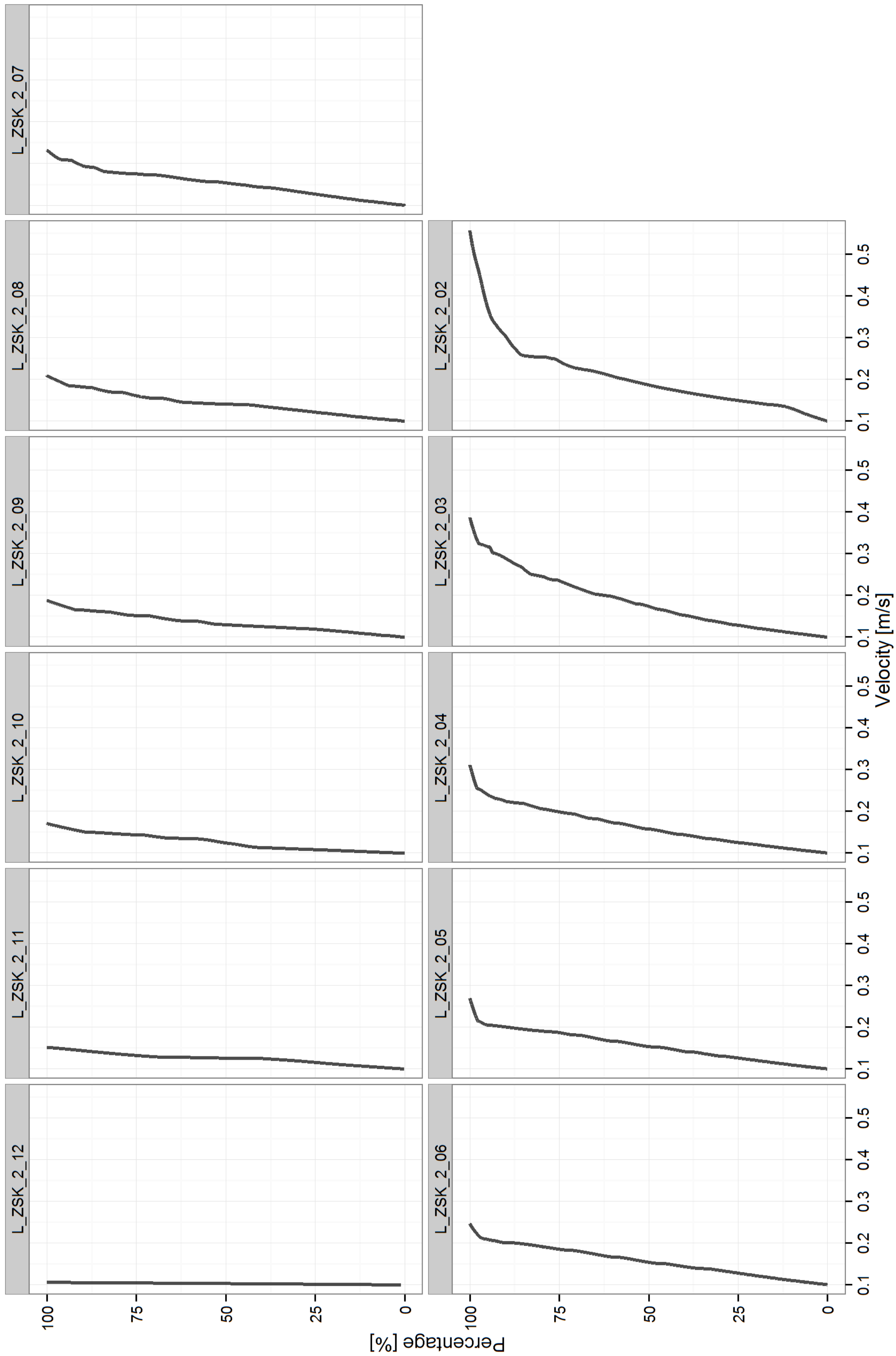


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Figure 3-1: Emptying: reference scenario – ZSK2

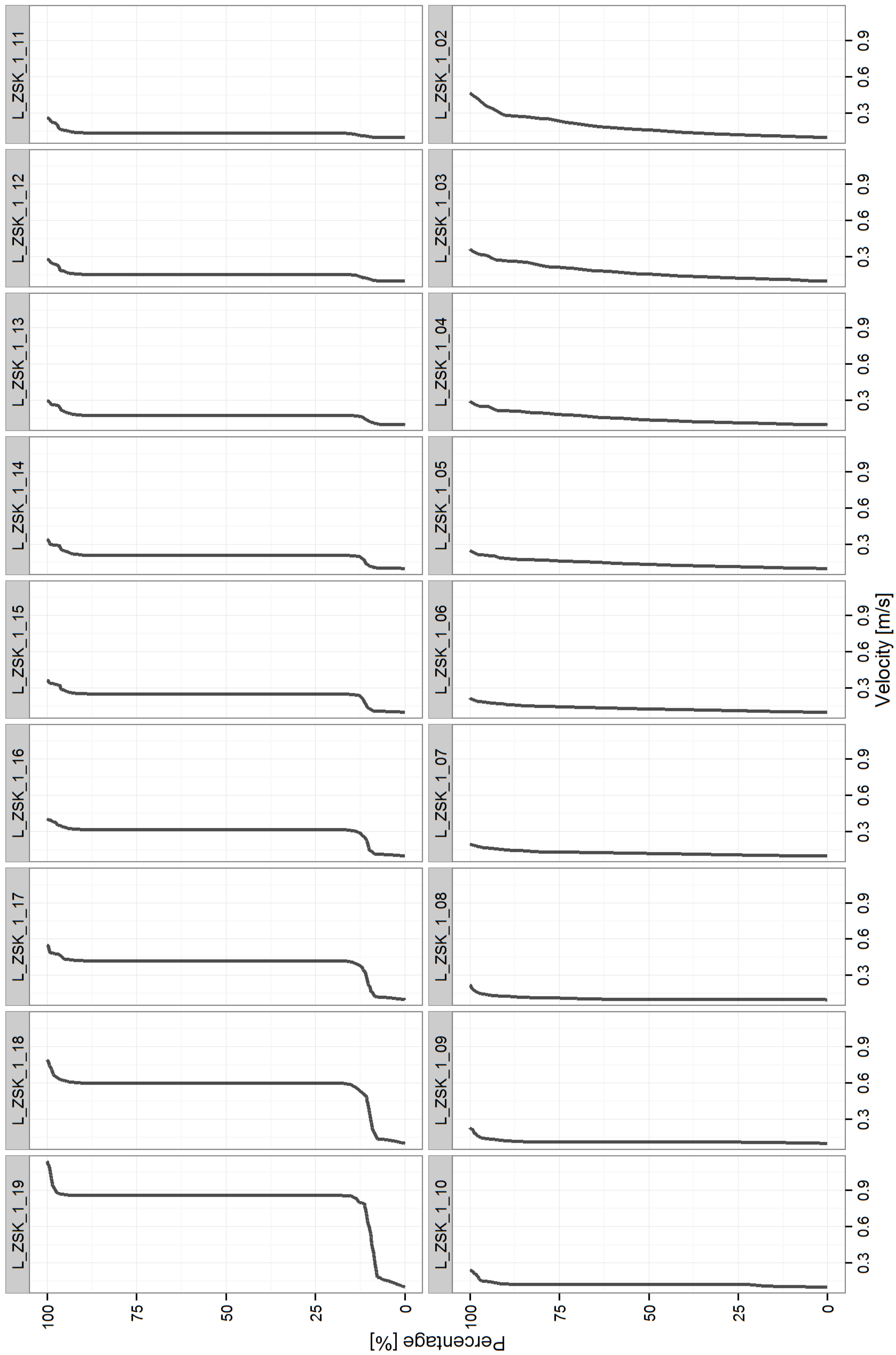

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Figure 3-2: Emptying: reference scenario – ZSK1

3.1.2 Quick refill

This scenario also produces a higher velocity the closer it gets to the weir KS2. In this case however, it goes up to 1.8 m/s in ZSK2 and up to 2.7 m/s right after the weir. The velocities cannot get much higher in ZSK2 while emptying the collector tunnel because there is no water that can be used to flush it. The only cleaning effect can come from the water stored within ZSK2 itself. The low velocities at the bottom (at weir KS1) are caused by the flow limitation of 0.6 m^3/s. The backwater effect that happens there breaks down the waves coming from the weir KS2 when it is lowered to refill ZSK1 to its upper limit. Figure 3-3 shows the velocity distribution of ZSK2 of the tunnel and Figure 3-4 the distribution of the ZSK1.

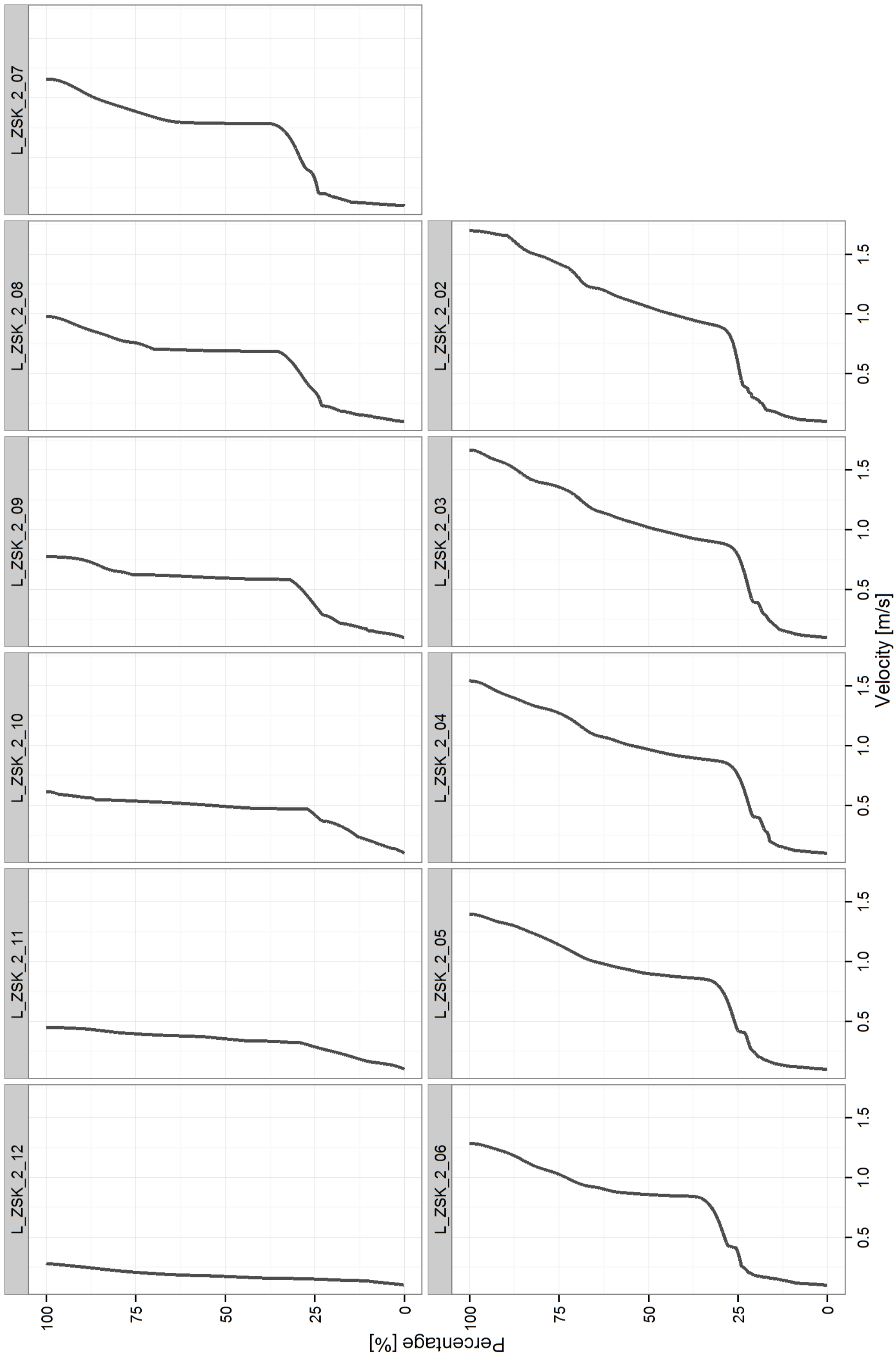

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Figure 3-3: Empting: quick refill scenario – ZSK2

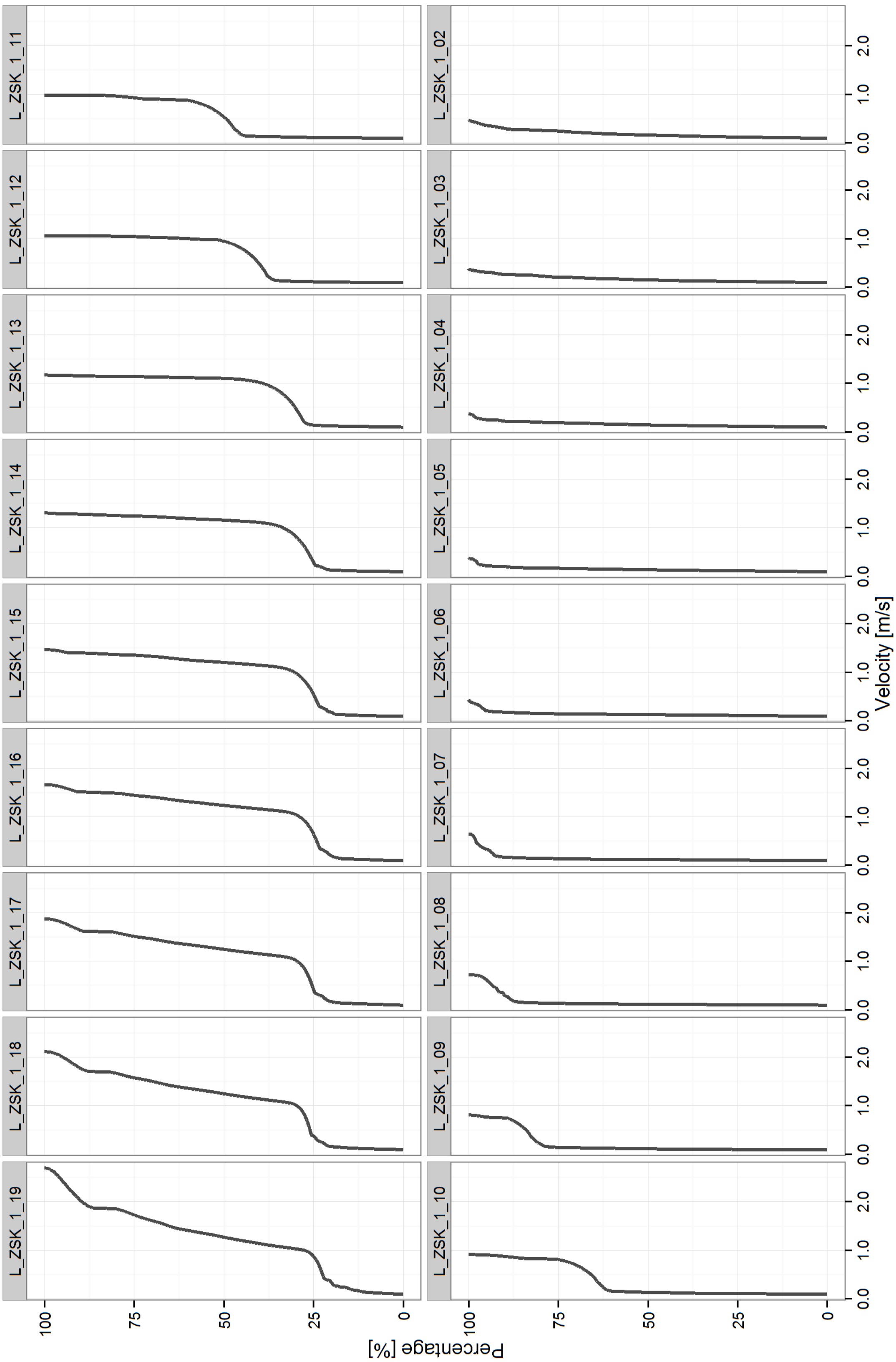

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Figure 3-4: Emptying: quick refill scenario – ZSK1

3.1.3 Refill on empty

To counteract the backwater effect at KS1 produced in the quick refill scenario, the water was emptied completely from ZSK1 before the stored water from above the weir KS2 was released into it. The outcome shows that the strategy works. A velocity of 1.5 m/s is the highest reached during the entire test. That means that the larger the amount of water that is released from ZSK2 is, the faster the wave arrives at the weir KS1. This scenario was tried by Holding Graz during some of their own tests and as a result, the immense air pressure built up by the produced wave lifted the manhole covers up. It had been assumed that the ventilation capacity, provided by the facility KS1, was enough to prevent that from happening, but as it turns out it was not. So before such a strategy is considered again, the manholes have to be bolted down first to prevent them from taking off again. Figure 3-5 and Figure 3-6 show the simulation results for this approach.

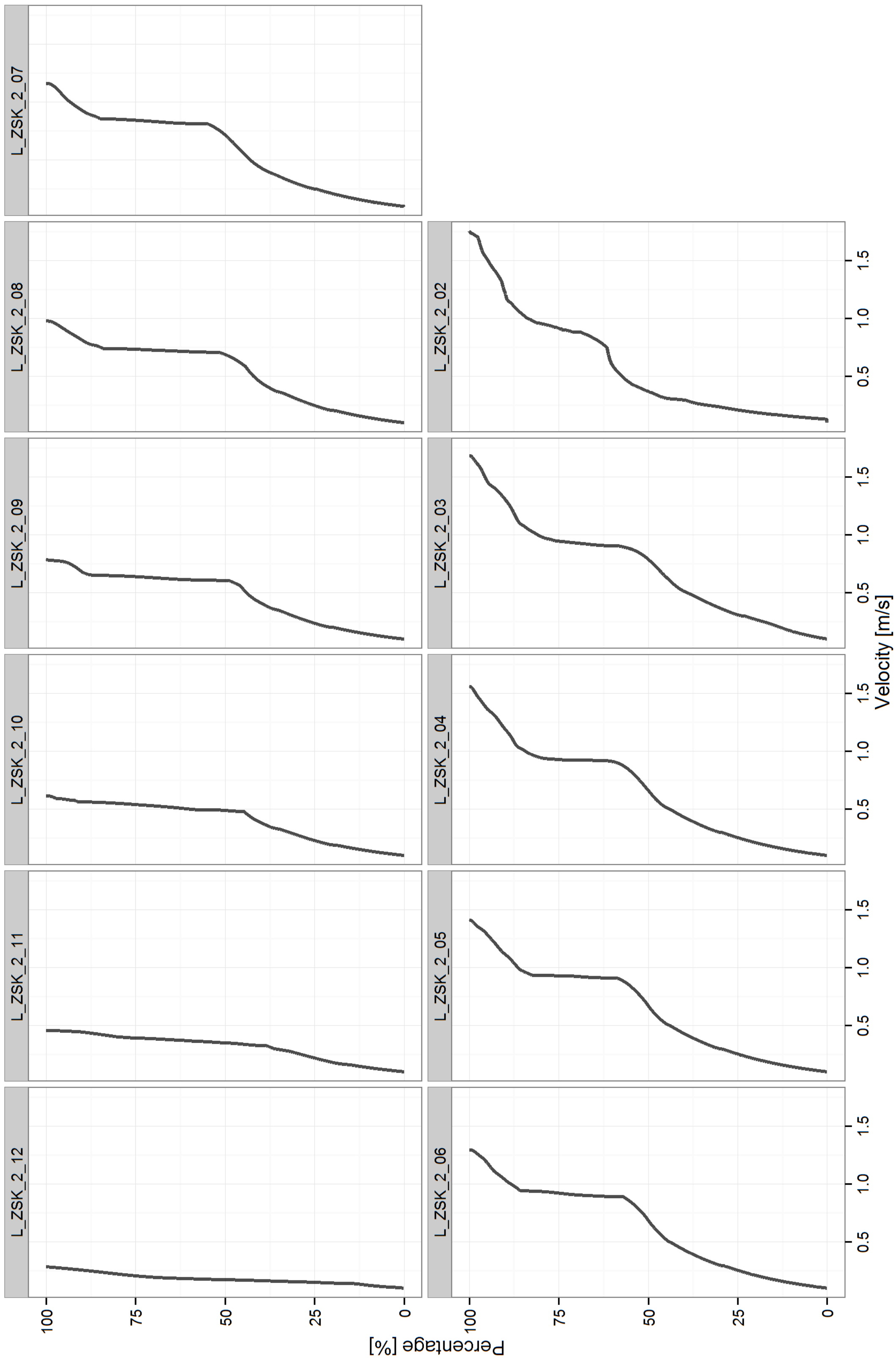

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Figure 3-5: Emptying: refill on empty scenario – ZSK2

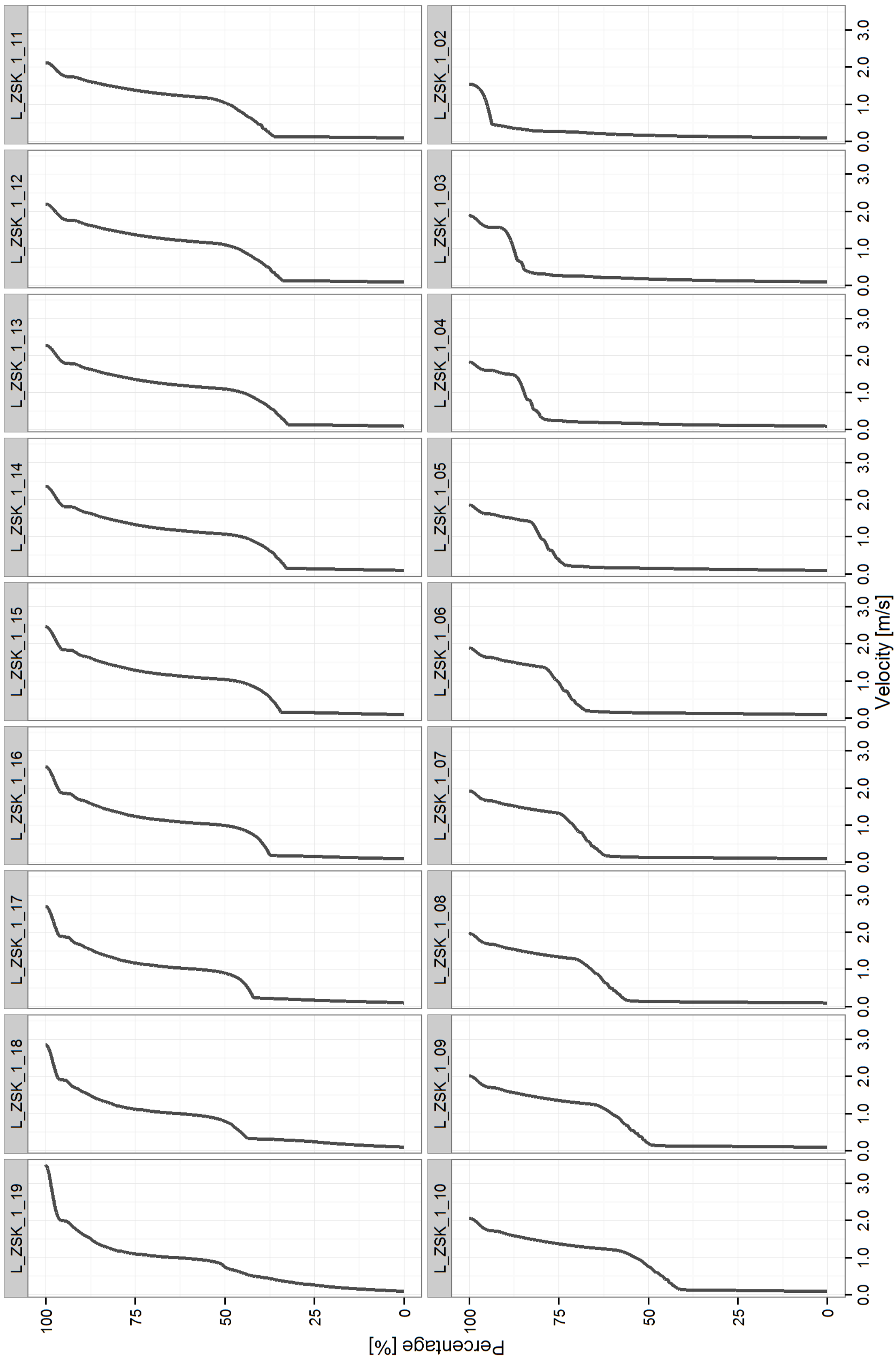

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Figure 3-6: Emptying: refill on empty scenario – ZSK1

3.1.4 Flushing Scenarios

The various flushing scenarios tested during the simulations showed that only the volume makes an actual difference concerning speed. This suggests that the bigger the wave is, the better the cleaning effect. One wave catching up to another one, forming a combined wave head and also generating turbulence on the ground cannot be replicated in a 1D hydrodynamic model.

Only the scenario that is currently in place (3W6M) and the best scenario found (4WSt) during the simulations will be discussed here. The results of the other schemes can be found in the appendix.

Reference with 3 waves with 6-minute intervals

This flushing scheme simply sends 3 waves with 6-minute intervals down the collector tunnel without intercepting them. The waves catch up to each other on the way but, because the model does not consider turbulence and ground shear, the waves flatten on the way and the acceleration of one wave catching up to another and transferring some of its speed onto the other is almost eliminated by that. At the two significant locations before the weirs KS2 and KS1, a wave speed of 1.1 m/s is reached. Figure 3-7 and Figure 3-8 below show the speed distribution in the collector tunnel for this scenario.

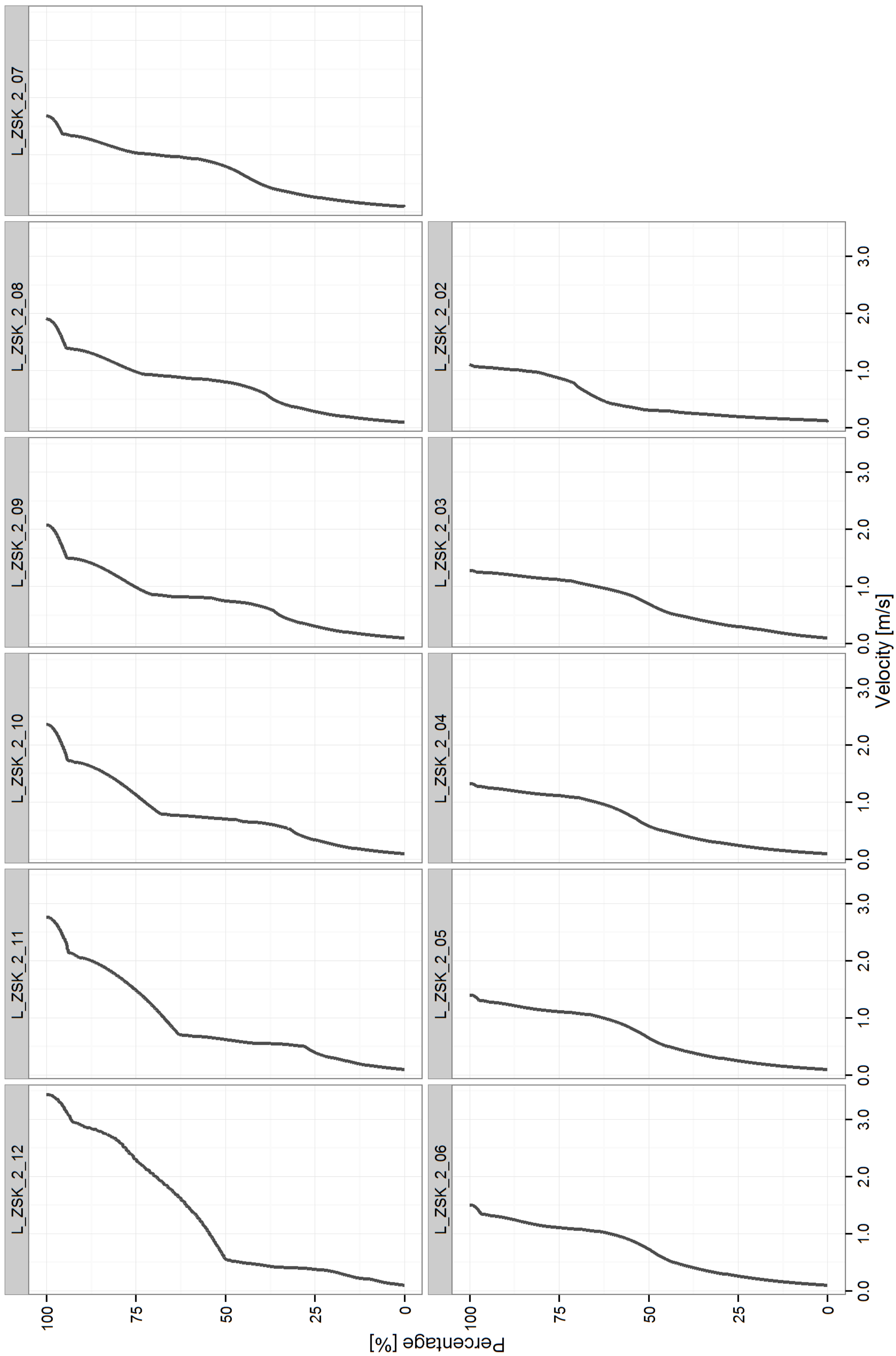

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Figure 3-7: Flushing: 3W6M – ZSK2

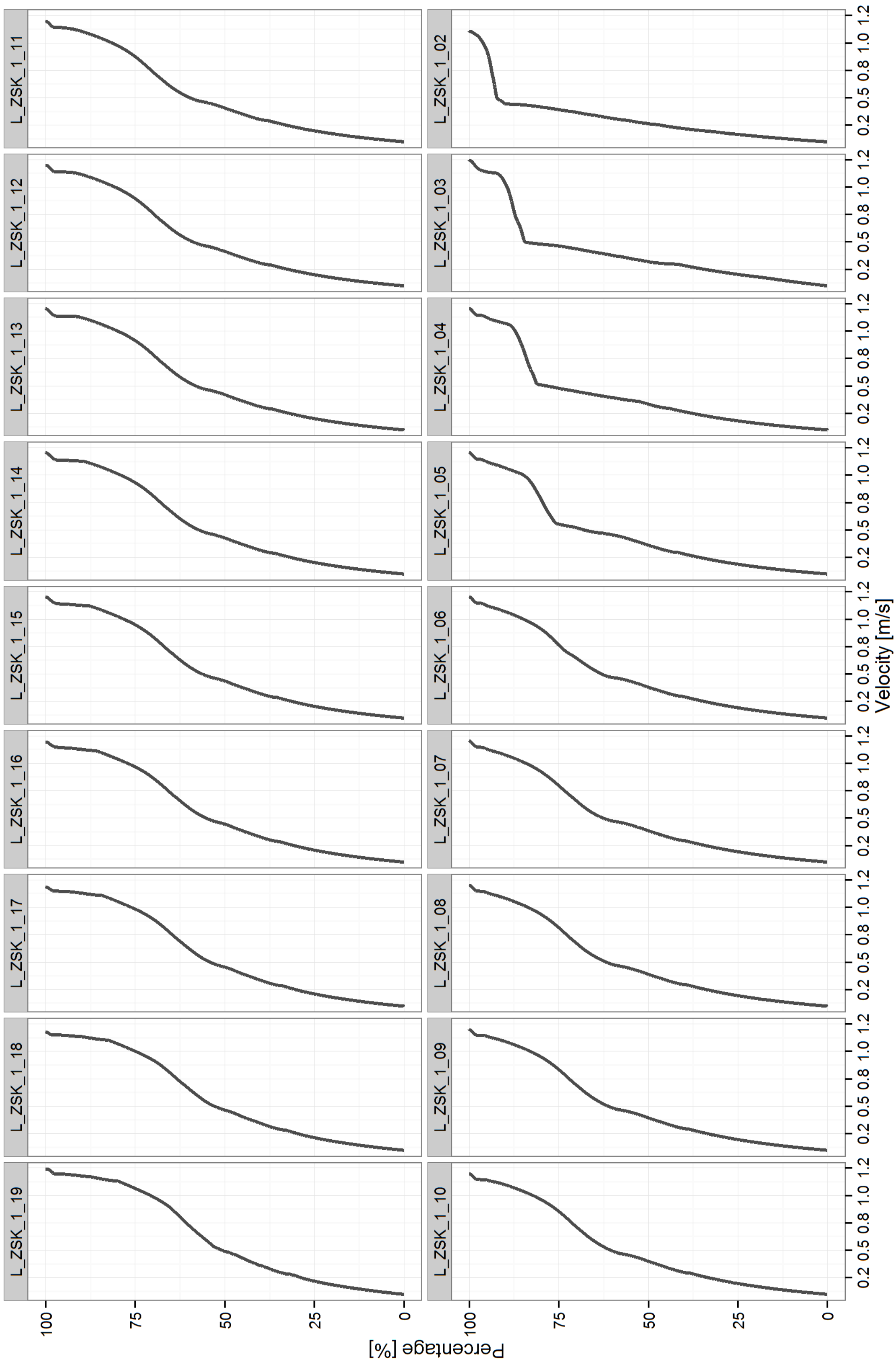

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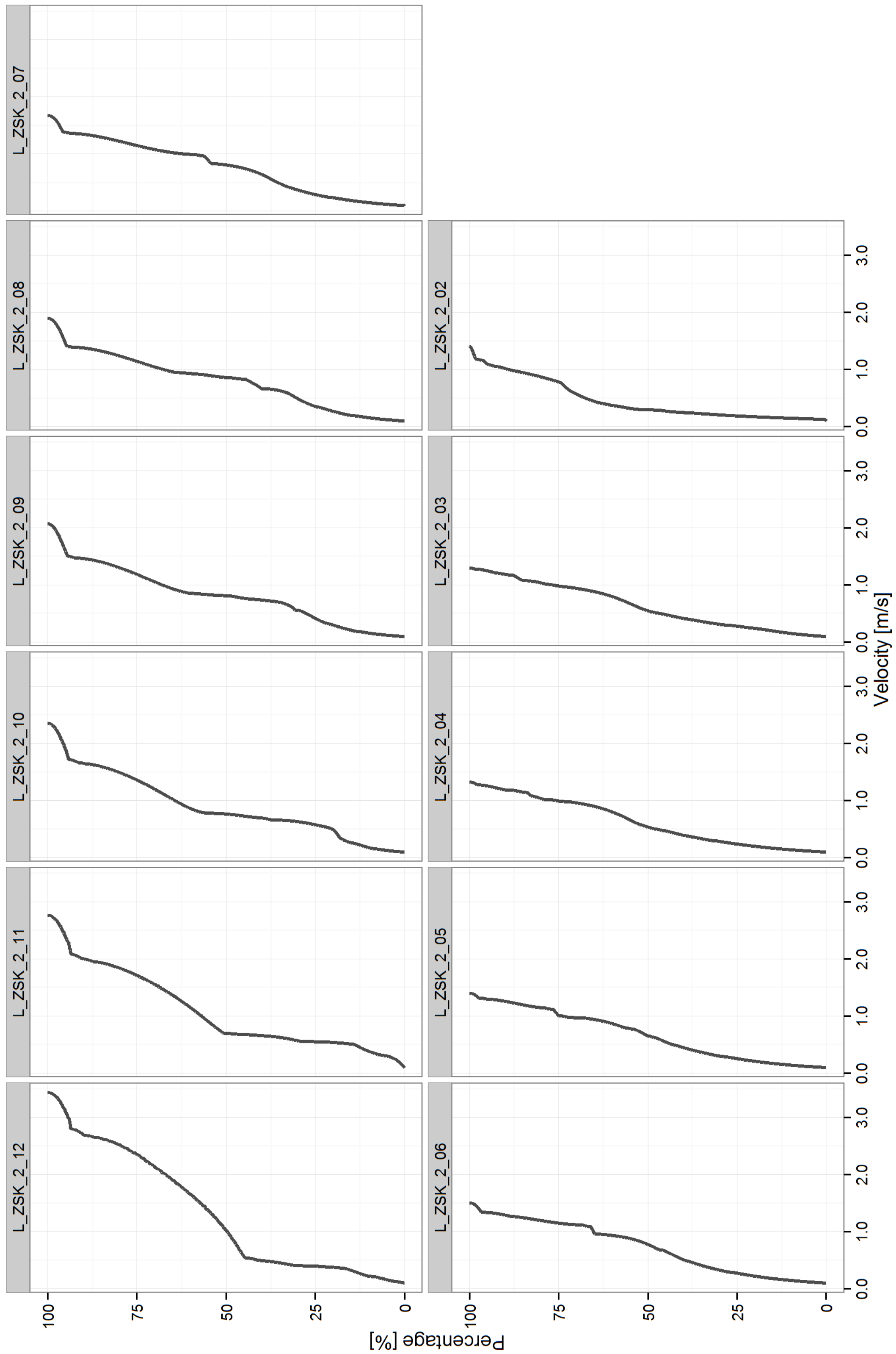
Figure 3-8: Flushing: 3W6M – ZSK1

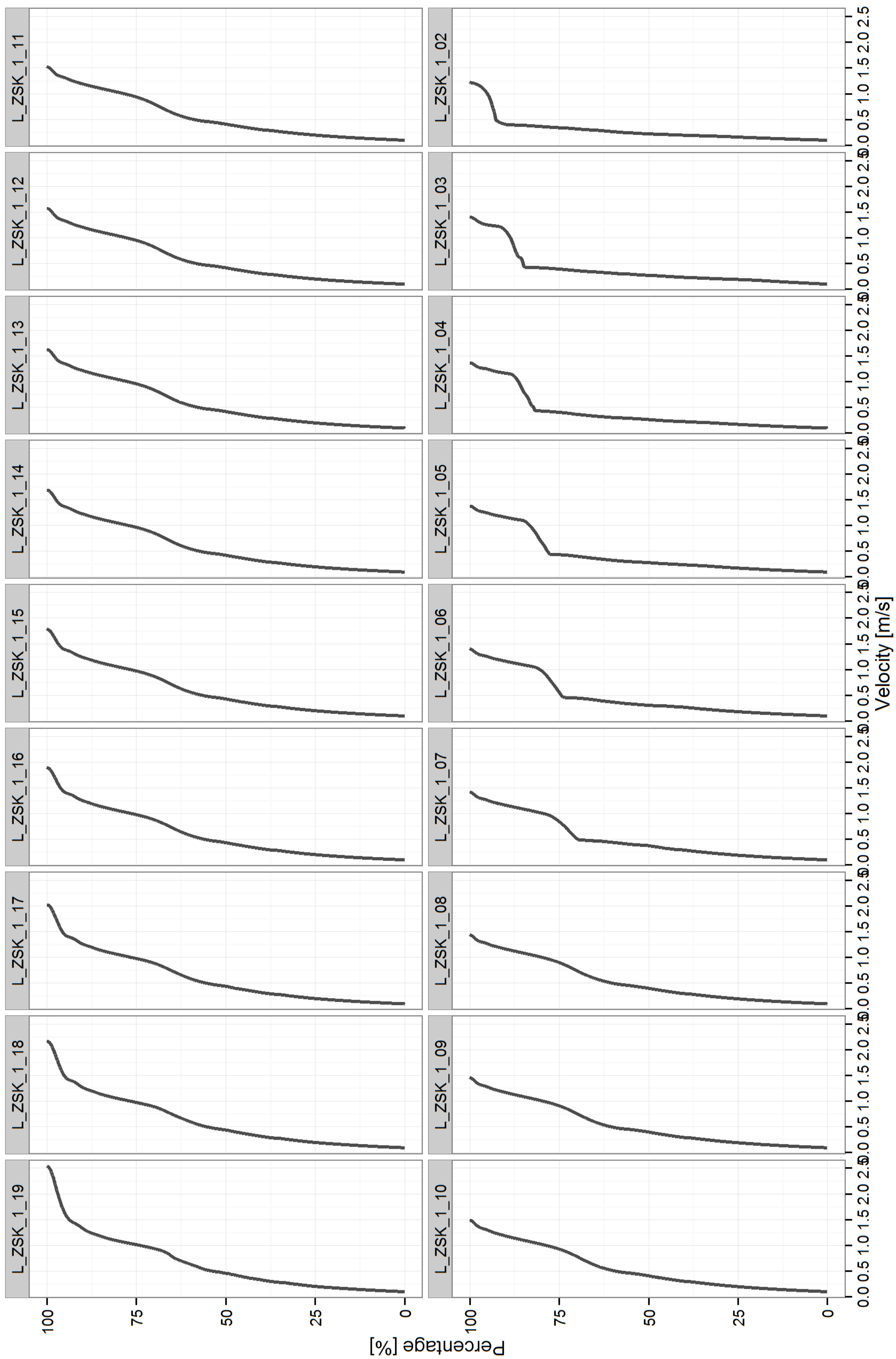
Most effective strategy found during the simulations (4WSt)

The most effective scenario was to generate four consecutive waves and to intercept them at the weir KS2 to release them together into ZSK1 of the tunnel to form one big wave. For ZSK2, the highest possible flushing speed is reached by sending the consecutive waves and in ZSK, a big wave is released that still has a velocity of about 1.2 m/s at the weir KS1. Figure 3-9 and Figure 3-10 show the velocity distribution of this approach.



Smallest index refers to the link most downstream

Figure 3-9: Flushing: 4WSt – ZSK2



Smallest index refers to the link most downstream

Figure 3-10: Flushing: 4WSt – ZSK1

3.1.5 Comparison

Table 3-2 gives an overview of all scenarios and lists their total durations until the point when the flow at the orifice towards the WWTP is less than 10 l/s. v_{maxKS2} describes the velocity in front of the weir KS2 and v_{maxKS1} the velocity in front of the weir KS1.

Table 3-2: Overview of all emptying and flushing scenarios with top speeds

Goal	Name	Duration	v_{maxKS2} [m/s]	v_{maxKS1} [m/s]
Emptying scenarios	RF	11h 53m	0.805	0.446
	RHF	04h 54m	0.773	0.455
	RZSK1F	09h 39m	0.834	0.450
	RZSK1HF	03h 22m	0.000	0.441
	RZSK2F	06h 10m	0.812	0.933
	RZSK2HF	03h 34m	0.773	0.941
	QF	11h 52m	1.791	0.445
	QHF	04h 56m	1.736	0.451
	QZSK1F	09h 40m	1.980	0.461
	QZSK1HF	03h 22m	0.000	0.441
	QZSK2F	04h 44m	1.867	1.543
	QZSK2HF	02h 56m	1.734	1.271
	EF	12h 03m	1.835	1.533
	EHF	05h 53m	1.794	1.250
	EZSK1F	10h 11m	1.543	1.298
	EZSK1HF	03h 22m	0.000	0.441
	EZSK2F	04h 44m	1.854	1.541
	EZSK2HF	02h 56m	1.716	1.270
Flushing scenarios	1W	03h 55m	0.915	0.907
	2W	03h 06m	1.060	1.062
	3W	03h 11m	1.086	1.111
	3W6M	03h 13m	1.060	1.106
	3W8M6M	03h 15m	1.053	1.079
	3W10M6M	03h 17m	1.045	1.061
	2WSt	03h 09m	1.528	1.102
	3WSt	03h 21m	1.335	1.159
	4WSt	03h 38m	1.438	1.210
	2WSt1W	03h 25m	1.358	0.922

In comparison, the emptying scenario refill-on-empty is clearly superior to the others when the focus depends only on wave speed. As for the flushing scenarios, the four waves intercepted at KS2 and then released together results in the highest velocity at the bottom of the tunnel. This shows that the biggest waves produce the highest speed and therefore the best cleaning efficiency under the assumption that wave velocity directly correlates with the ability of the wave to reduce deposits.

The combination of first emptying the tunnel with the refill-on-empty scenario and then flushing it with the 3WSt scheme produced velocities of 2.1 m/s at KS2 and

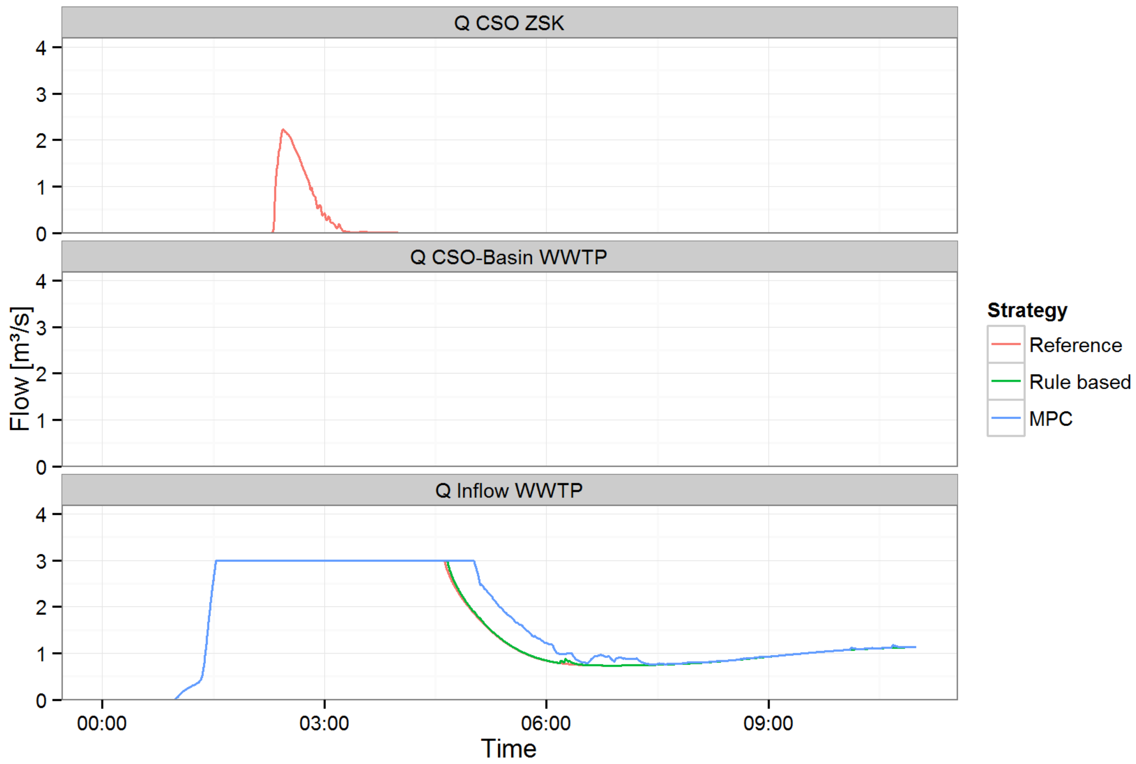
1.5 m/s at KS1. 3WSt was used in this combination as a compromise to not generate too much extra volume for the WWTP. The speed distribution for this setup can be found in the appendix.

3.2 Control scenarios for RTC in the ZSK

The following subchapter will show how even a simple implementation of RTC can improve a seemingly static system. As described in chapter 2.2.2.2, the reference system works completely without reactions to the current state of the network. The RTC strategies used here only control the weir K0_EINL that separates the collector tunnel and the main collector (see chapter 2.1.5.1). The rule based RTC (if implemented in reality) makes control decisions based on the water depth sensors at KS1 and the CSO basin. The efficiency of the strategies will be shown for three different events representing storms with 20, 30 and a 50-year return periods (described in chapter 2.1.3.1).

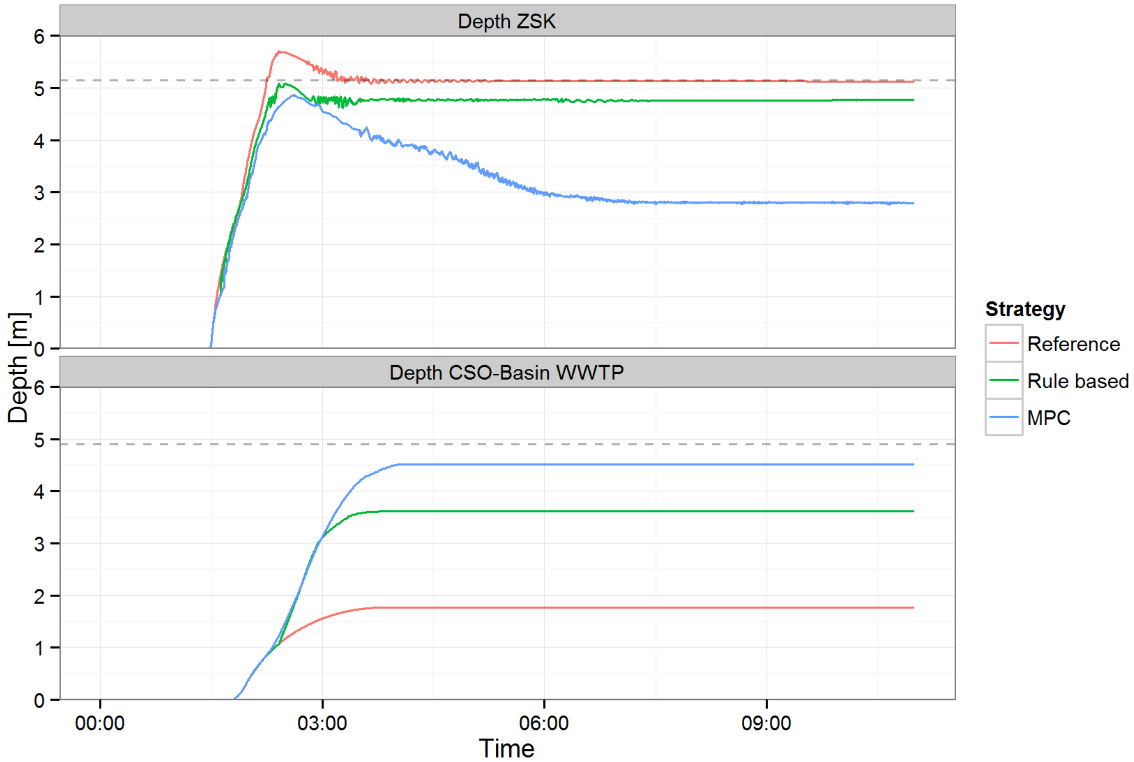
3.2.1 20 year return period

In this test scenario a storm with a 20-year return period was simulated. This was the first return period for a 90-minute storm event duration that triggered a CSO.



With: Q CSO ZSK … Overflow from the ZSK; Q CSO basin WWTP … Overflow from the CSO basin at the WWTP; Q Inflow WWTP … Inflow towards the WWTP

Figure 3-11: Flows for the 20-year return period



With: Depth ZSK … Water level in KS1; Depth CSO basin WWTP … Water level in the CSO basin at the WWTP

Figure 3-12: Water levels of ZSK and CSO basin for the 20 year return period

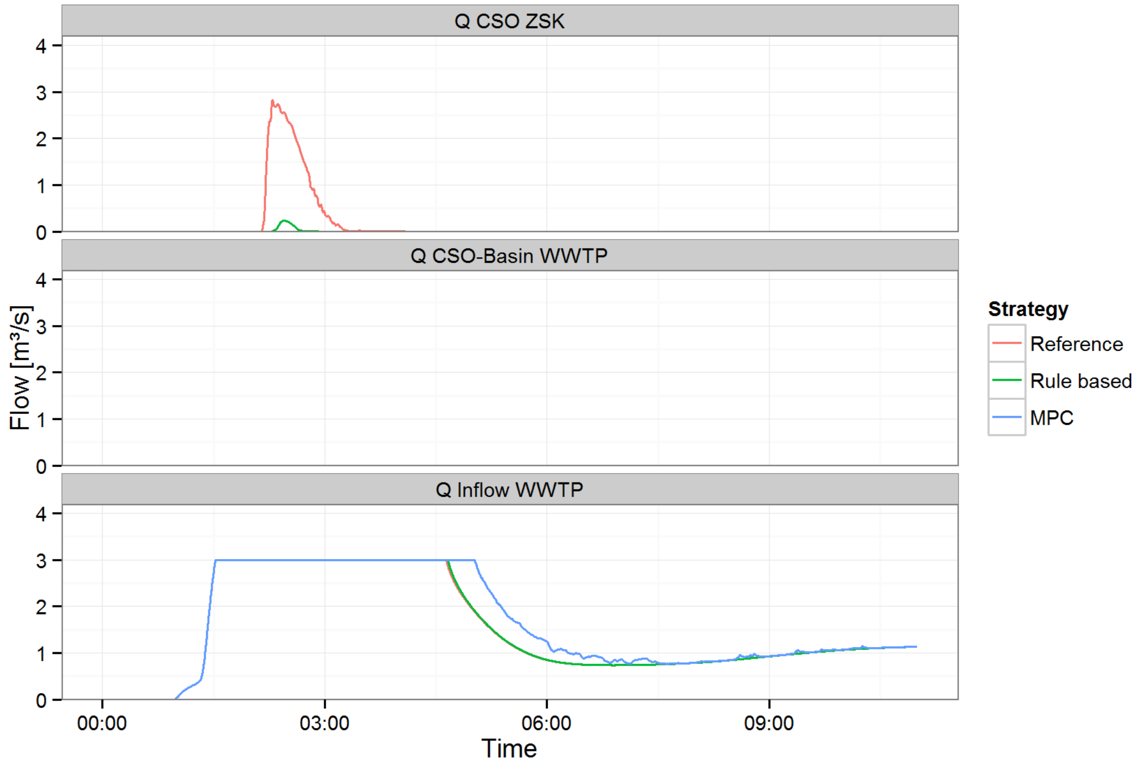
Figure 3-12 shows that the collector tunnel fills up within a half hour in all three scenarios. Then the reference scenario does not take a control action, which leads to a CSO while the CSO basin at the WWTP is not even filled halfway (it only reaches a water level of below 2 m). The rule based and the MPC strategies however, take control decisions and open K0_EINL which results in a higher water level in the CSO basin and no overflow from either of the two possible sources. The MPC strategy also channels runoff towards the WWTP after the imminent danger of an overflow is over (see Figure 3-11), which explains the higher water level in the CSO basin and the bigger inflow volume to the WWTP. The rule based strategy however only channels runoff towards the WWTP while the water level in the ZSK is above 4.79 m and stops immediately after it drops below that limit again. This behavior means that the total inflow volume to the WWTP is almost the same as in the reference scenario. So the rule based strategy only channels the overflow volume that would occur in a non-controlled system to the CSO basin. Finally Table 3-3 shows that both RTC strategies reach a 100% reduction in CSO volume.

Table 3-3: Total flow for the 20-year return period

	Reference	MPC	Rule based
Overflow ZSK	3624.1 m^3	0.0 m^3	0.0 m^3
Overflow WWTP	0.0 m^3	0.0 m^3	0.0 m^3
Inflow WWTP	58658.8 m^3	62749.1 m^3	58863.2 m^3
Total overflow	3624.1 m^3	0.0 m^3	0.0 m^3
Total reduction	--	100%	100%

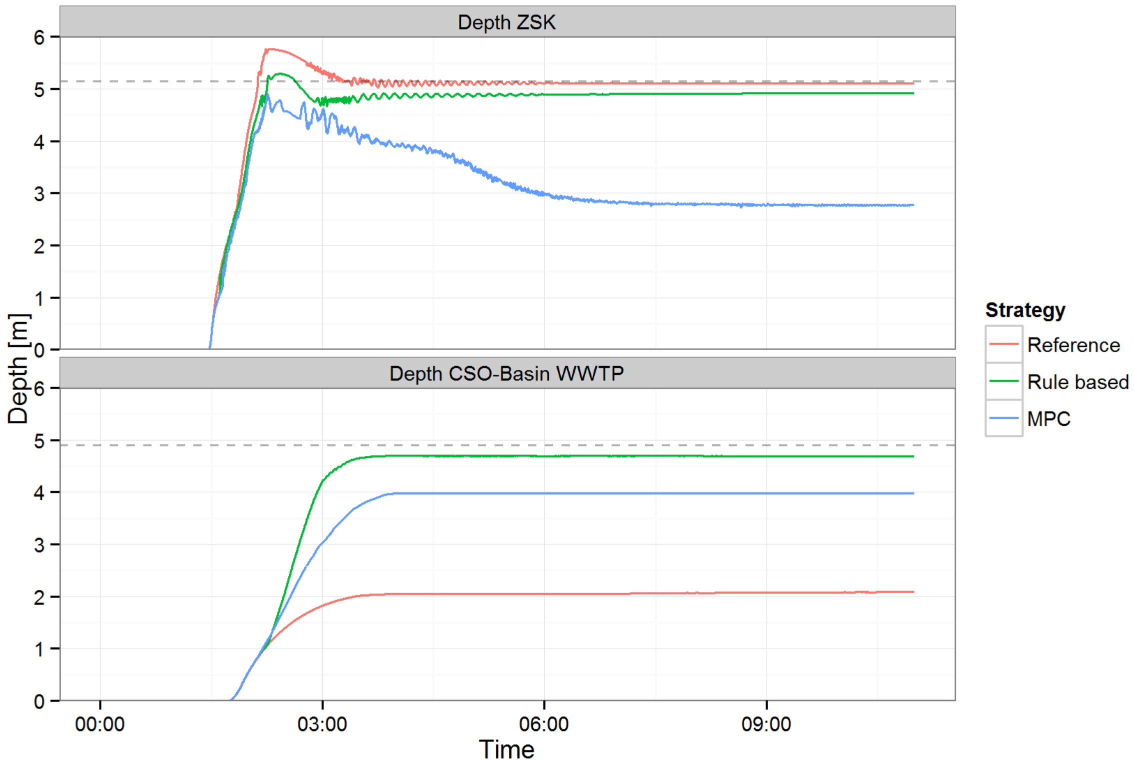
3.2.2 30 year return period

This scenario simulates a storm with a 30-year return period. In this simulation the rule based RTC fails to prevent a CSO for the first time.



With: Q CSO ZSK … Overflow from the ZSK; Q CSO basin WWTP … Overflow from the CSO basin at the WWTP; Q Inflow WWTP … Inflow towards the WWTP

Figure 3-13: Flows for the 30-year return period



With: Depth ZSK … Water level in KS1; Depth CSO basin WWTP … Water level in the CSO basin at the WWTP

Figure 3-14: Water levels of ZSK and CSO basin for the 30 year return period

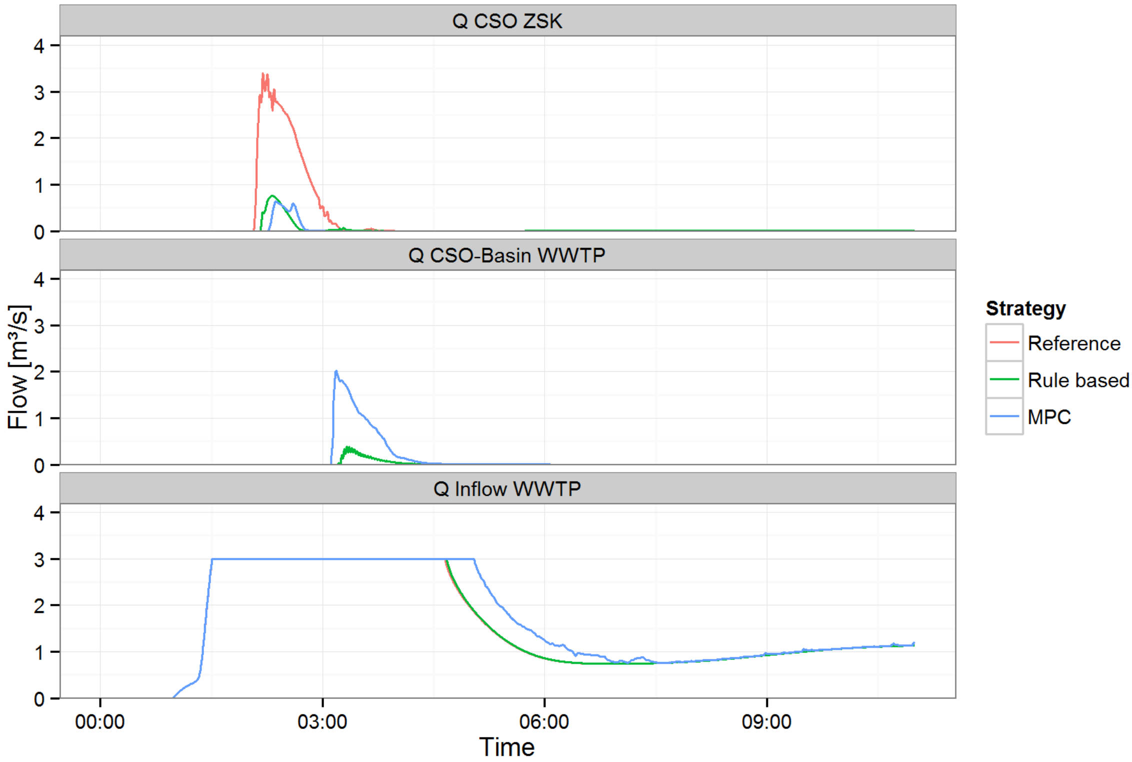
Figure 3-13 and Figure 3-14 show the significant flows and the water levels in both storage facilities. Whereas the reference scenario again quickly triggers a CSO, both of the other strategies manage to redirect the water towards the CSO basin at the WWTP as they did for the 20-year event. This time however, the rule based RTC soon fills up both facilities and, although much smaller, it also triggers a CSO. The MPC strategy is able to supply the storage basin at the WWTP with a less intense flow so that the water level curve in the ZSK is not as steep. This leads to an outcome where the WWTP receives more water over a longer period and reduces the runoff peak just enough to prevent an overflow. Table 3-4 however shows that both RTC strategies reach a CSO volume reduction of above 96%.

Table 3-4: Total flow for the 30-year return period

	Reference	MPC	Rule based
Overflow ZSK	5383.7 m^3	0.0 m^3	188.5 m^3
Overflow WWTP	0.0 m^3	0.0 m^3	0.0 m^3
Inflow WWTP	58891.0 m^3	62986.0 m^3	59008.7 m^3
Total overflow	5383.7 m^3	0.0 m^3	188.5 m^3
Total reduction	--	100%	96.5%

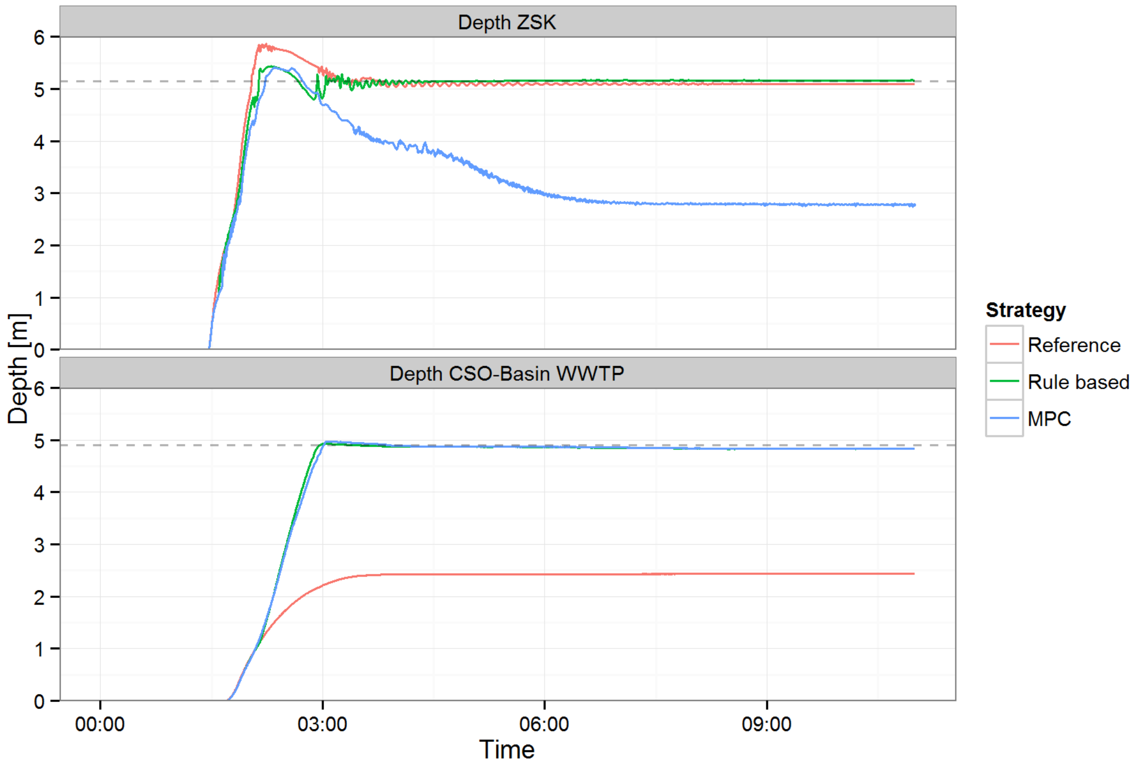
3.2.3 50 year return period

In the 50-year return period scenario all strategies produced overflows, which means that the storage capacity is completely full and without further adjustments of the RTC strategies these overflows cannot be prevented.



With: Q CSO ZSK … Overflow from the ZSK; Q CSO basin WWTP … Overflow from the CSO basin at the WWTP; Q Inflow WWTP … Inflow towards the WWTP

Figure 3-15: Flows for the 50-year return period



With: Depth ZSK … Water level in KS1; Depth CSO basin WWTP … Water level in the CSO basin at the WWTP

Figure 3-16: Water levels of ZSK and CSO basin for the 50 year return period

In this scheme, the reference scenario fails again, as it did in the other situations, because no control actions were taken. Figure 3-16 also shows that MPC and rule based RTC react very similarly when dealing with the given conditions. However the same behavior that helped the MPC strategy from overflowing in the 30-year event fails here. It delays the overflow at the ZSK and channels the occurring peak towards the WWTP where it quickly fills the basin because this time, the runoff peak is too high to be treated by the plant. Next, all the rechanneled water in the pipes between ZSK and CSO basin flows over at the WWTP in addition to the water that triggers the CSO at the collector tunnel. Only then the MPC realizes the overflow at the basin and closes K0_EINL. This results in a bigger CSO volume than the one of the rule based RTC as can be seen in Table 3-5. A difference between MPC and rule based RTC of 38.9% is rather significant and shows that the MPC strategy needs to be adjusted.

Table 3-5: Total flow for the 50-year return period

	Reference	MPC	Rule based
Overflow ZSK	6971.4 m^3	746.4 m^3	997.7 m^3
Overflow WWTP	0.0 m^3	3501.1 m^3	538.6 m^3
Inflow WWTP	59059.1 m^3	63225.1 m^3	59173.2 m^3
Total overflow	6971.4 m^3	4247.5 m^3	1536.3 m^3
Total reduction	--	39.1%	78%

3.2.4 Issue with the MPC

The MPC strategy showed some problems when looking at the control actions taken during the simulation. Right now the MPC only works towards the goal of minimizing the CSO volume. Plotting the set pointer for the orifice for the storm event with a 30-year return period (Figure 3-17) shows that the orifice is constantly moving. If it reacted like that in reality, it would not last longer than a few months before it needed its first revision.

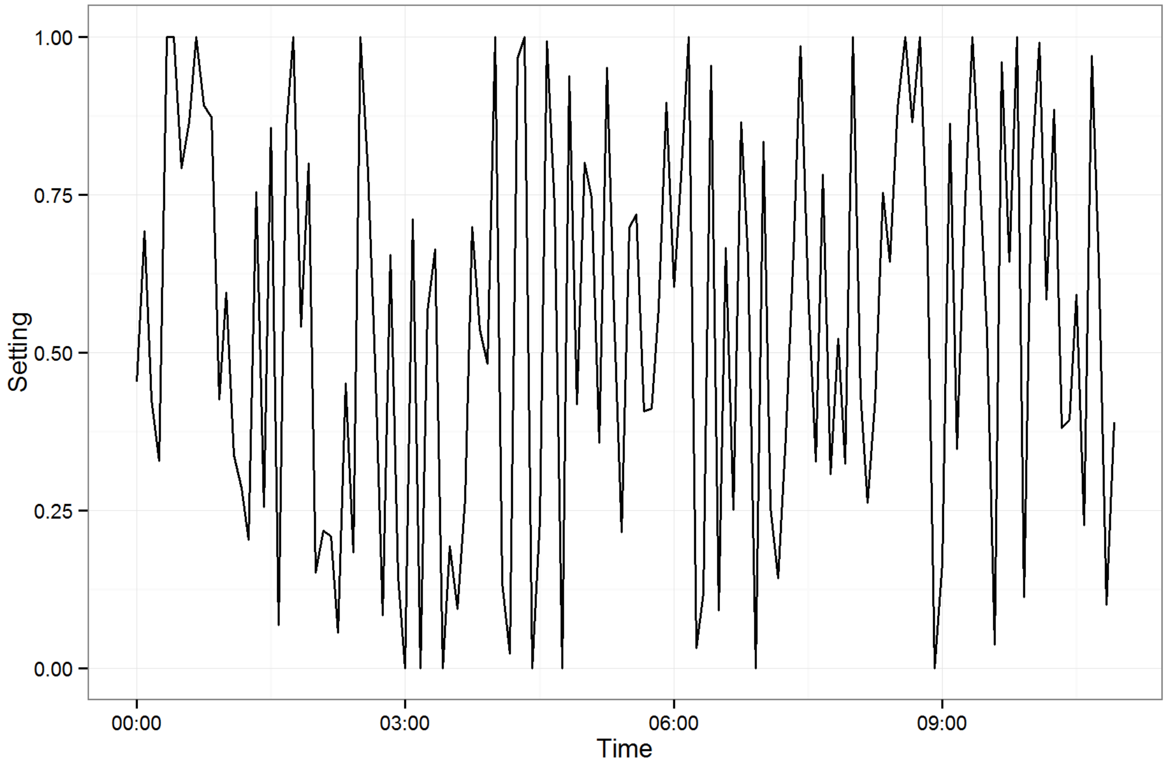


Figure 3-17: Set pointer for the 30-year return period's event

4 Summary, Conclusion and Outlook

This thesis aimed to show the maximal possible use of the infrastructure in place in the south of Graz, the ZSK. By building this facility, the city of Graz and Holding Graz created a system that ensures safety and functionality of its urban drainage system and in addition offers a lot of opportunities for Graz University of Technology to develop new strategies and some new approaches in sewer operation.

The goals were to create an accurate integrated model that would later be used to test different emptying and flushing scenarios with it. Furthermore two RTC strategies were developed featuring a rule based RTC and an MPC strategy. All developed scenarios were simulated and tuned to gain the best possible results.

The emptying and flushing scenarios were compared with each other to find the most effective one to be suggested to Holding Graz for future implementation in the actual facility. The scenarios showed promising results and could also be easily translated into manual control schemes if an RTC system is not be implemented in the near future. However in combination with an overall RTC strategy the whole system could work on its own and control decisions from the staff would only have to be made in emergencies.

The RTC scenarios were developed to show what is possible with only a little control effort. These strategies could be executed by a normal PC, and as the connection between sensors and a central control location at the WWTP already exists, the costs to implement those strategies would be minimal.

4.1 Emptying and flushing scenarios in the ZSK

The implementations of the cleaning scenarios clearly showed that the highest velocity can only be achieved by releasing the biggest possible volume at once so that the flushing wave is not spread out too much. However, effects of turbulence and shear cannot be considered in a 1D hydrodynamic model, which means that this model is not enough to make an accurate conclusion about the cleaning efficiency of the different approaches. Nevertheless, to compare strategies to each other with the assumption that the speed of a wave correlates with its capability to reduce deposits, the resolution of the integrated model is enough and it was clearly shown that the refill-on-empty scenario (see chapter 3.1.3) in combination with the 3WSt flushing scheme promises good results.

To get a better idea of what exactly happens during emptying and flushing and if the cleaning effect of flushing waves is enough to prevent deposits, or at least reduce them, a more dimensional hydrodynamic model would have to be created and used together with a turbulence model. Only such a model is capable of showing the processes taking place on the invert.

Furthermore a couple of factors are not clear yet and can only be distinguished by collecting data from the ZSK while it is in use. These factors are:

- Type and composition of the sediments in the ZSK
- Consequences of sediments transported by the river water considering the treatment capacity of the WWTP
- Potential points of weakness in the system

Furthermore a plan for a combination of cleaning sessions with flushing and mechanical cleaning has to be figured out to effectively keep sediments from accumulating in the ZSK.

4.2 Control scenarios for RTC in the ZSK

During the first simulations, a smaller treatment capacity for the WWTP was tried because, based on previous experience, it was known that it sometimes does not work at its full capacity. This however, resulted in a sole overflow of the CSO basin at the WWTP while the collector tunnel was not filled completely. The reason for this is that the weir that cuts off the excess from the main collector and channels it to the ZSK at B25 only has a fixed weir crest. And once the runoff is past this point, no control action can be taken to efficiently use the complete available storage volume of the combined collector tunnel and storage basin. While discussing this topic with the sewer operator, it turned out that there could have been a mistake in the model originally provided by Holding Graz. This needs to be looked into and fixed after the question is answered. The discussion also revealed that the orifice cutting off water from the main collector to manage the stream to the WWTP will be updated soon. In the future this needs to be incorporated into the model as well.

Other than that, the implementation of RTC (rule based or MPC) proved to be a viable measure to lower the CSO volume during storm events. The control of a single spot allowed the use of the full storage capacity provided. Adding, for example, the pump to empty the CSO basin while there is additional treatment capacity would improve the result even more. Further improvement would also include adjusting the MPC to minimize the control actions taken to lengthen the lifespan of the regulators.

4.3 Recommendations and Outlook

The RTC strategies can have a significant effect on the reduction of CSO volume. If a CSO cannot be prevented completely during a storm event, the storages could at least be emptied faster to ensure the functionality of the system if a second rainstorm hit the city shortly after the first.

The emptying and flushing scenarios developed and tested in this thesis can definitely be worked into the control strategy for the ZSK, because they can even be applied without RTC in place by simply manually triggering them.

This thesis only intended to show the potential of the installed system and how it can be used most efficiently. In the future, adaptions will be made to the ZSK like the second stage upstream. These alterations need to be incorporated into the model and the control strategies. With an efficient RTC strategy in place throughout the whole facility, the goal set by the OEWAV Guideline 19 could be reached or even exceeded.

In addition to simply extending the model, integrated control should be the next step. An integrated simulation model that brings rain prediction, the sewer system of Graz, the ZSK and even the WWTP into one system that interacts at each interface would be the most efficient way to handle storm events. Furthermore, fault detection for sensors, instant data processing and fault handling in case of a system failure should be considered in such a model. Graz University of Technology is currently working on such a project including these and new approaches, like risk management, in the process. With the help of Holding Graz, this project could drastically reduce the stress on the ecosystem and could be an example of how a modern urban drainage system should be managed.

List of Tables

List of Figures

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Appendix

In the appendix, all velocity distributions of the emptying and flushing scenarios are attached to show the differences between the various initial statuses and the advantages and disadvantages of the other flushing schemes.

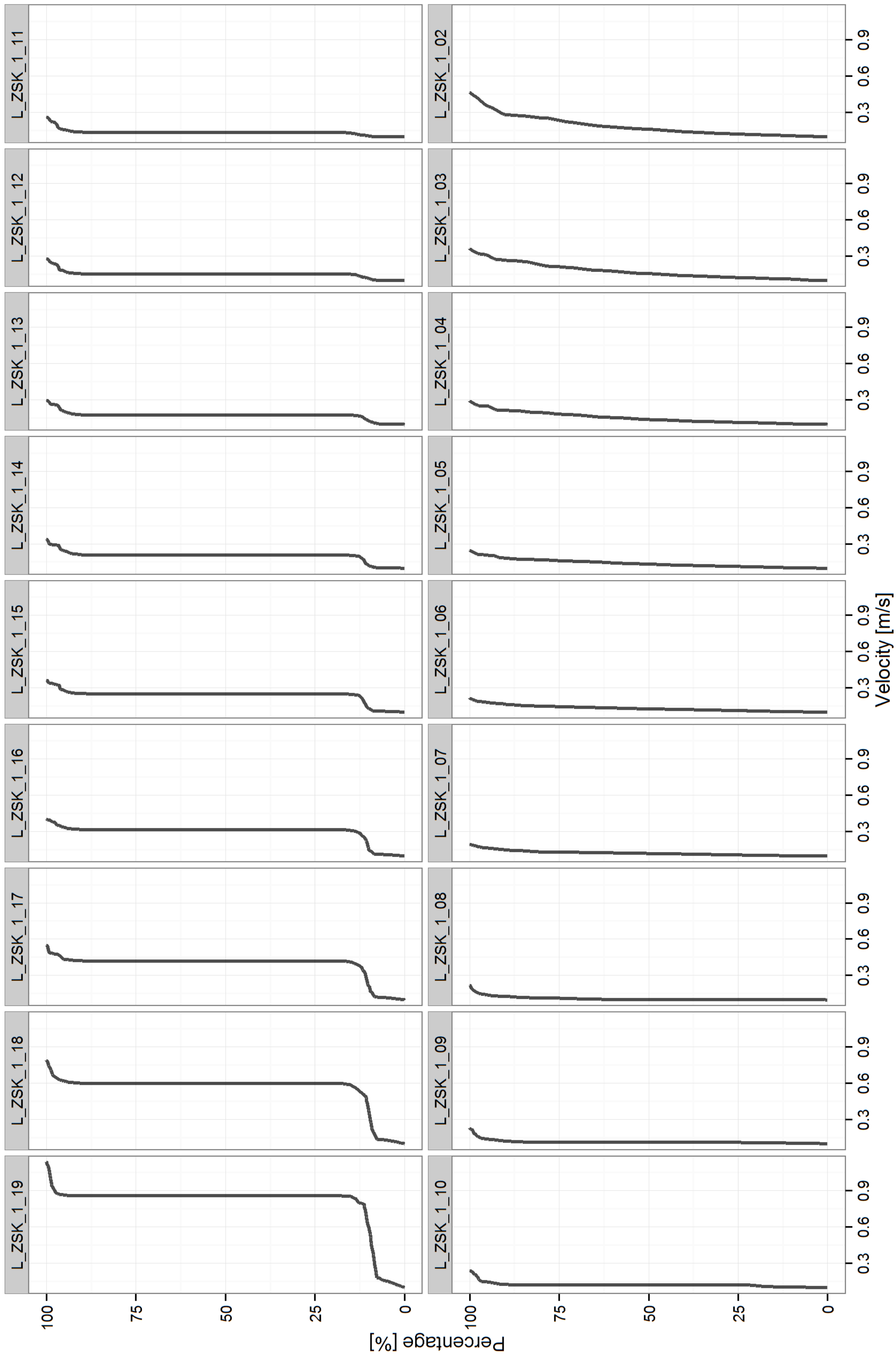
First the emptying scenarios are attached followed by the flushing scenarios and at the end is the optimal strategy.

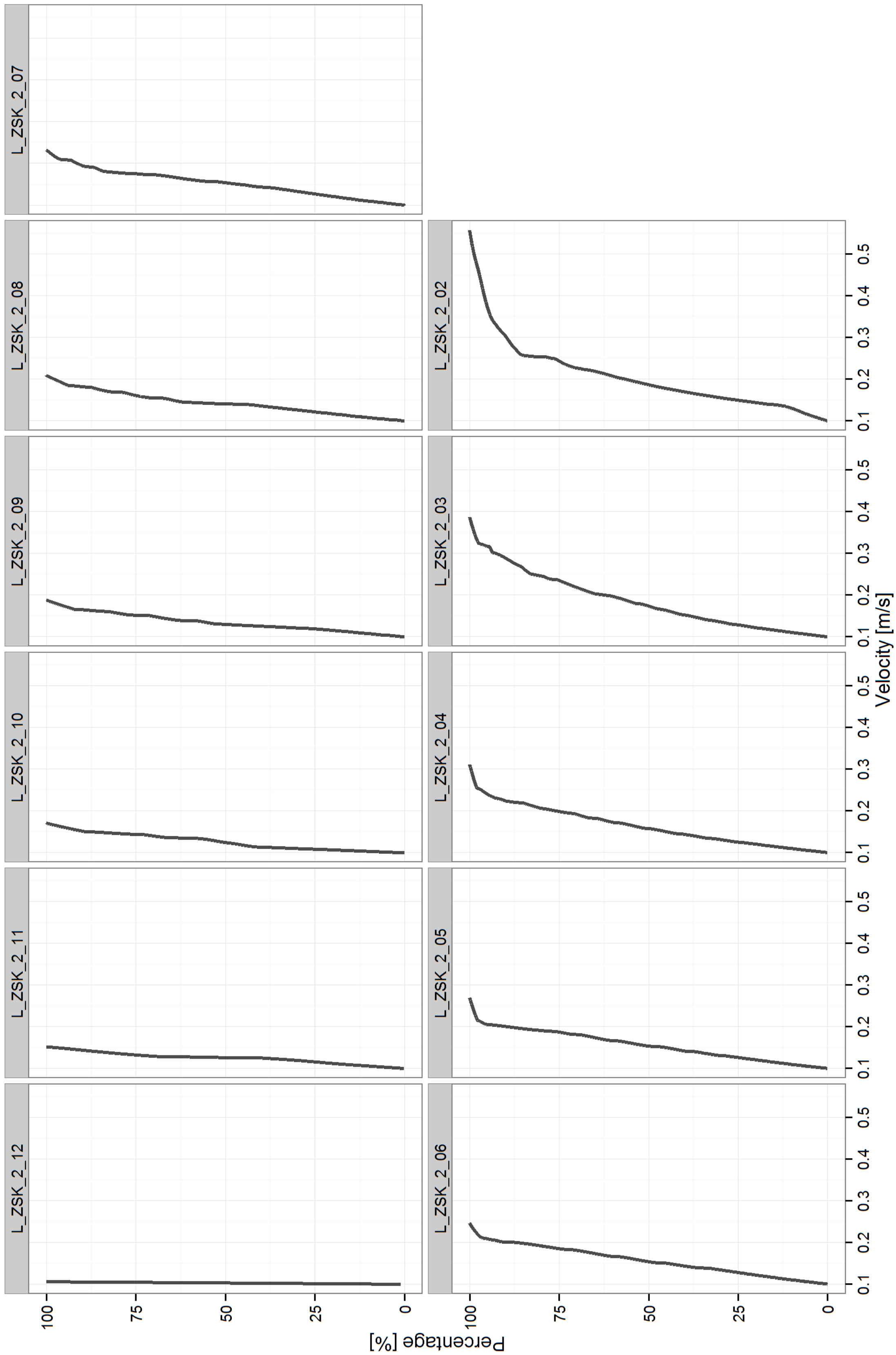
Emptying Scenarios

Overview

Name	Description	v_{maxKS2} [m/s]	v_{maxKS1} [m/s]
RF	Reference scenario FULL	0.805	0.446
RHF	Reference scenario HALFFULL	0.773	0.455
RZSK1F	Reference scenario ZSK1 FULL	0.834	0.450
RZSK1HF	Reference scenario ZSK1 HALFFULL	0.000	0.441
RZSK2F	Reference scenario ZSK2 FULL	0.812	0.933
RZSK2HF	Reference scenario ZSK2 HALFFULL	0.773	0.941
QF	Quick refill scenario FULL	1.791	0.445
QHF	Quick refill scenario HALFFULL	1.736	0.451
QZSK1F	Quick refill scenario ZSK1 FULL	1.980	0.461
QZSK1HF	Quick refill scenario ZSK1 HALFFULL	0.000	0.441
QZSK2F	Quick refill scenario ZSK2 FULL	1.867	1.543
QZSK2HF	Quick refill scenario ZSK2 HALFFULL	1.734	1.271
EF	Refill-on-empty scenario FULL	1.835	1.533
EHF	Refill-on-empty scenario HALFFULL	1.794	1.250
EZSK1F	Refill-on-empty scenario ZSK1 FULL	1.543	1.298
EZSK1HF	Refill-on-empty scenario ZSK1 HALFFULL	0.000	0.441
EZSK2F	Refill-on-empty scenario ZSK2 FULL	1.854	1.541
EZSK2HF	Refill-on-empty scenario ZSK2 HALFFULL	1.716	1.270

Reference scenario – ZSK1 full; ZSK2 full





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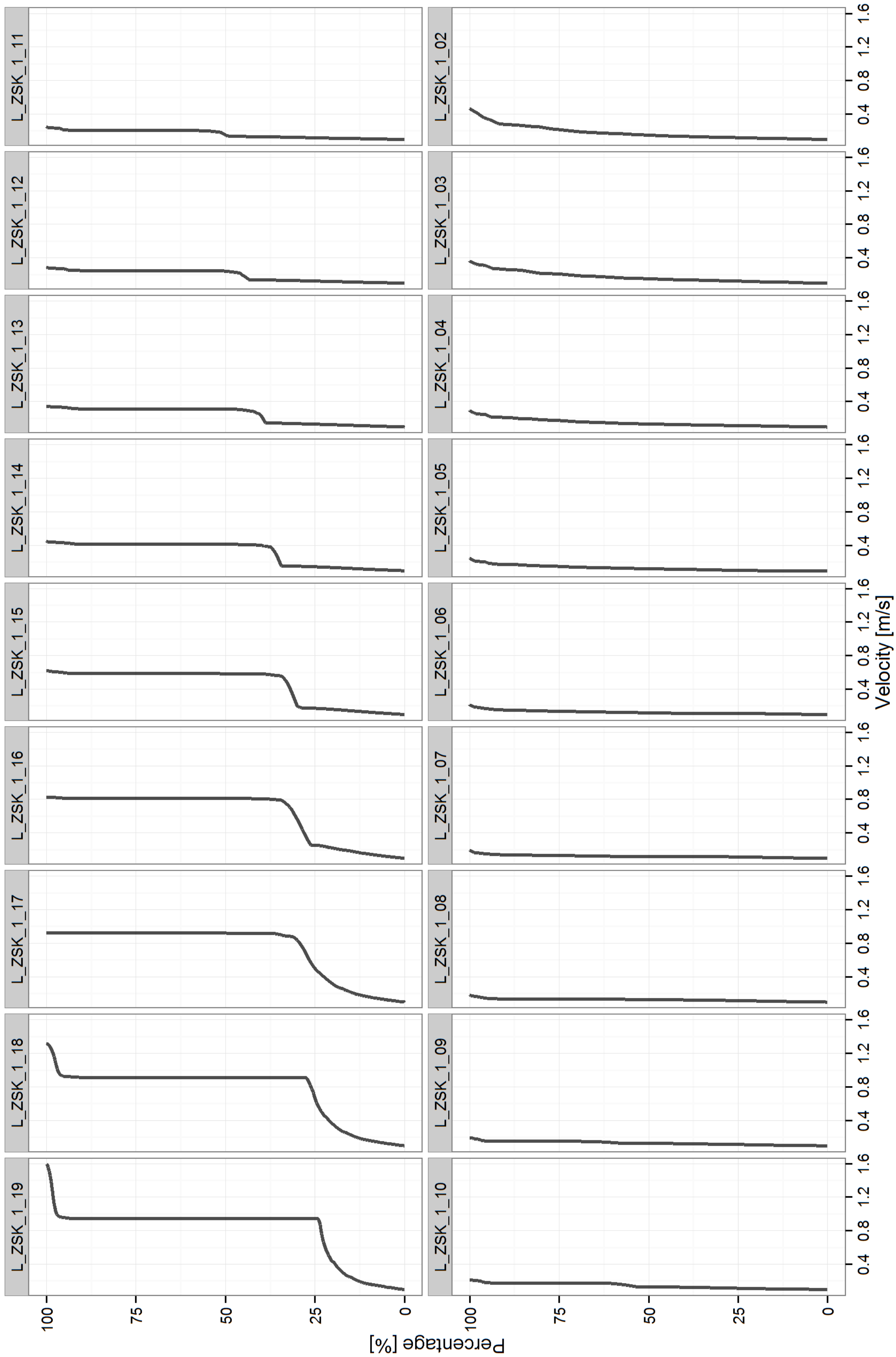
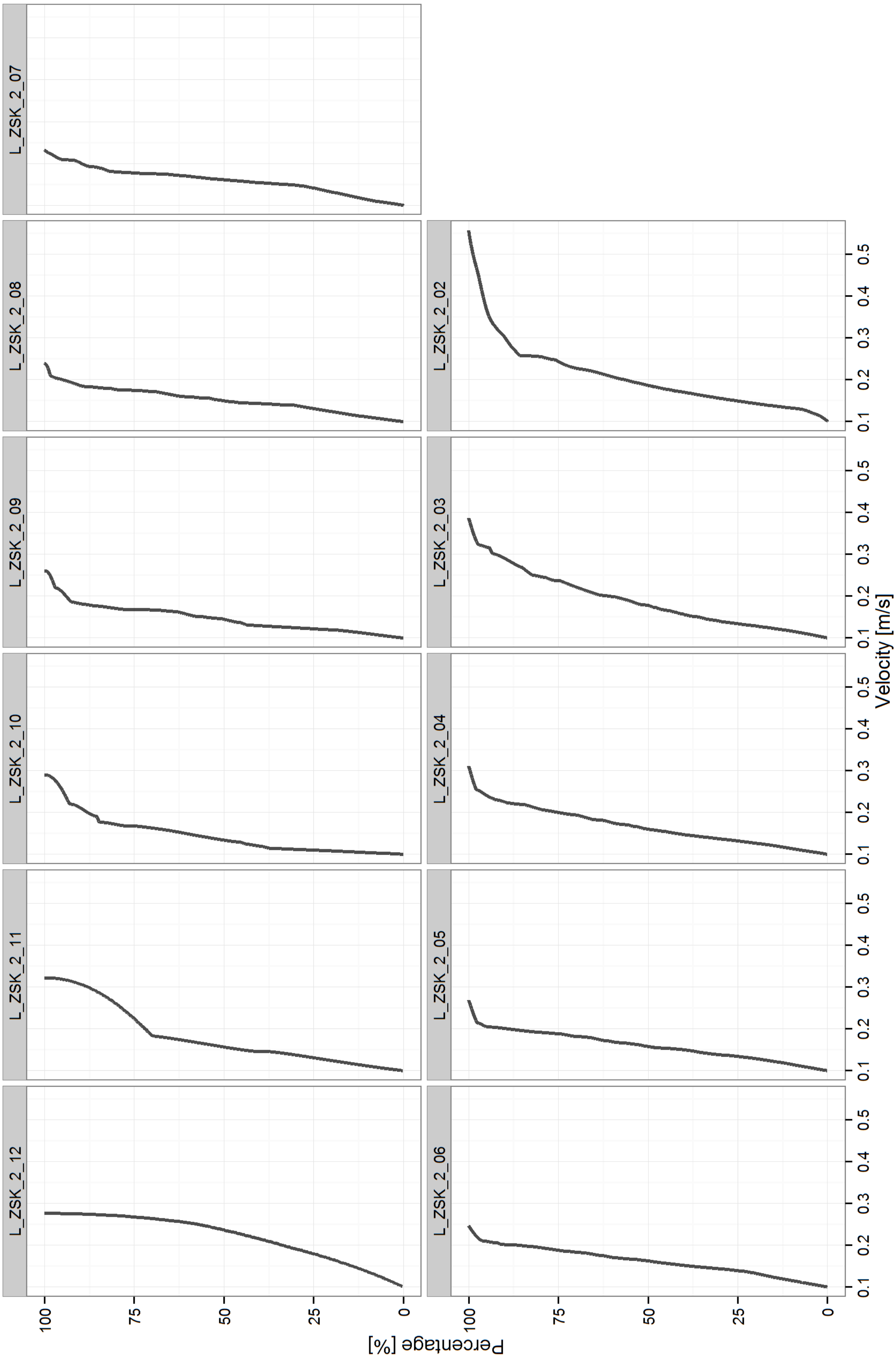
L_ZSK_2_11
L_ZSK_2_12
L_ZSK_2_02
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
Percentage [%]
100
75
50
25
0
Velocity [m/s]
0.1
0.2
0.3
0.4
0.5

Reference scenario – ZSK1 half full; ZSK2 half full





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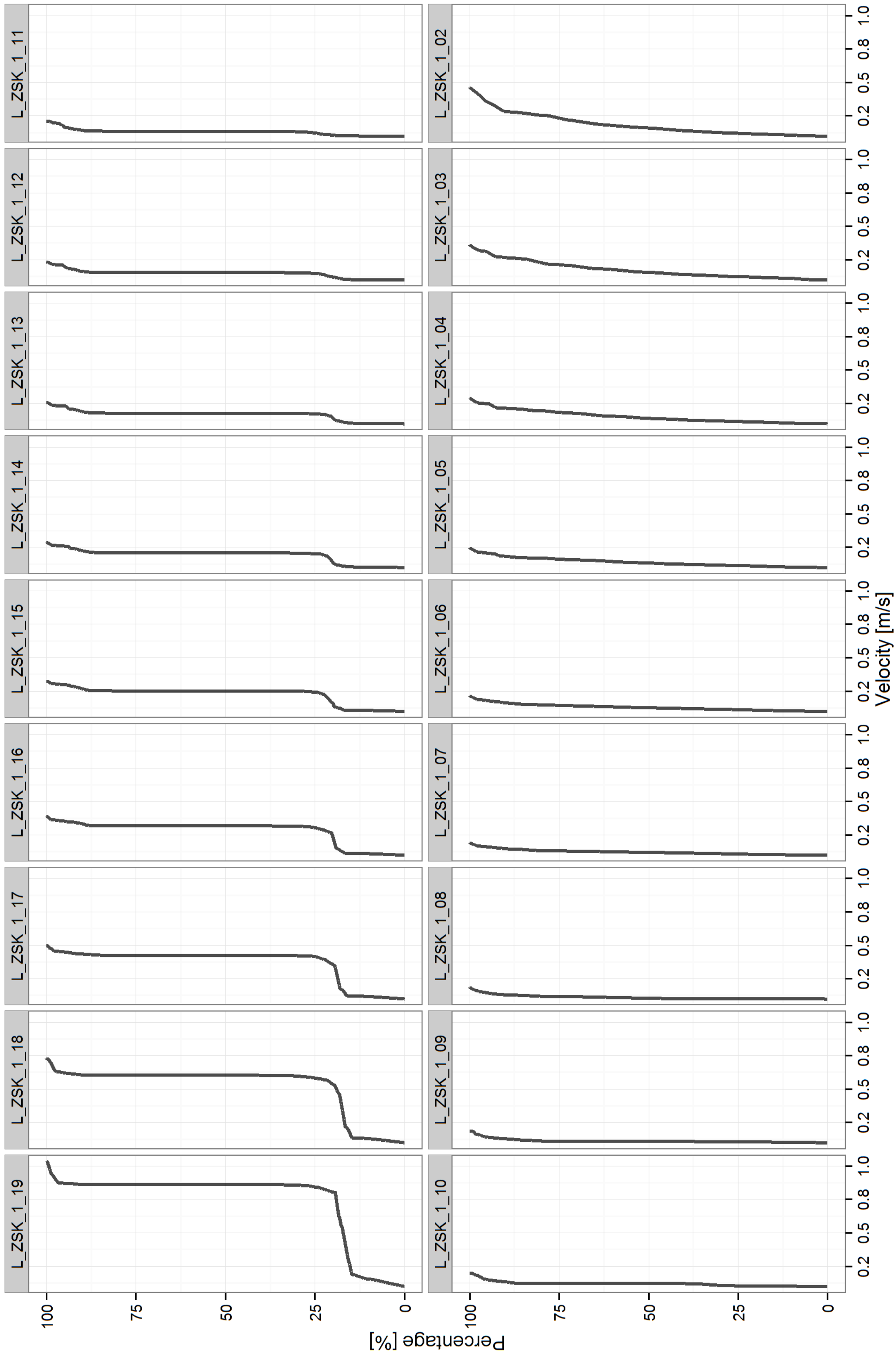
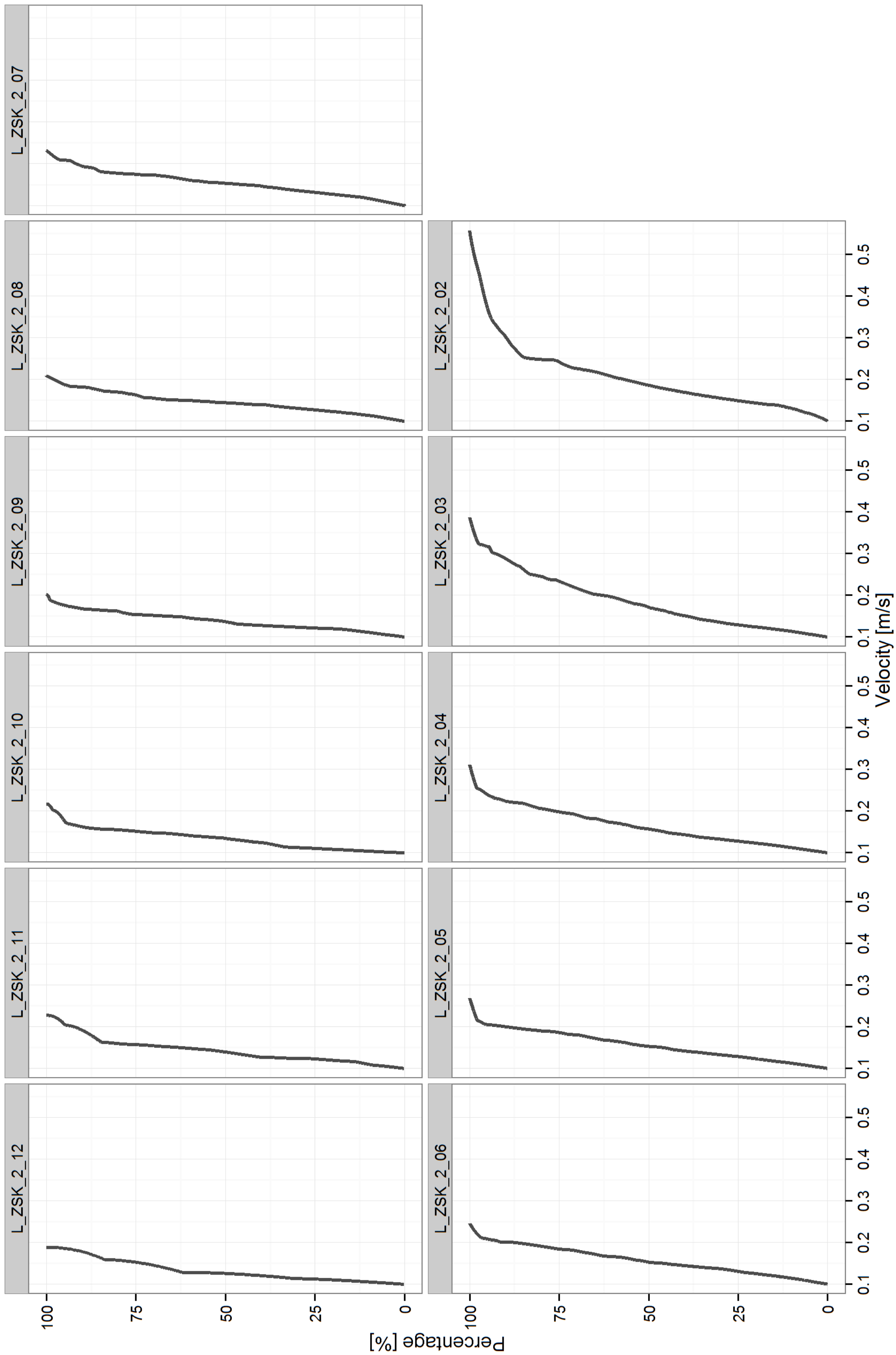
L_ZSK_2_11
L_ZSK_2_12
L_ZSK_2_02
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
Percentage [%]
100
75
50
25
0
Velocity [m/s]
0.1
0.2
0.3
0.4
0.5

Reference Scenario – ZSK1 full; ZSK2 empty

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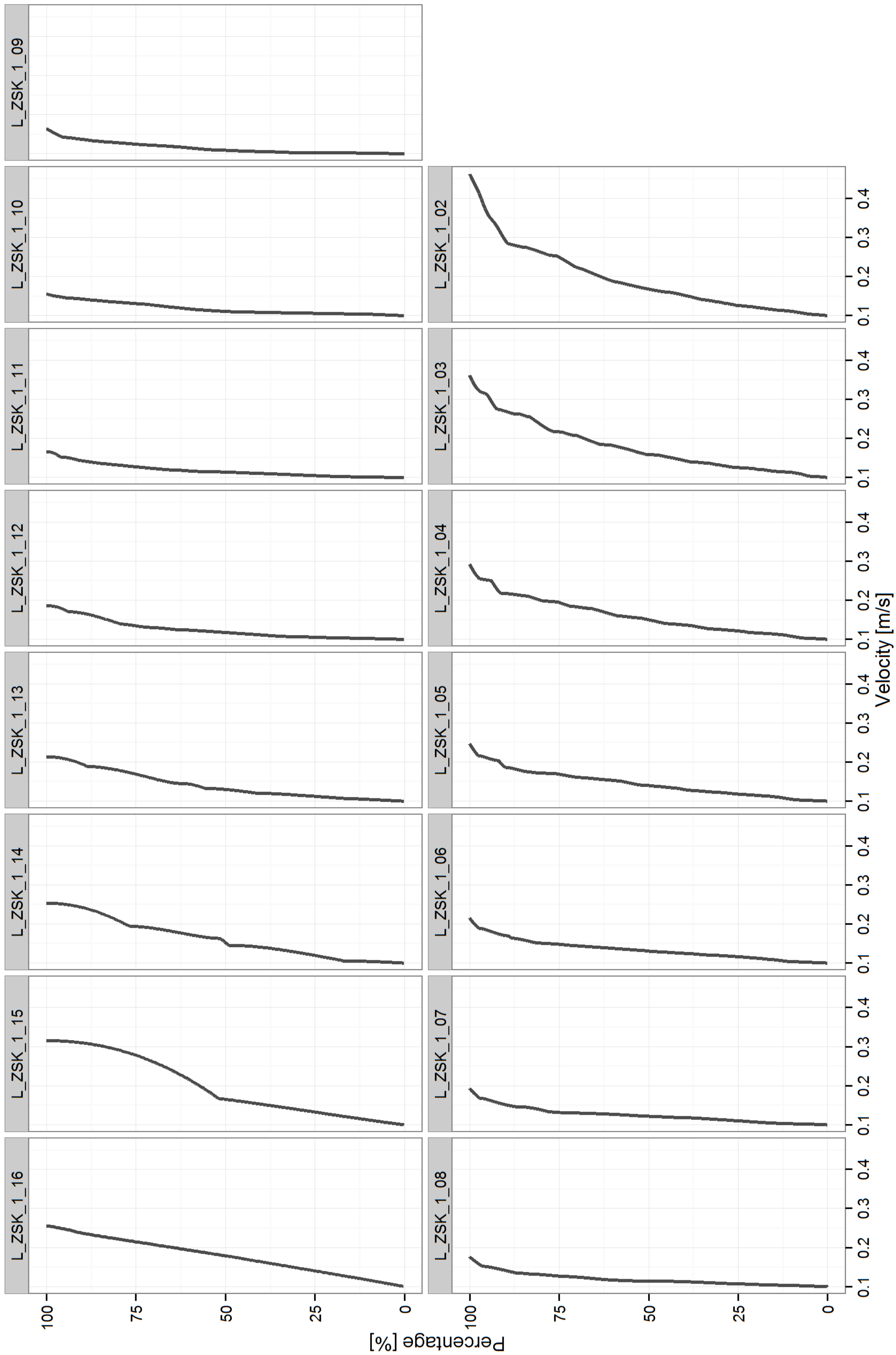
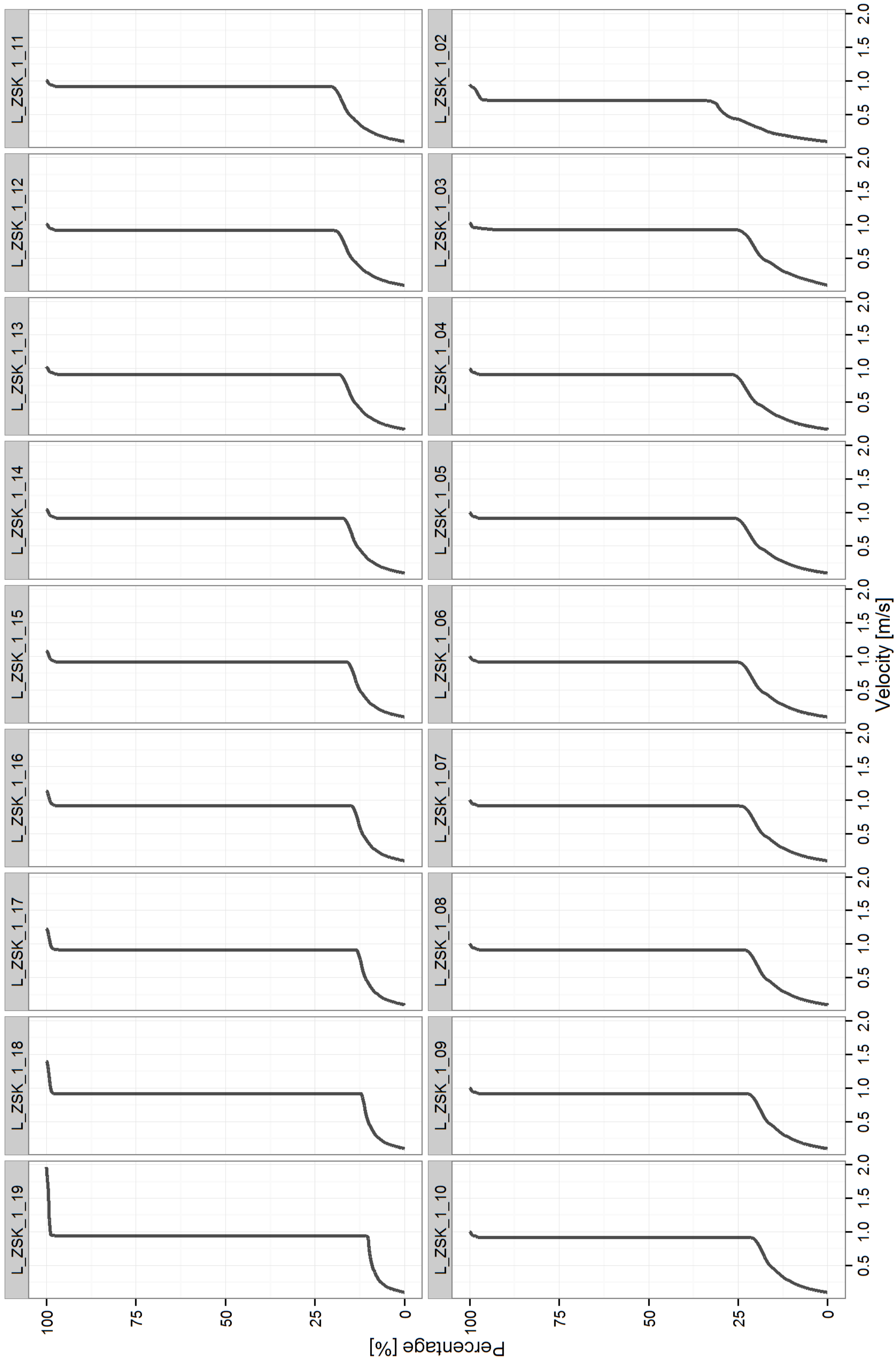
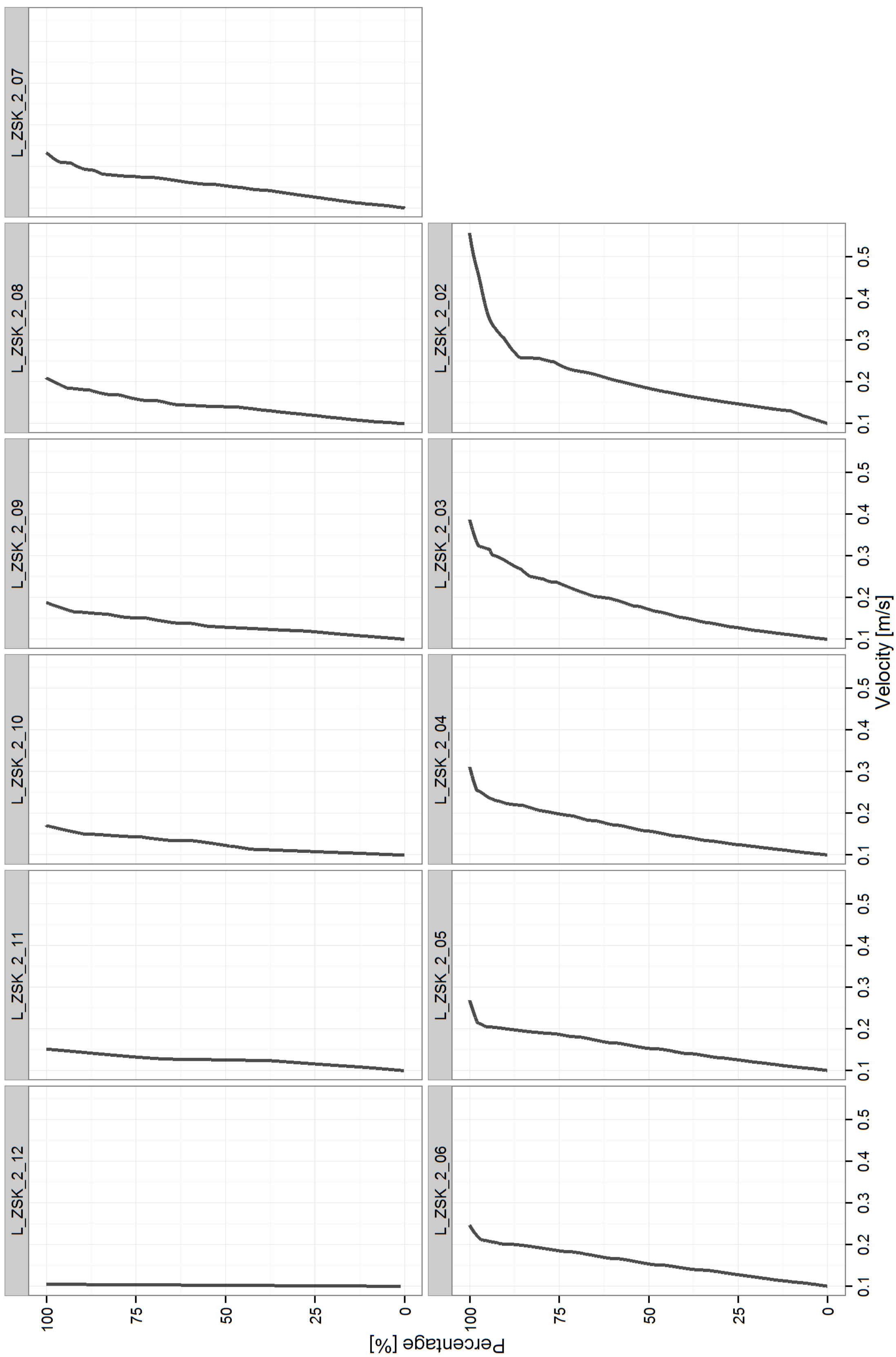
L_ZSK_2_02
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
Percentage [%]
Velocity [m/s]

Reference Scenario – ZSK1 half full; ZSK2 empty



Reference Scenario – ZSK1 empty; ZSK2 full





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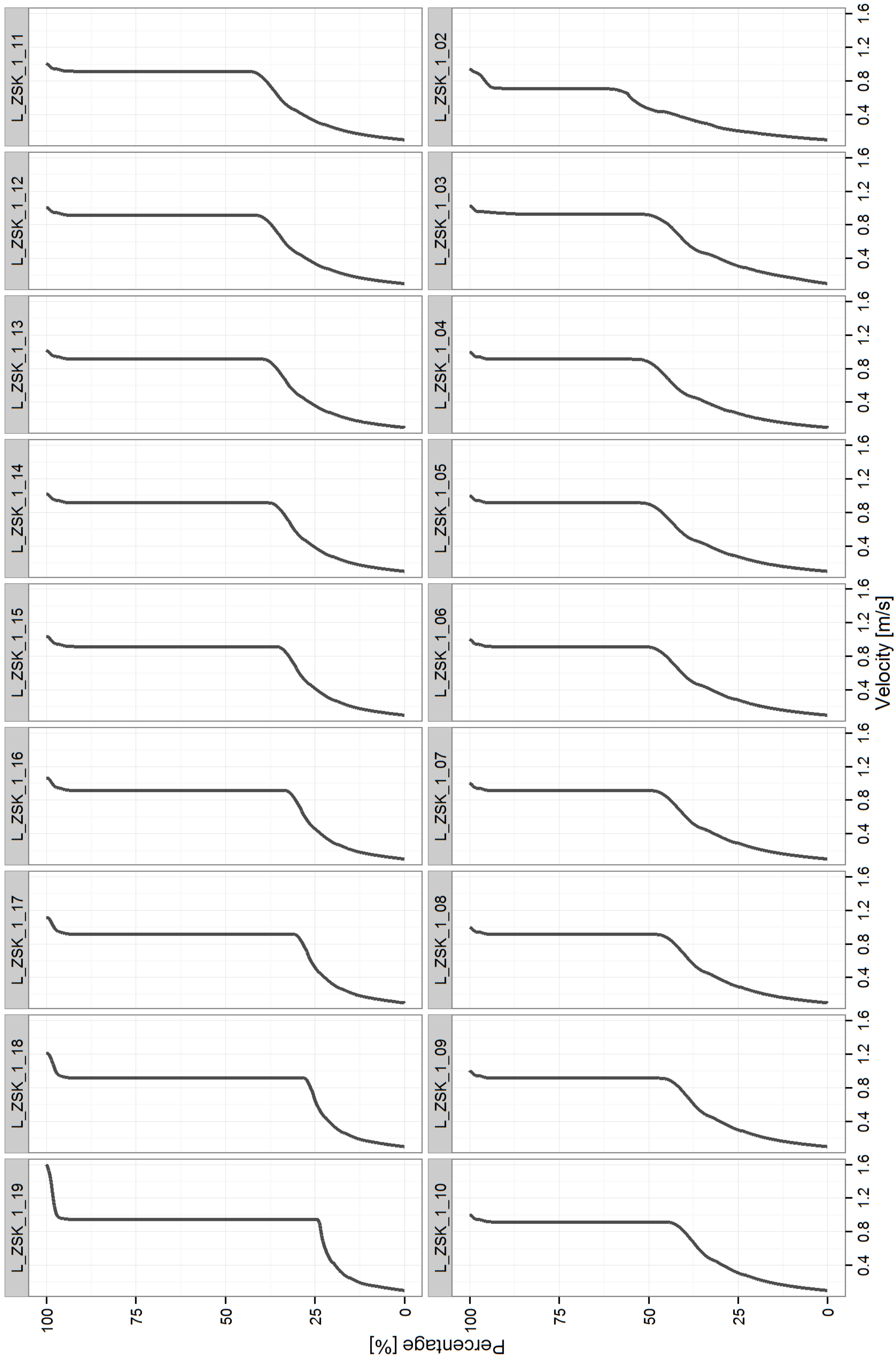
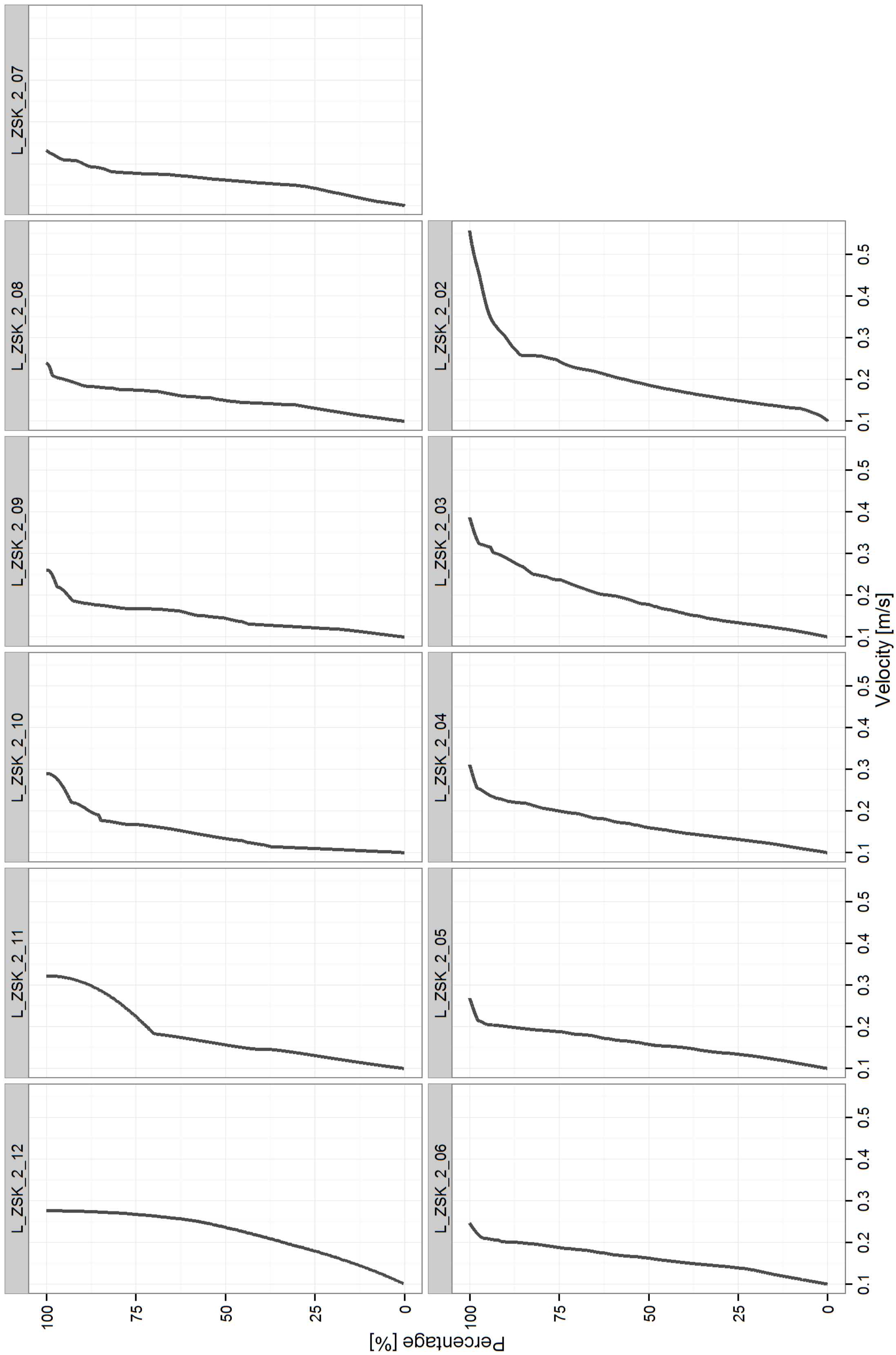
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
Percentage [%]
100
75
50
25
0
Velocity [m/s]
0.1
0.2
0.3
0.4
0.5

Reference Scenario – ZSK1 empty; ZSK2 half full

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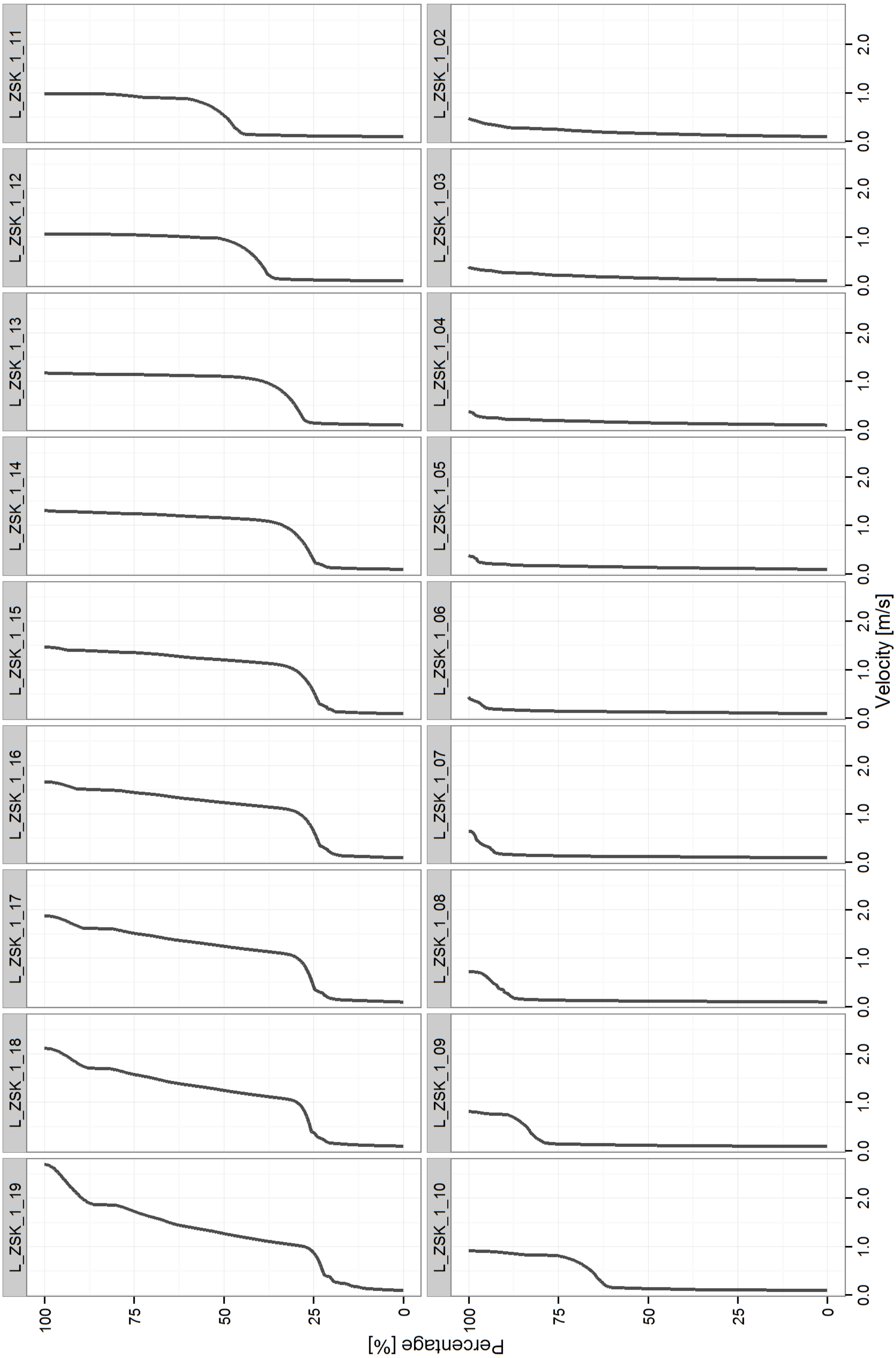
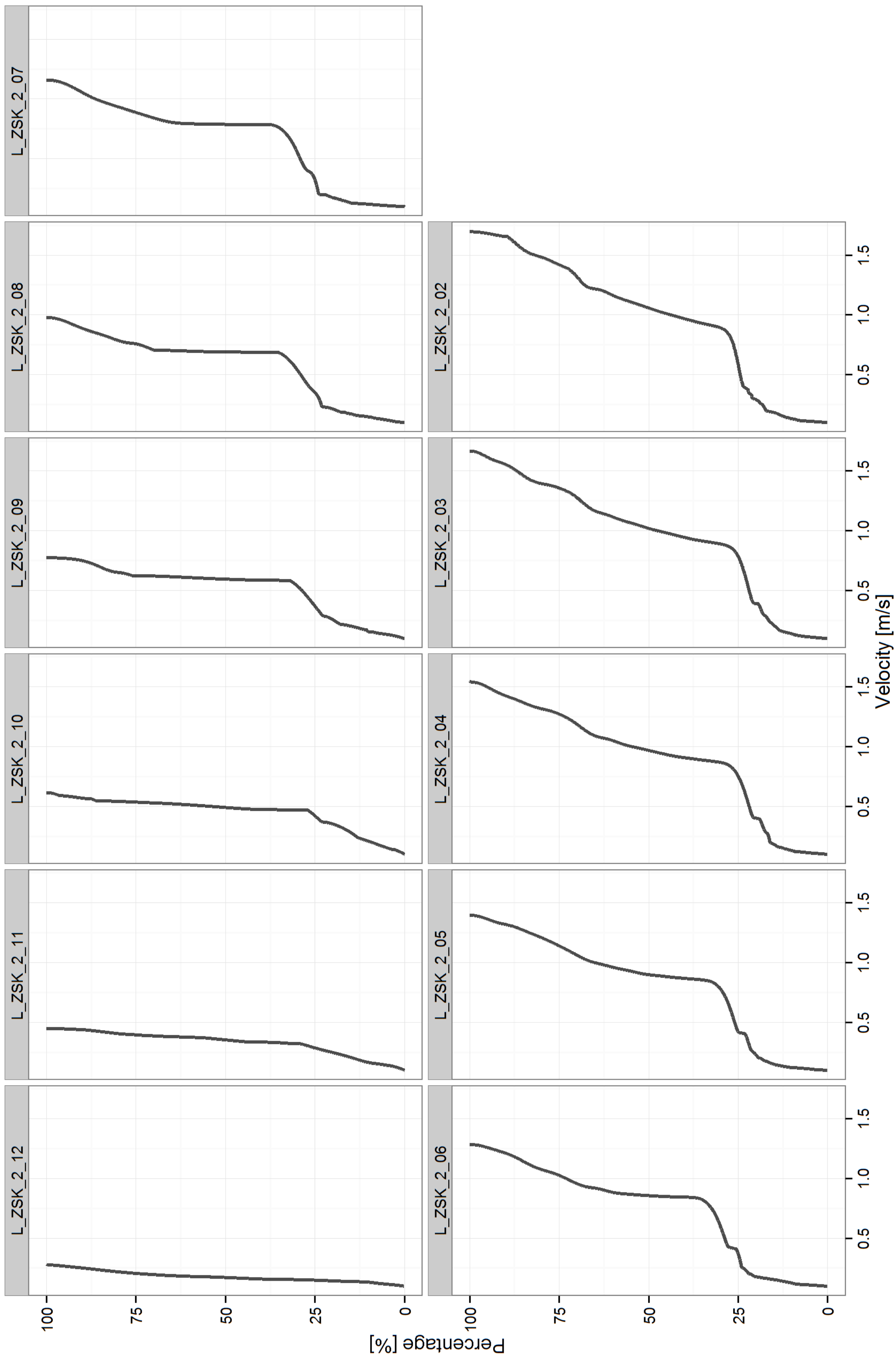
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
Percentage [%]
Velocity [m/s]
100
75
50
25
0
0.1
0.2
0.3
0.4
0.5

Quick refill scenario – ZSK1 full; ZSK2 full





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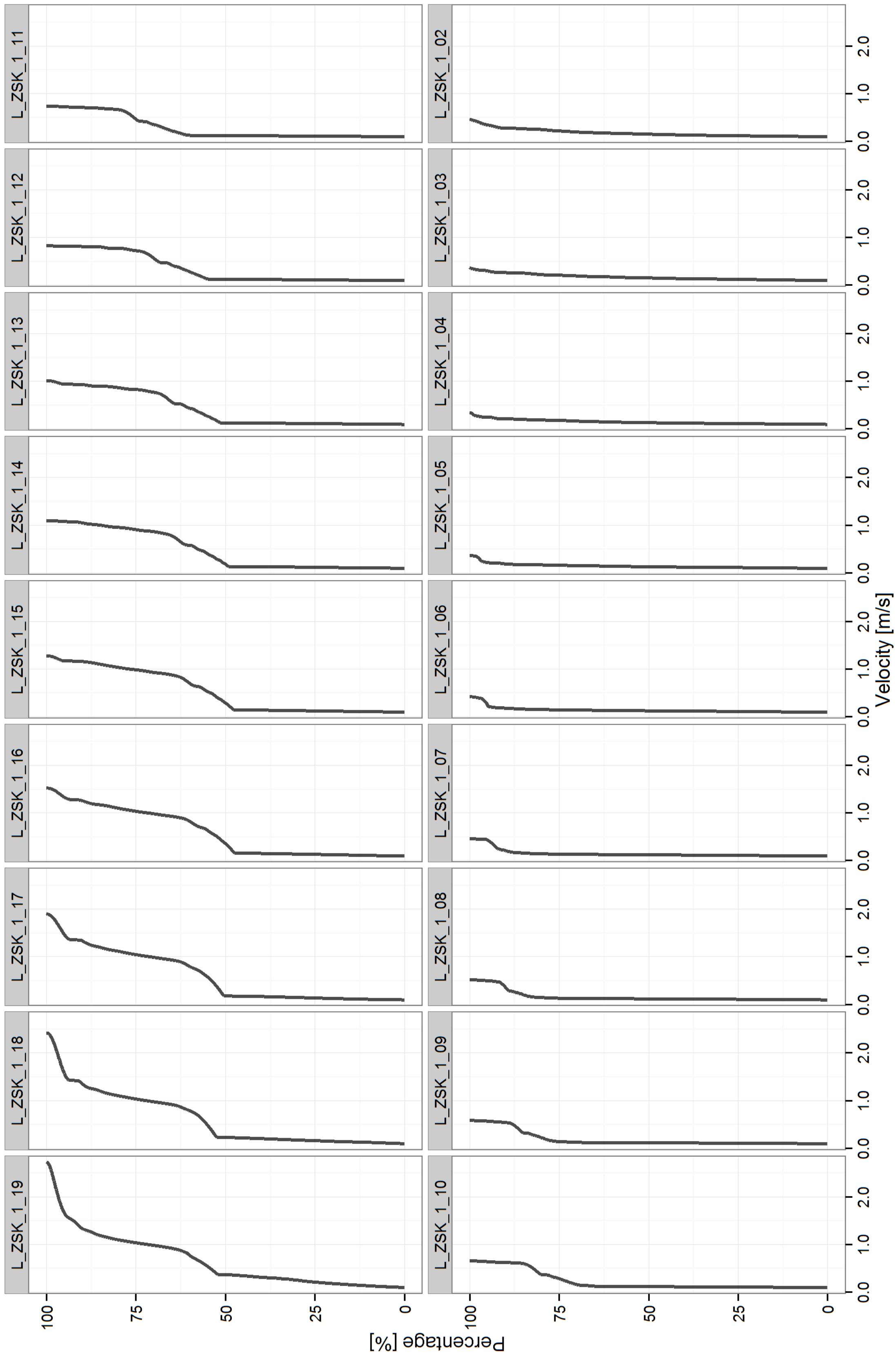
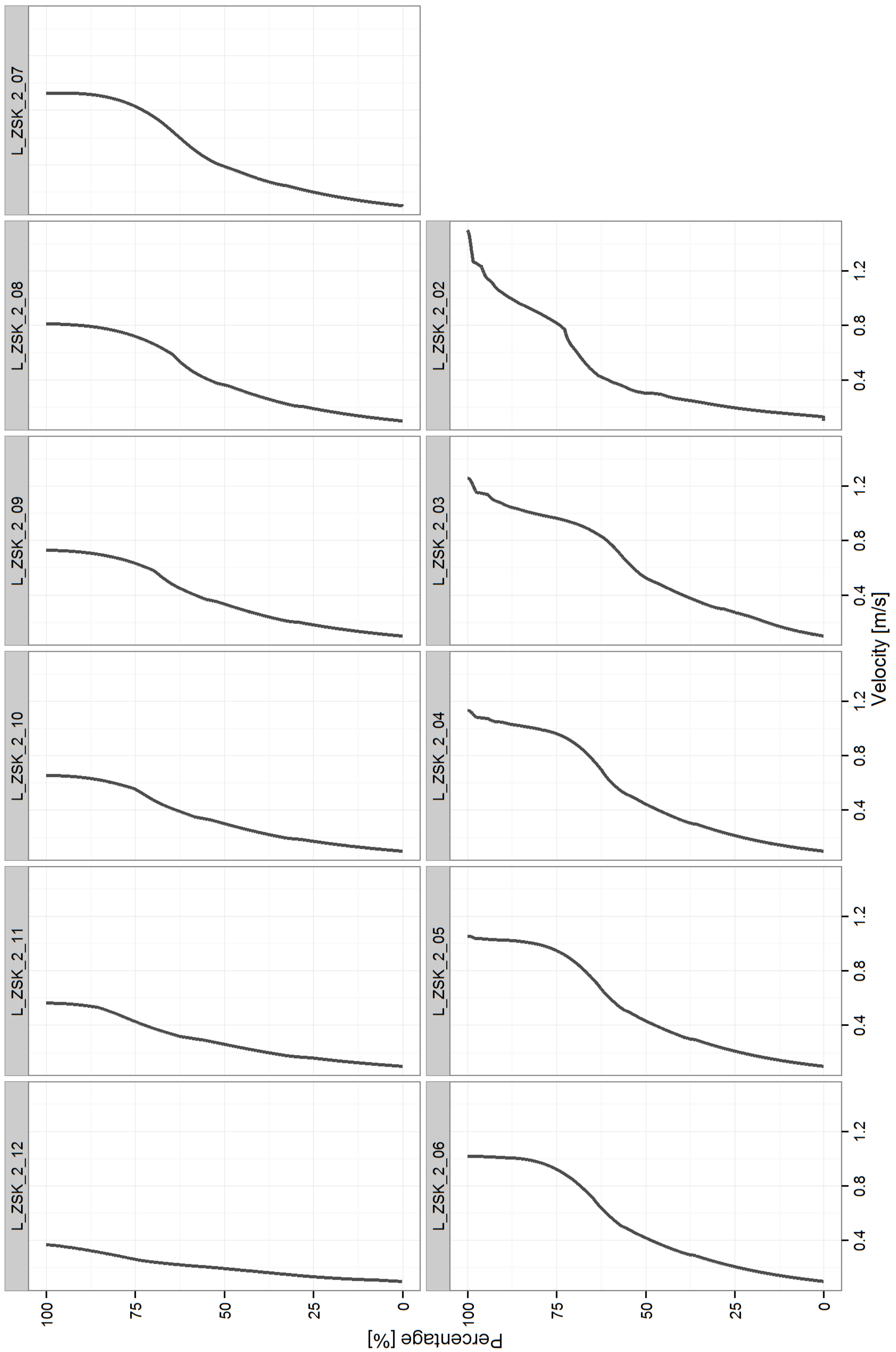
L_ZSK_2_11
L_ZSK_2_12
L_ZSK_2_02
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
Velocity [m/s]
Percentage [%]
0.5
1.0
1.5
100
75
50
25
0

Quick refill scenario – ZSK1 half full; ZSK2 half full





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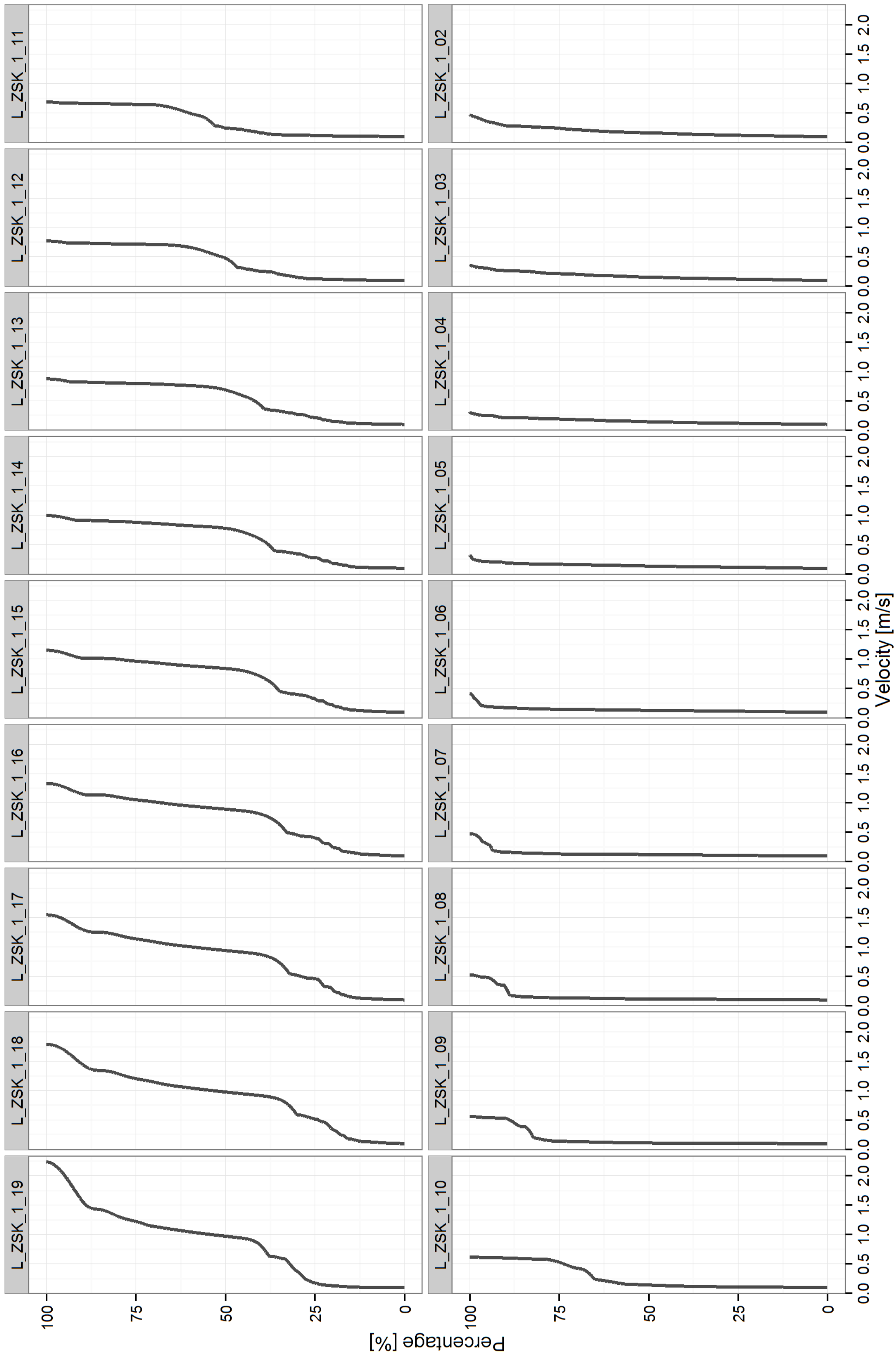
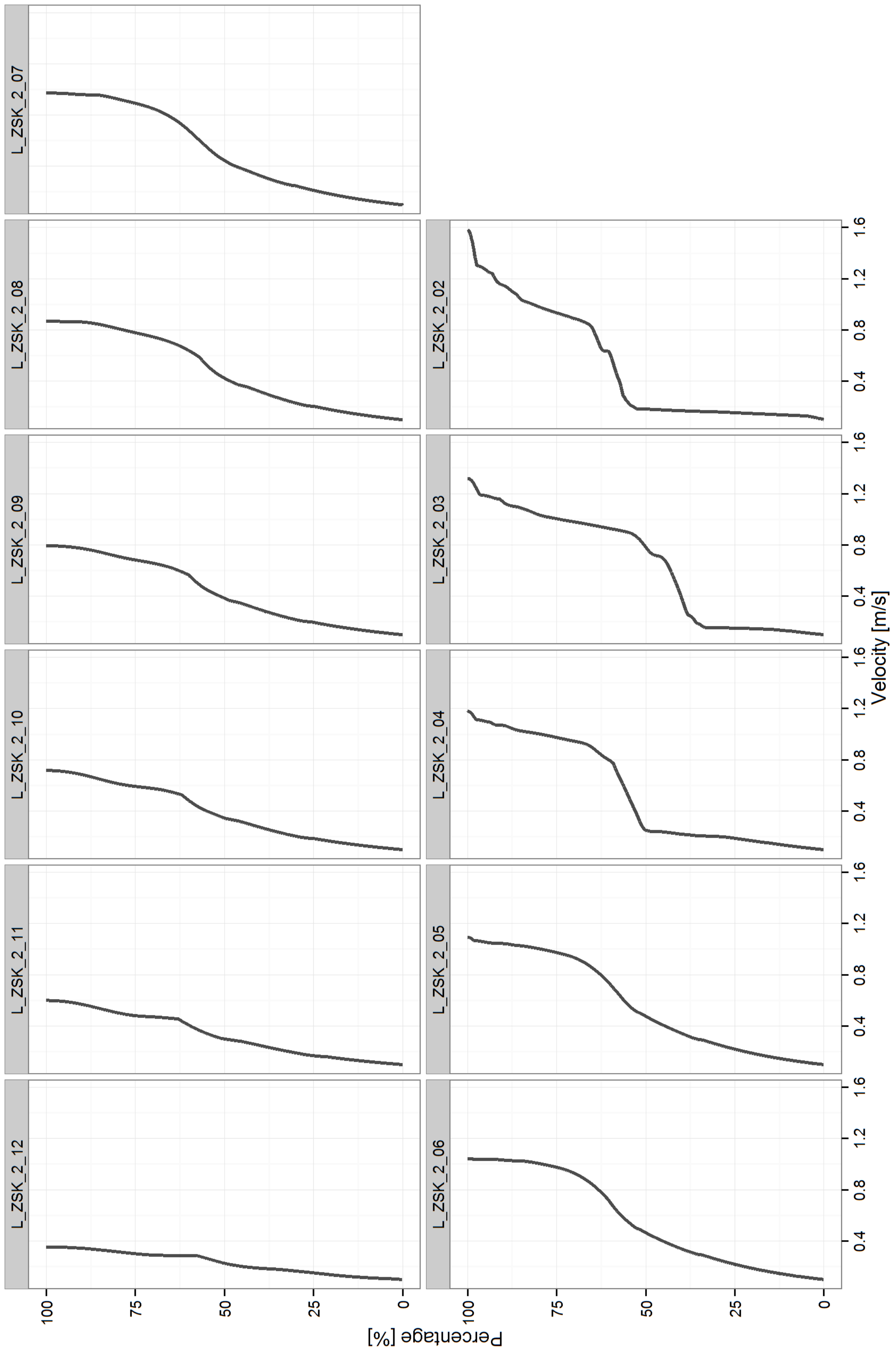
L_ZSK_2_02
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
Percentage [%]
100
75
50
25
0
Velocity [m/s]
0.4
0.8
1.2

Quick refill scenario – ZSK1 full; ZSK2 empty

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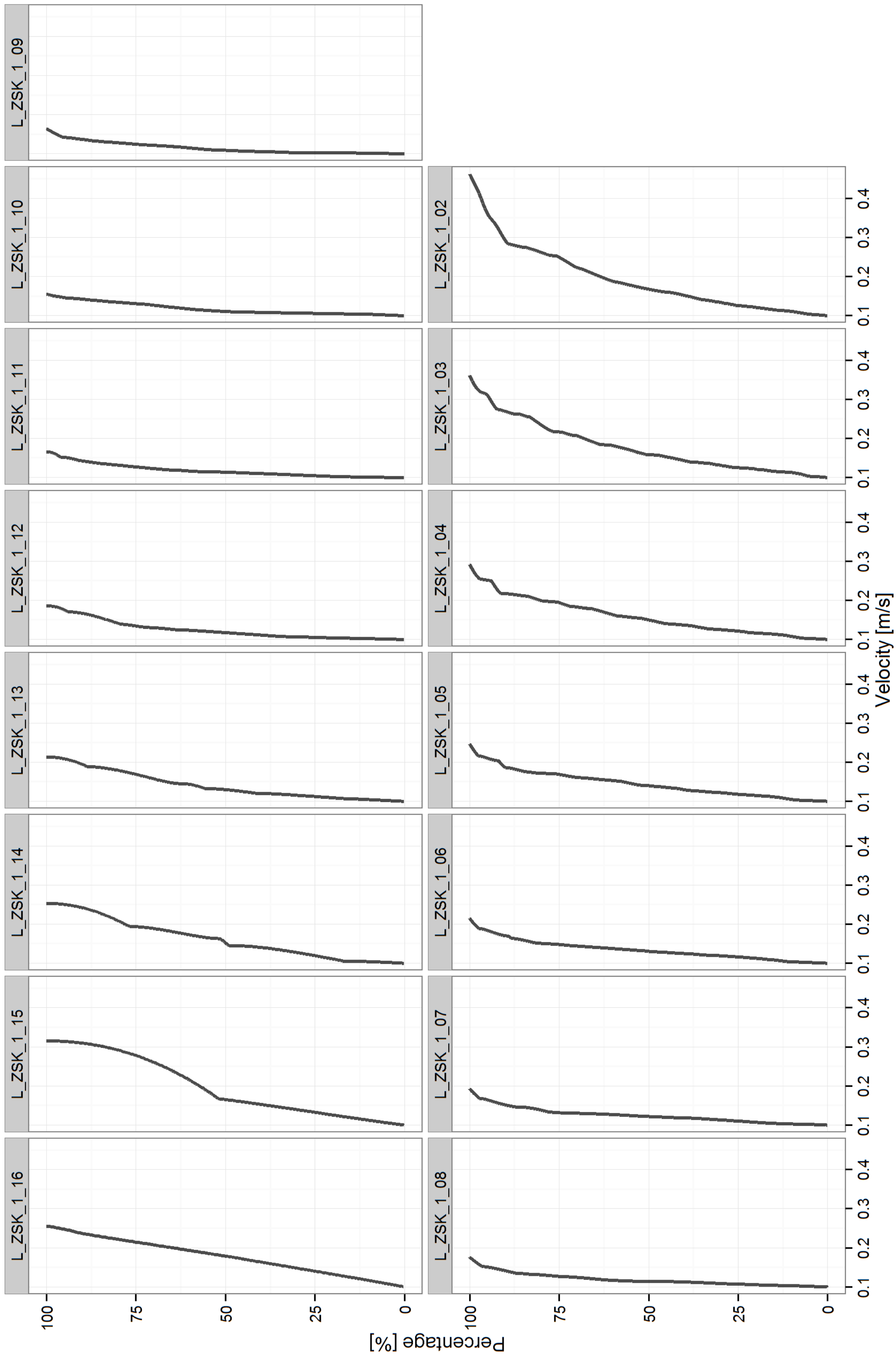
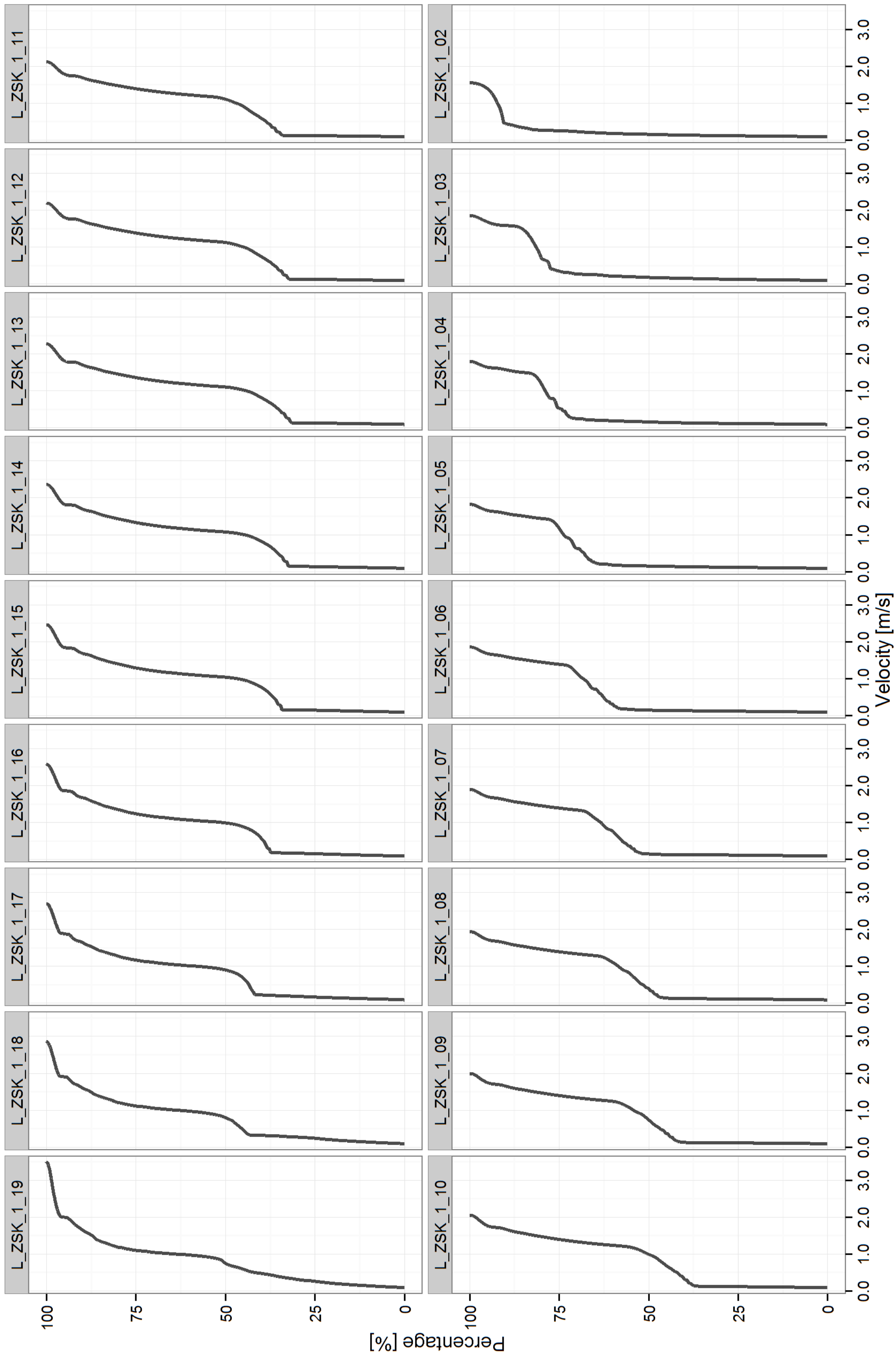
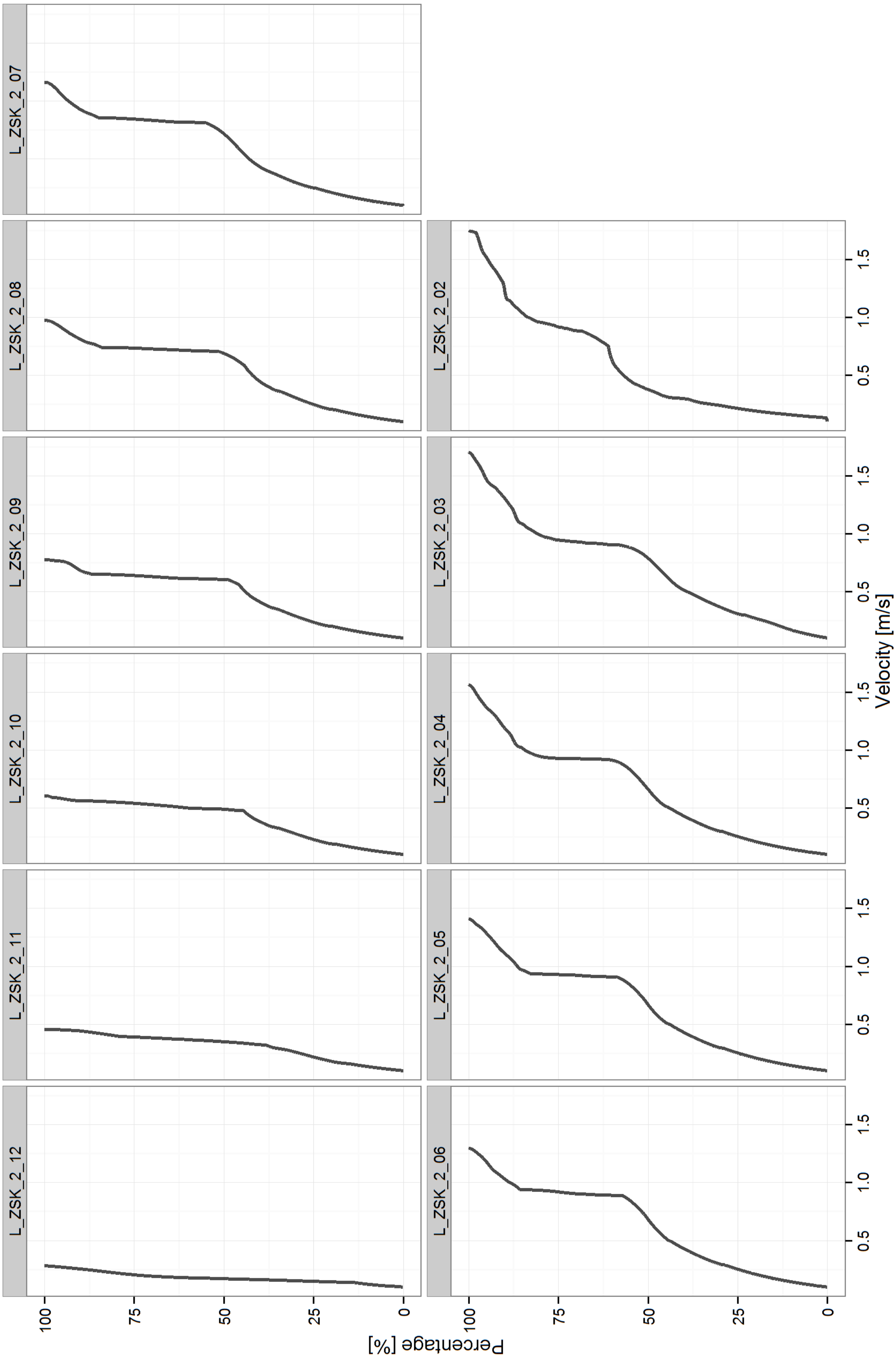
L_ZSK_2_02
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
Percentage [%]
Velocity [m/s]
100
75
50
25
0
0.4
0.8
1.2
1.6

Quick refill scenario – ZSK1 half full; ZSK2 empty



Quick refill scenario – ZSK1 empty; ZSK2 full

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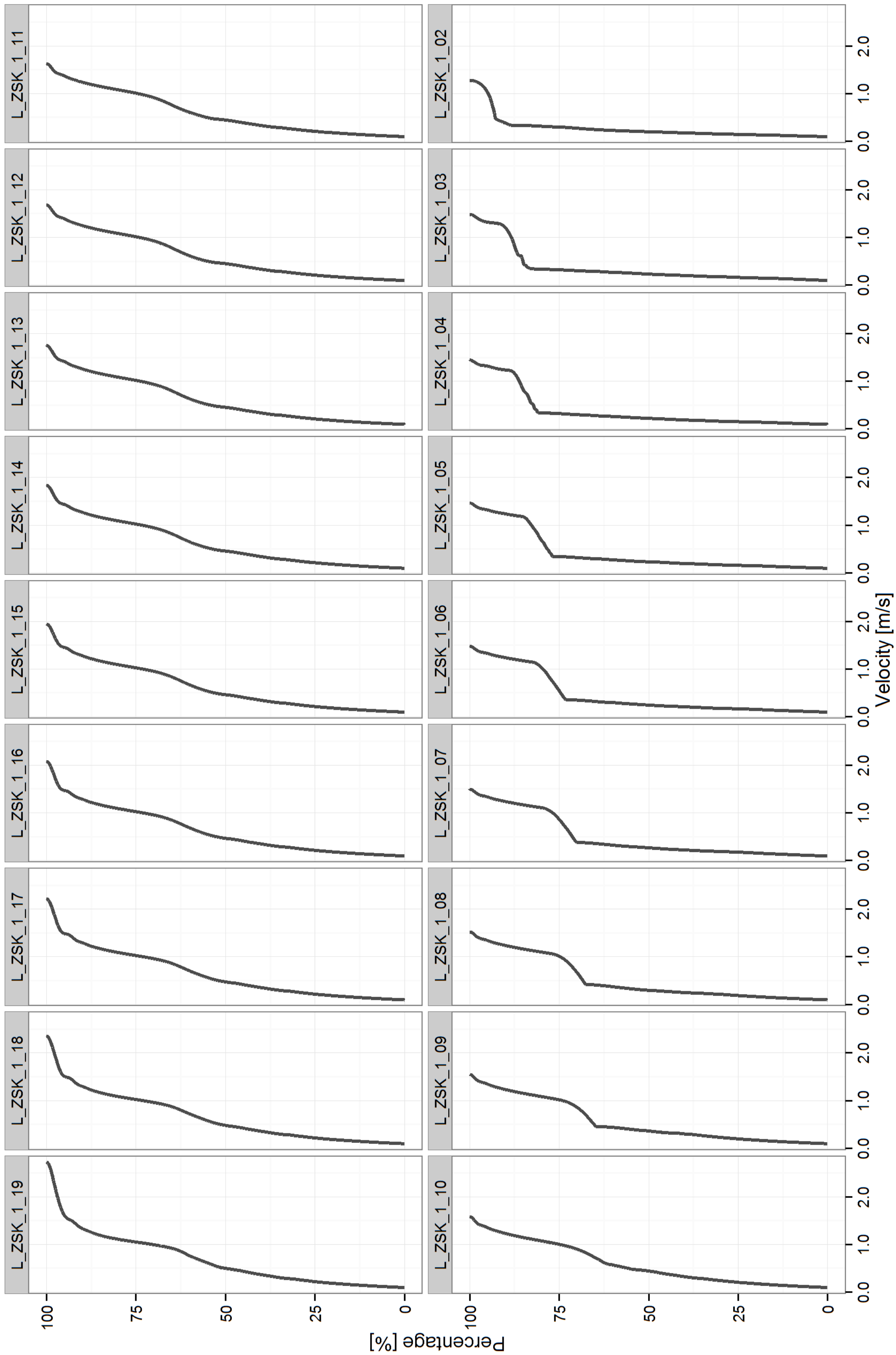
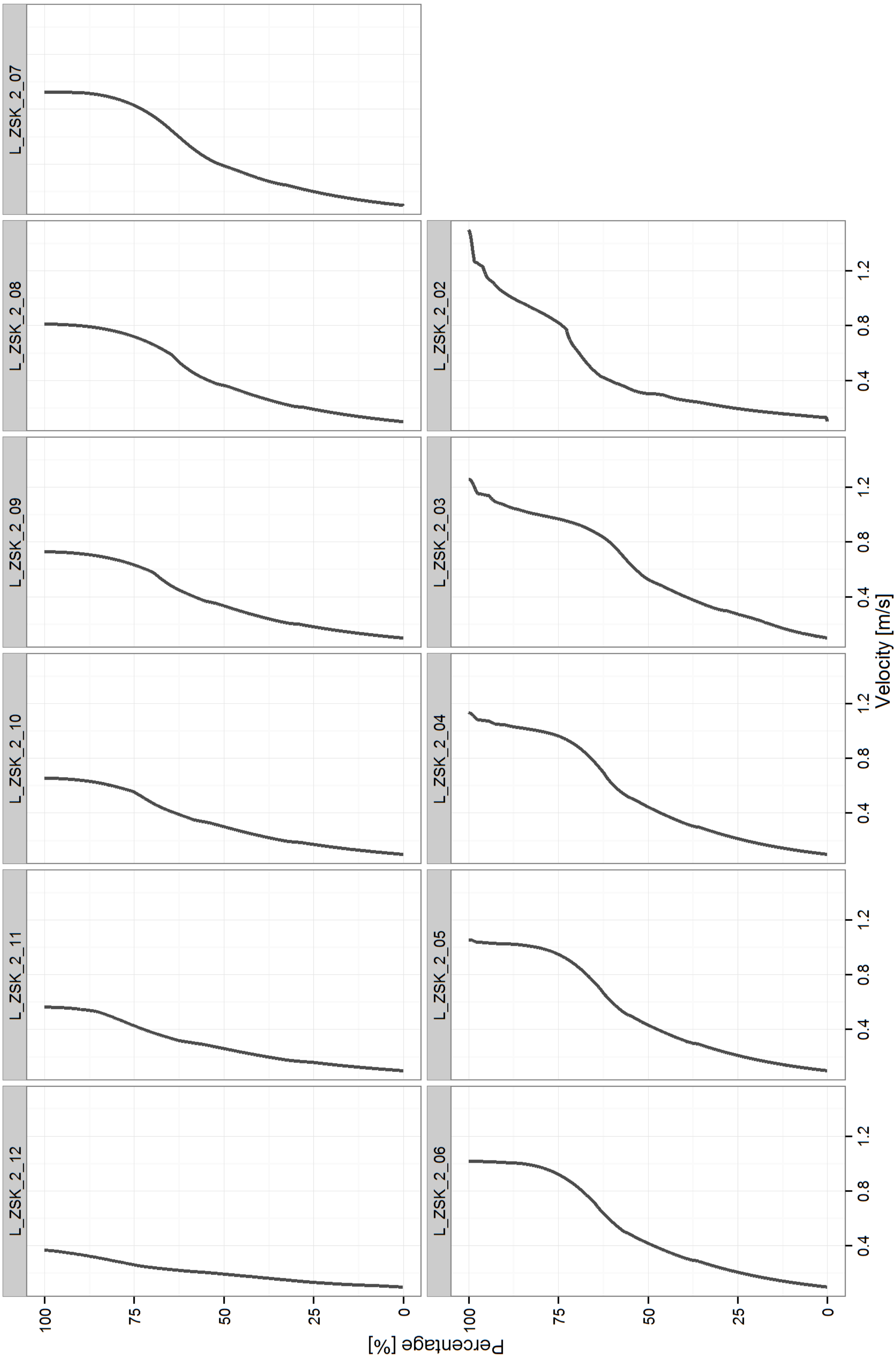
L_ZSK_2_12
L_ZSK_2_02
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
Percentage [%]
Velocity [m/s]
100
75
50
25
0
0.5
1.0
1.5

Quick refill scenario – ZSK1 empty; ZSK2 half full

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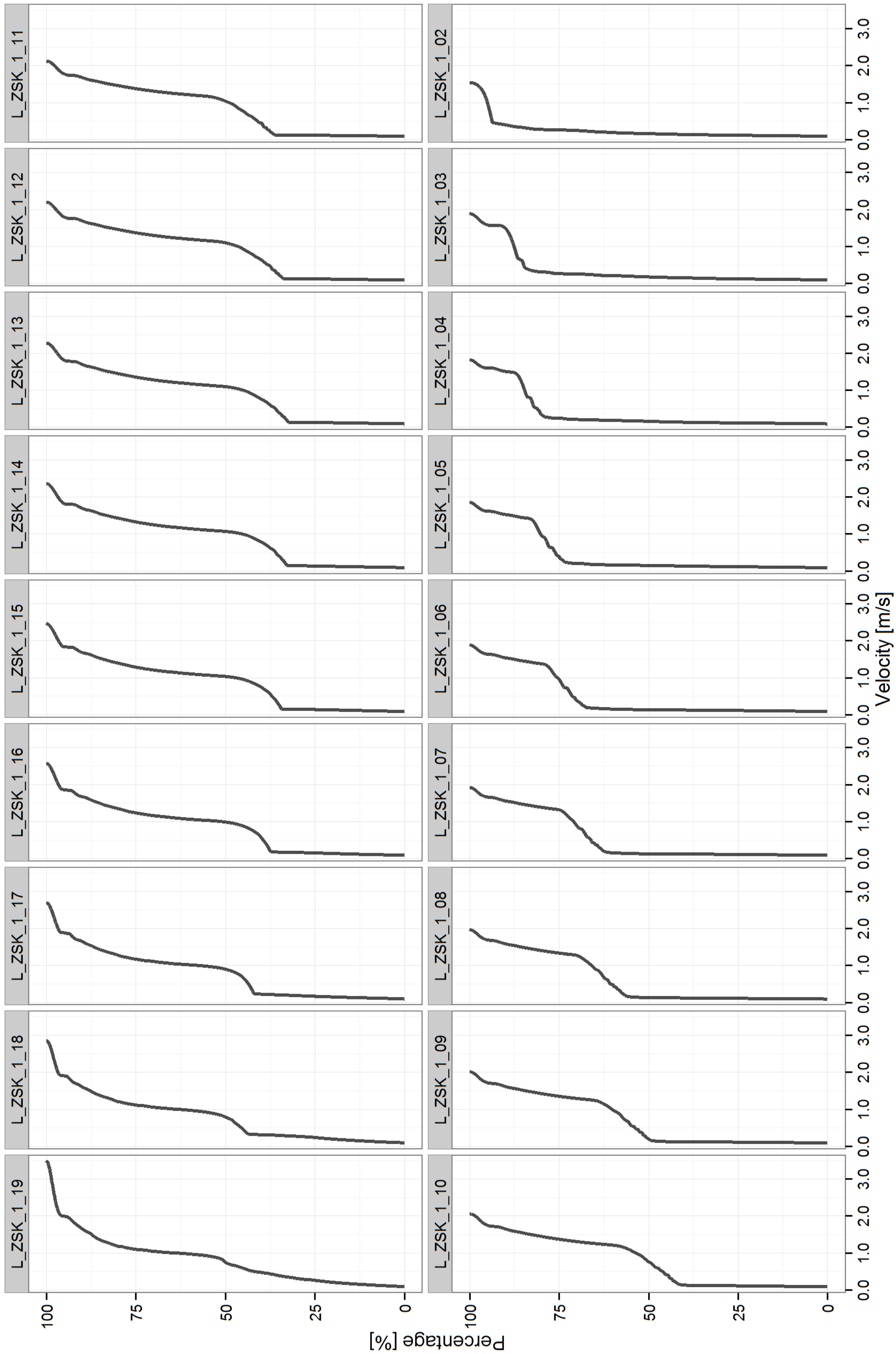
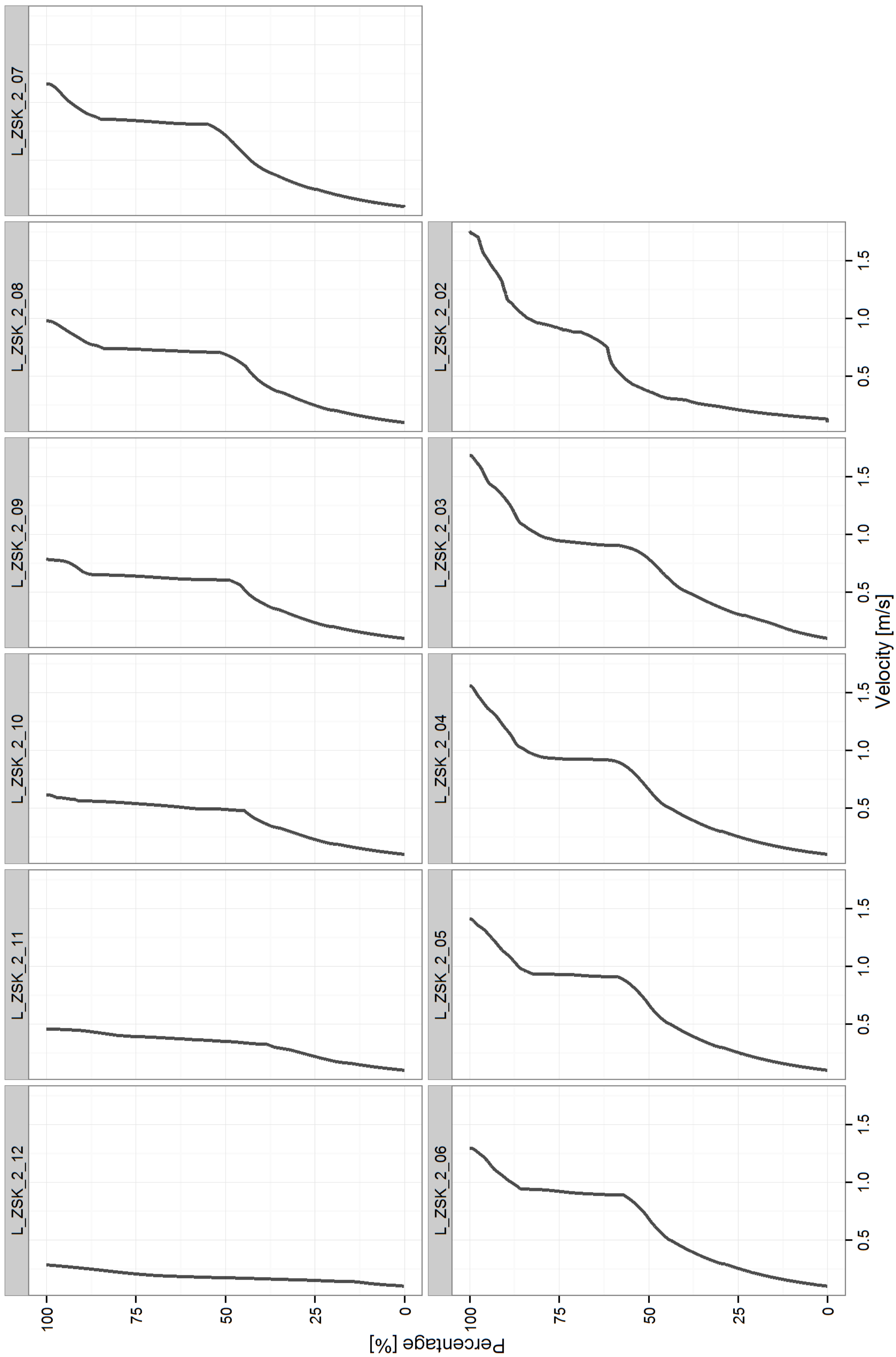
L_ZSK_2_02
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
Percentage [%]
100
75
50
25
0
Velocity [m/s]
0.4
0.8
1.2

Refill-on-empty scenario – ZSK1 full; ZSK2 full

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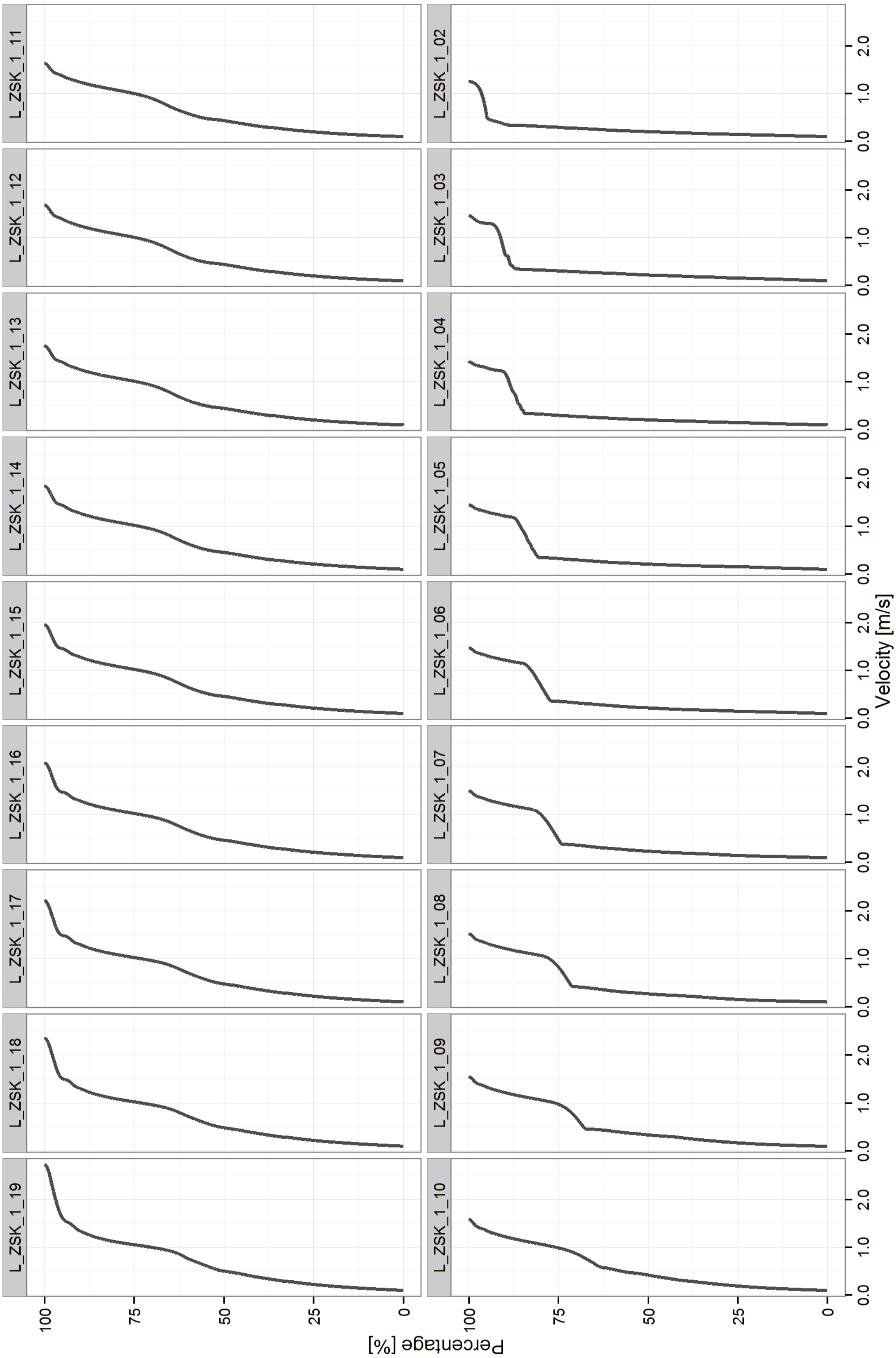
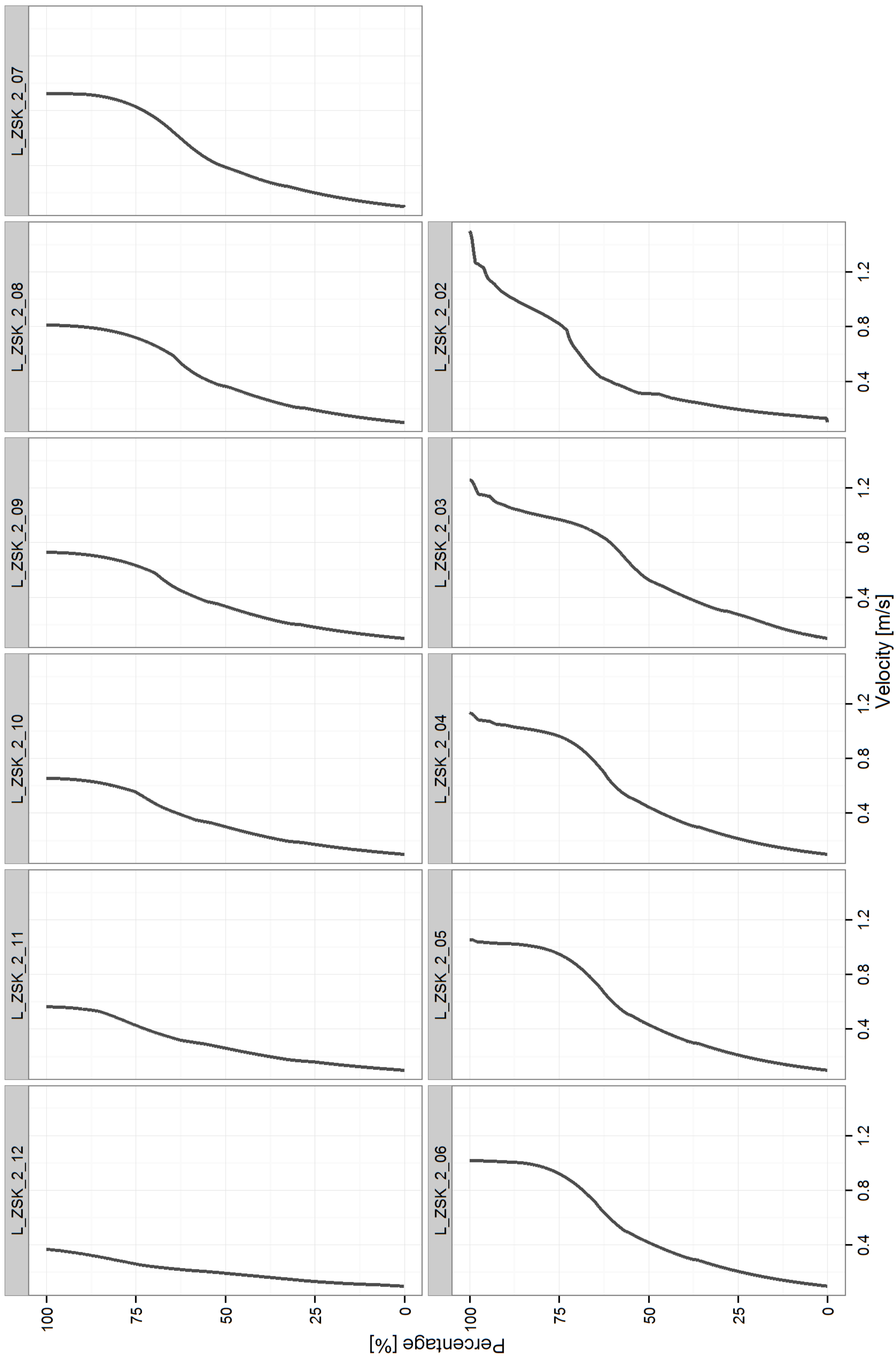
L_ZSK_2_12
L_ZSK_2_02
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
Percentage [%]
Velocity [m/s]
100
75
50
25
0
0.5
1.0
1.5

Refill-on-empty scenario – ZSK1 half full; ZSK2 half full

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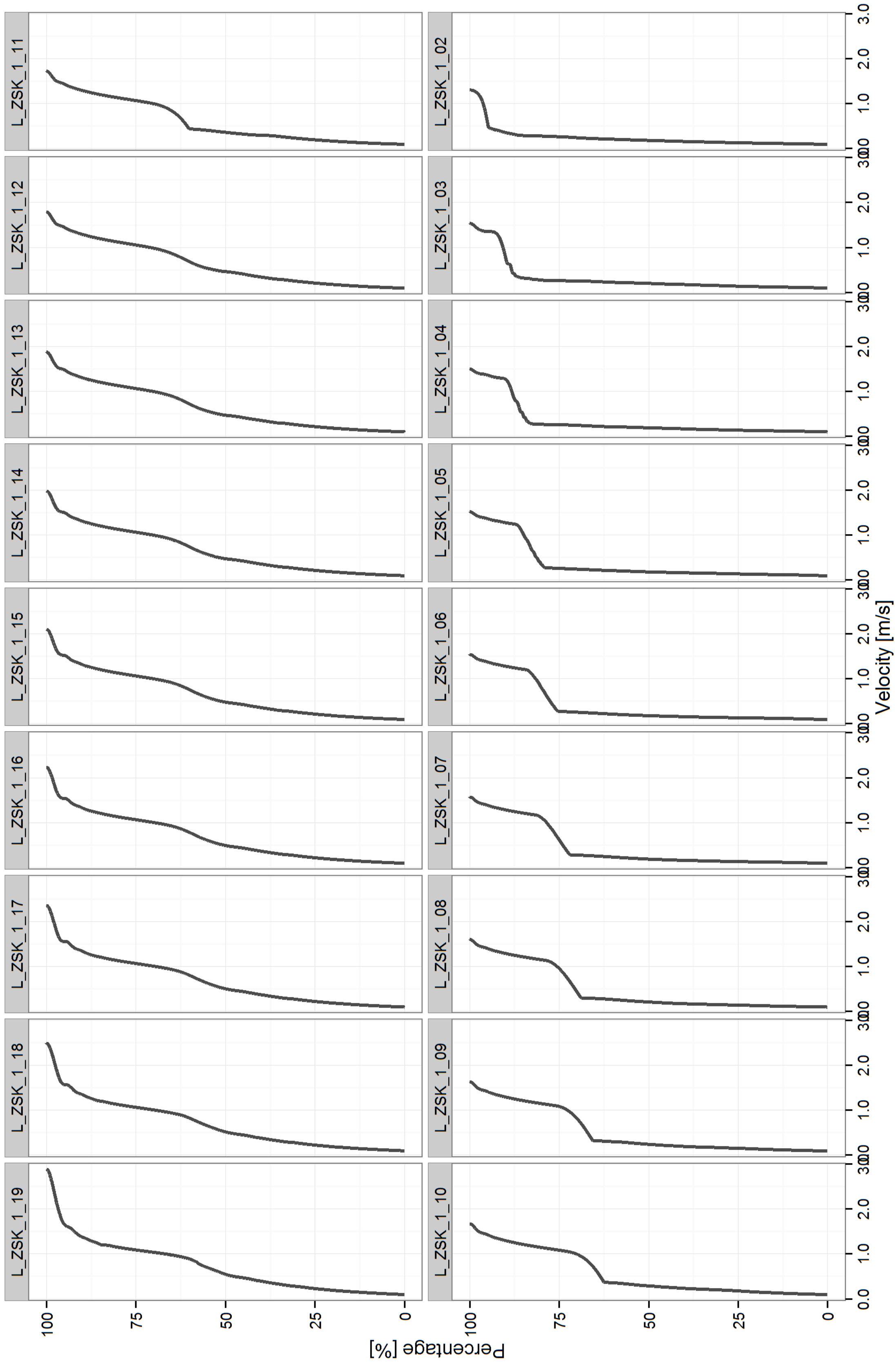
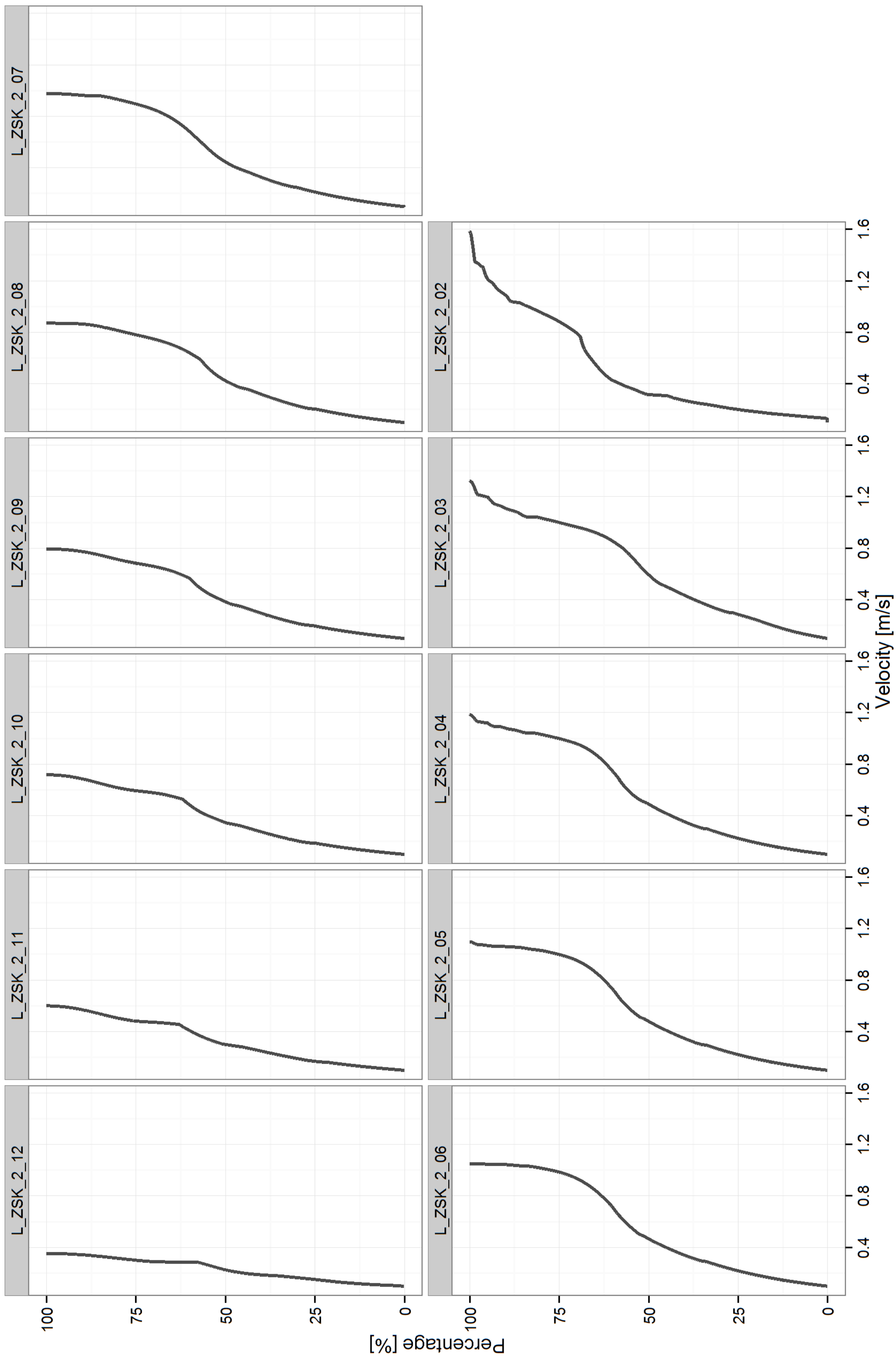
L_ZSK_2_11
L_ZSK_2_12
L_ZSK_2_02
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
Percentage [%]
Velocity [m/s]

Refill-on-empty scenario – ZSK1 full; ZSK2 empty

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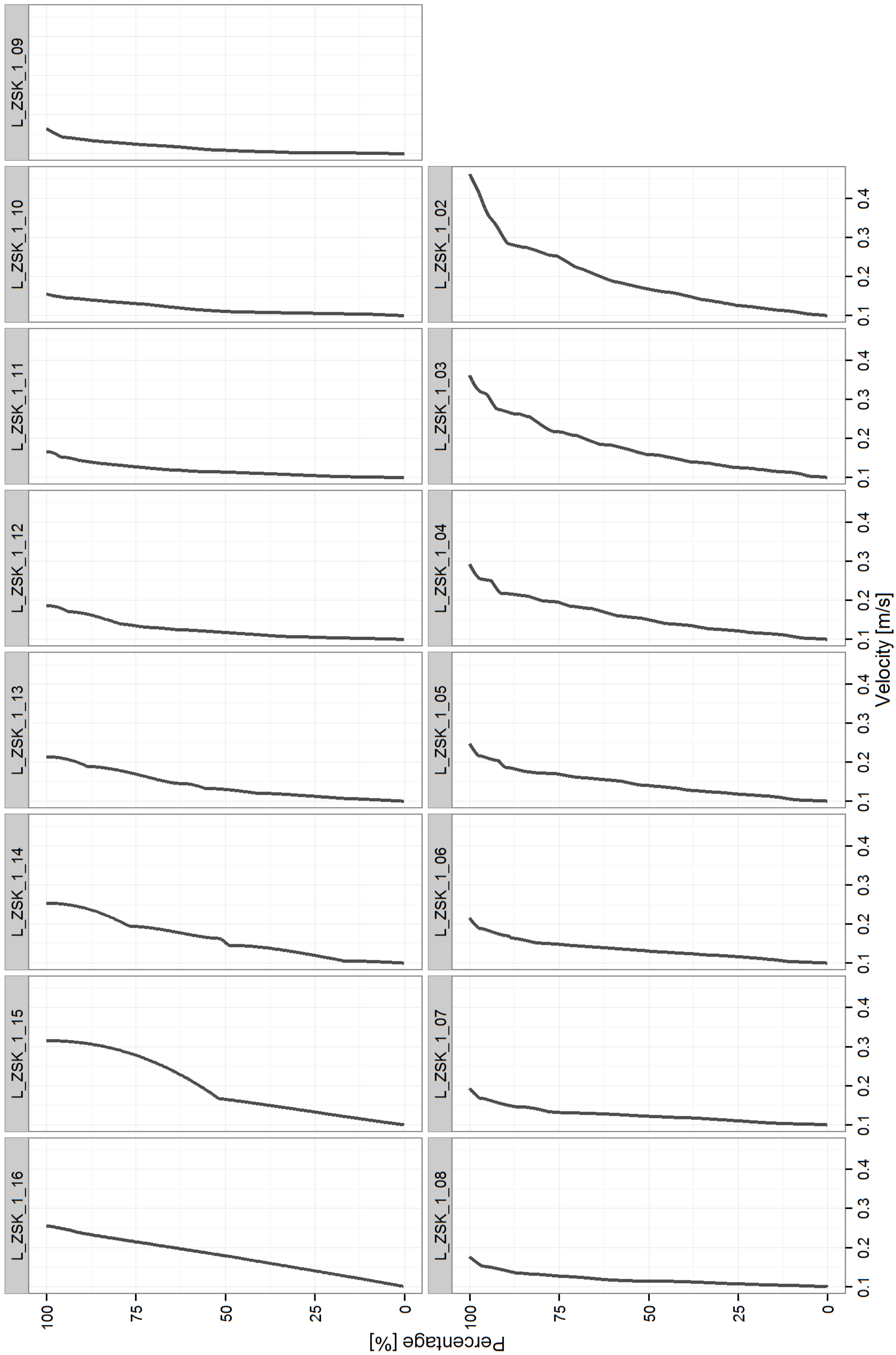
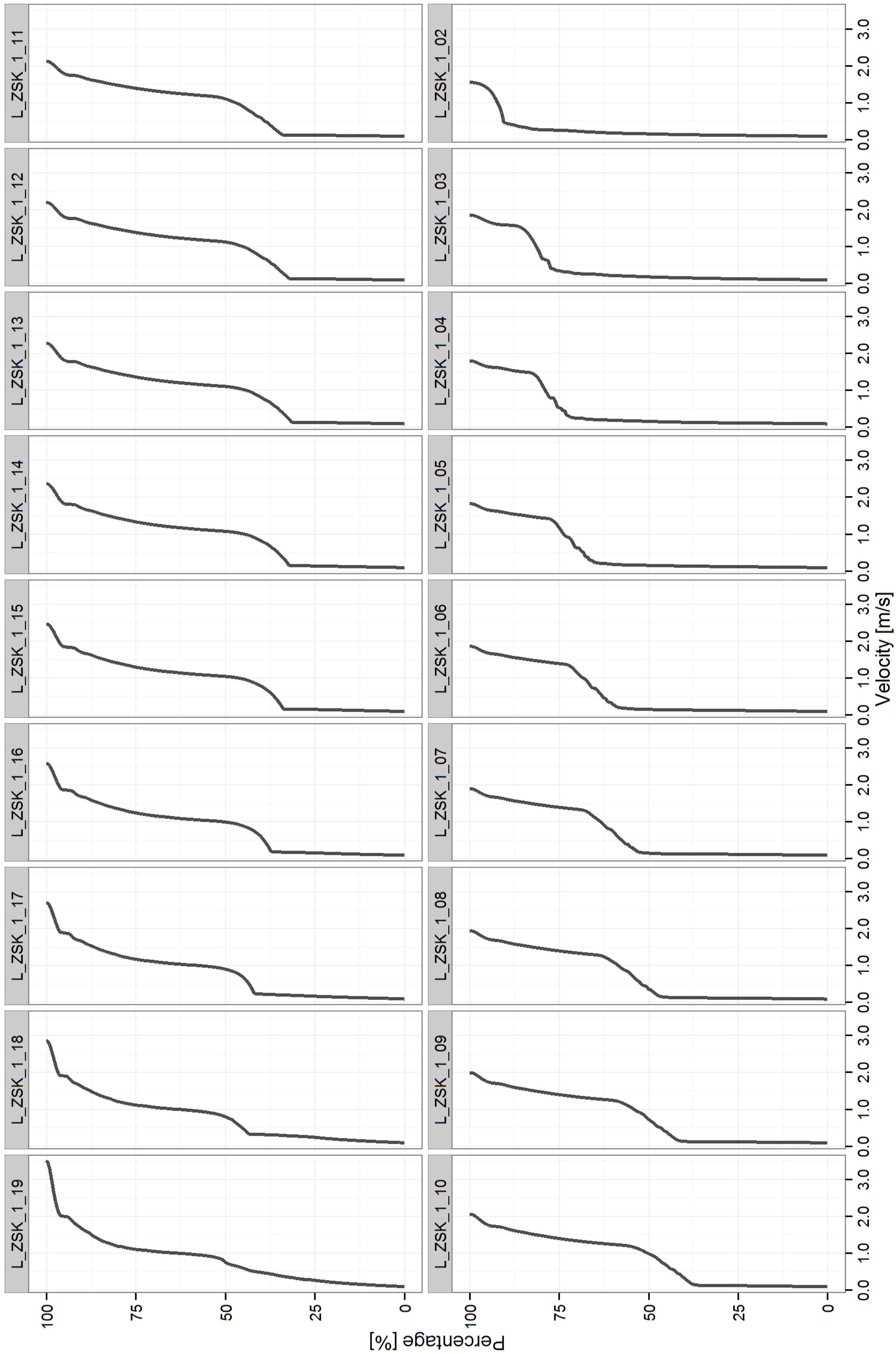
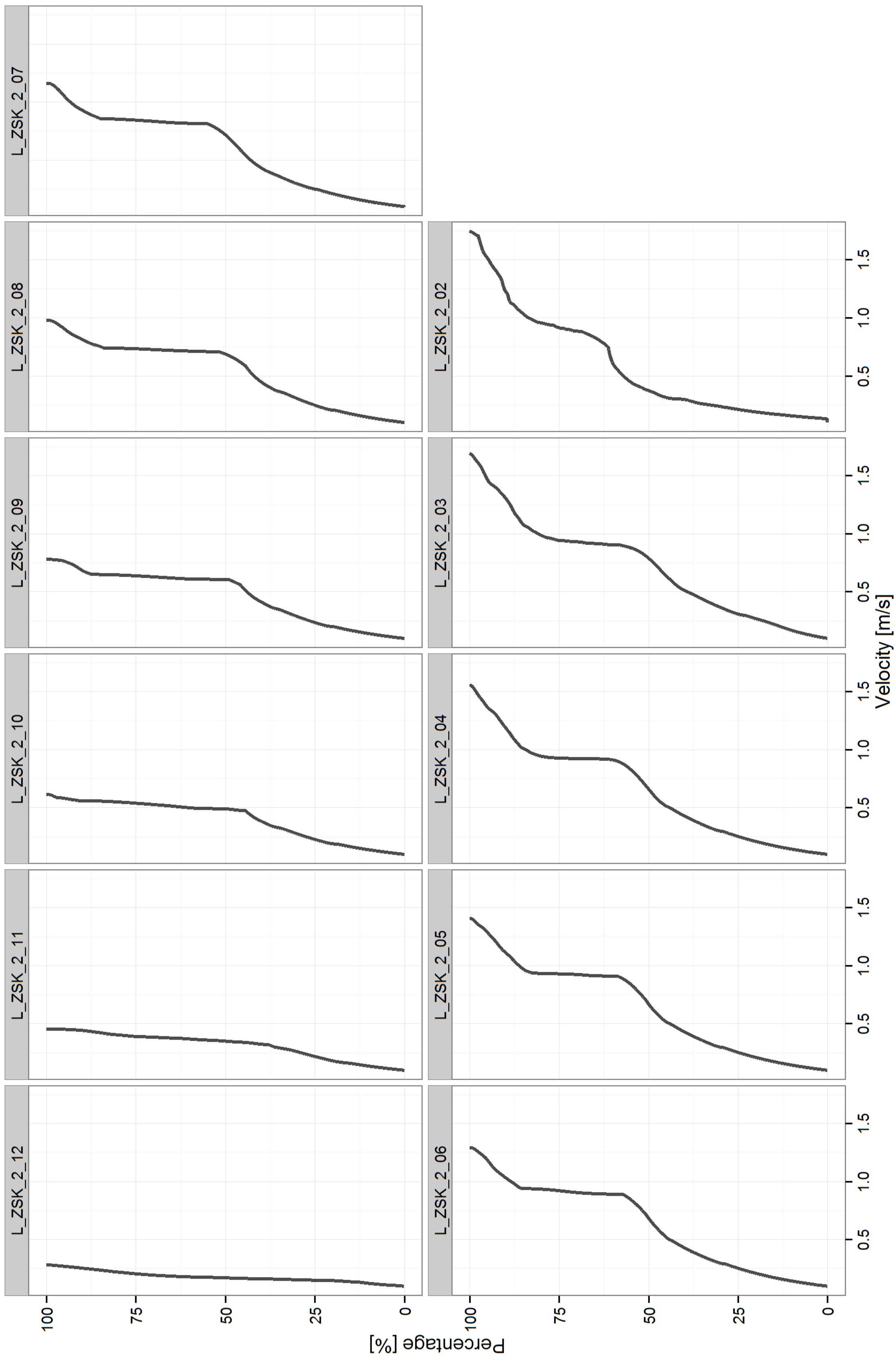
L_ZSK_2_11
L_ZSK_2_12
L_ZSK_2_02
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
Percentage [%]
100
75
50
25
0
Velocity [m/s]
0.4
0.8
1.2
1.6

Refill-on-empty scenario – ZSK1 half full; ZSK2 empty



Refill-on-empty scenario – ZSK1 empty; ZSK2 full

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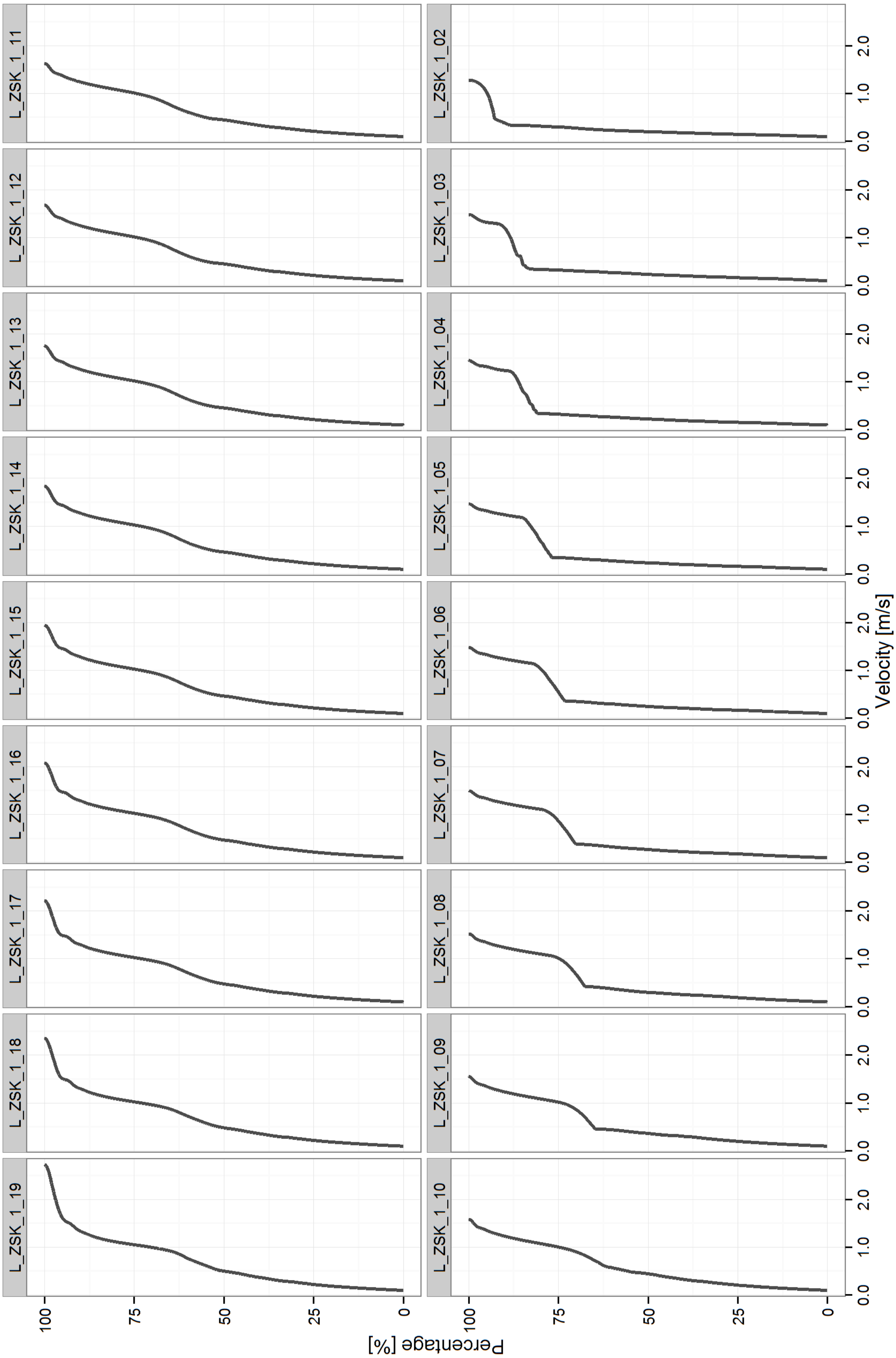
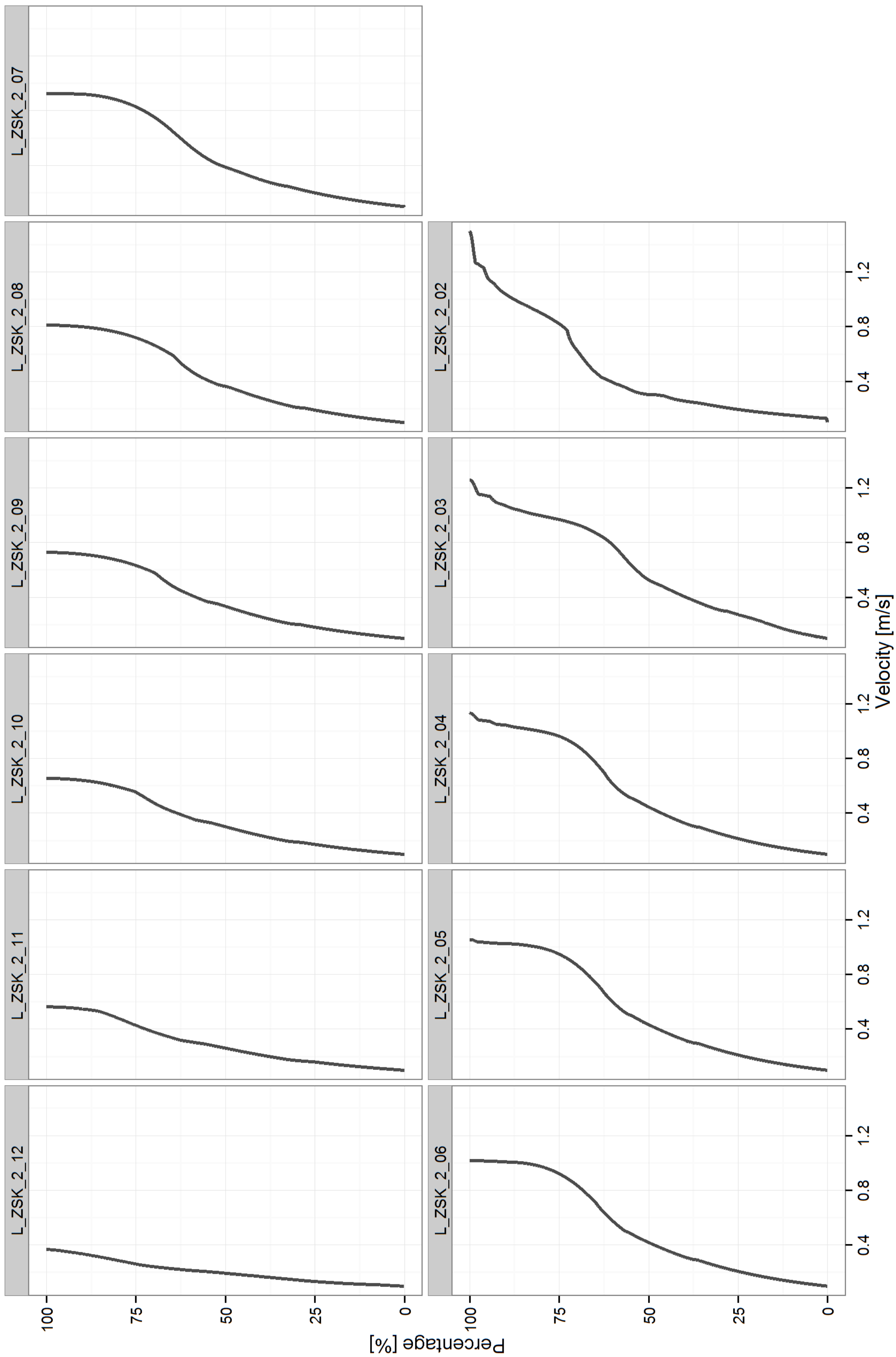
L_ZSK_2_11
L_ZSK_2_12
L_ZSK_2_02
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
Percentage [%]
100
75
50
25
0
Velocity [m/s]
0.5
1.0
1.5

Refill-on-empty scenario – ZSK1 empty; ZSK2 half full

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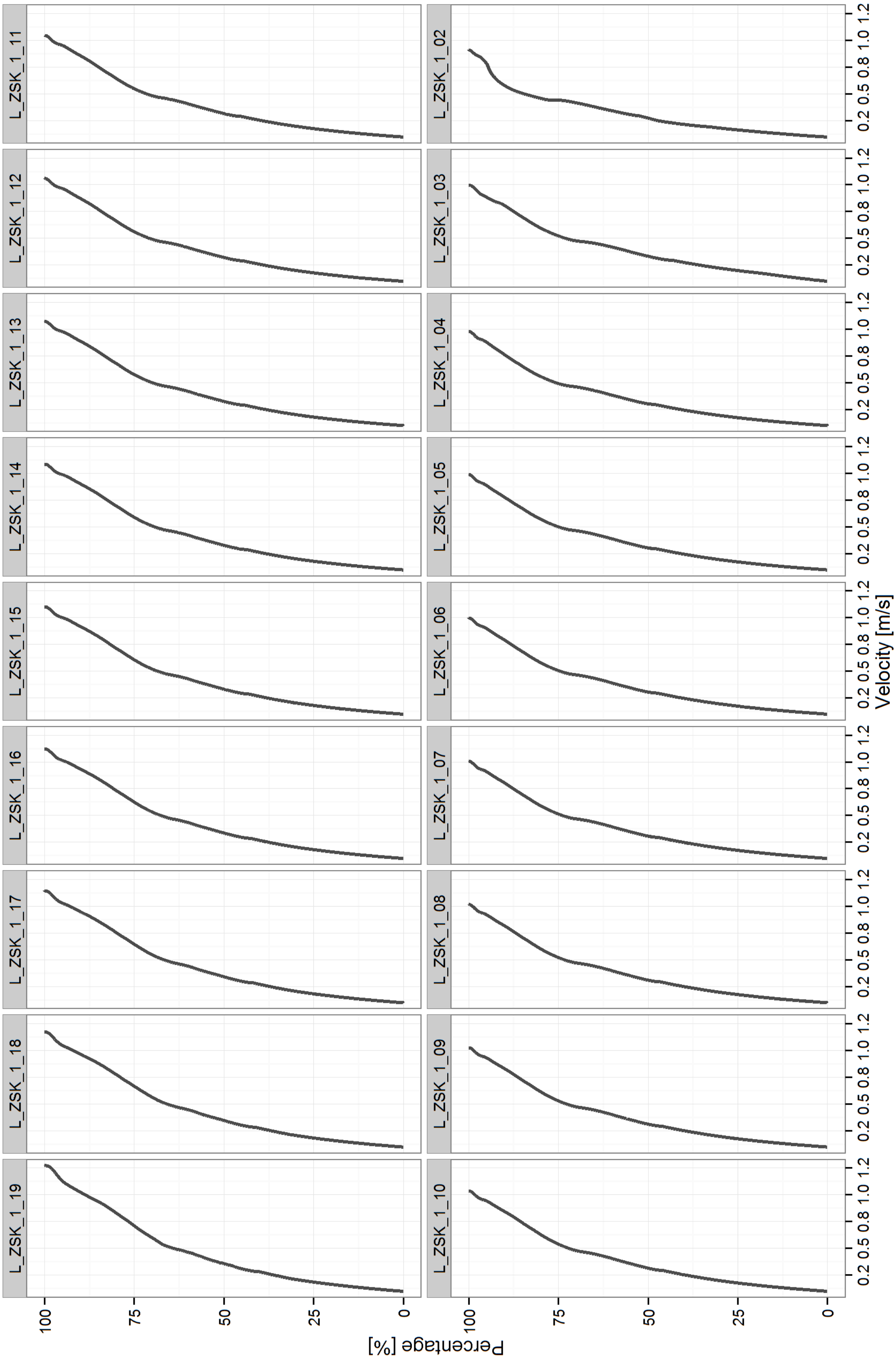
L_ZSK_2_11
L_ZSK_2_12
L_ZSK_2_02
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
100
75
50
25
0
Percentage [%]
0.4
0.8
1.2
Velocity [m/s]

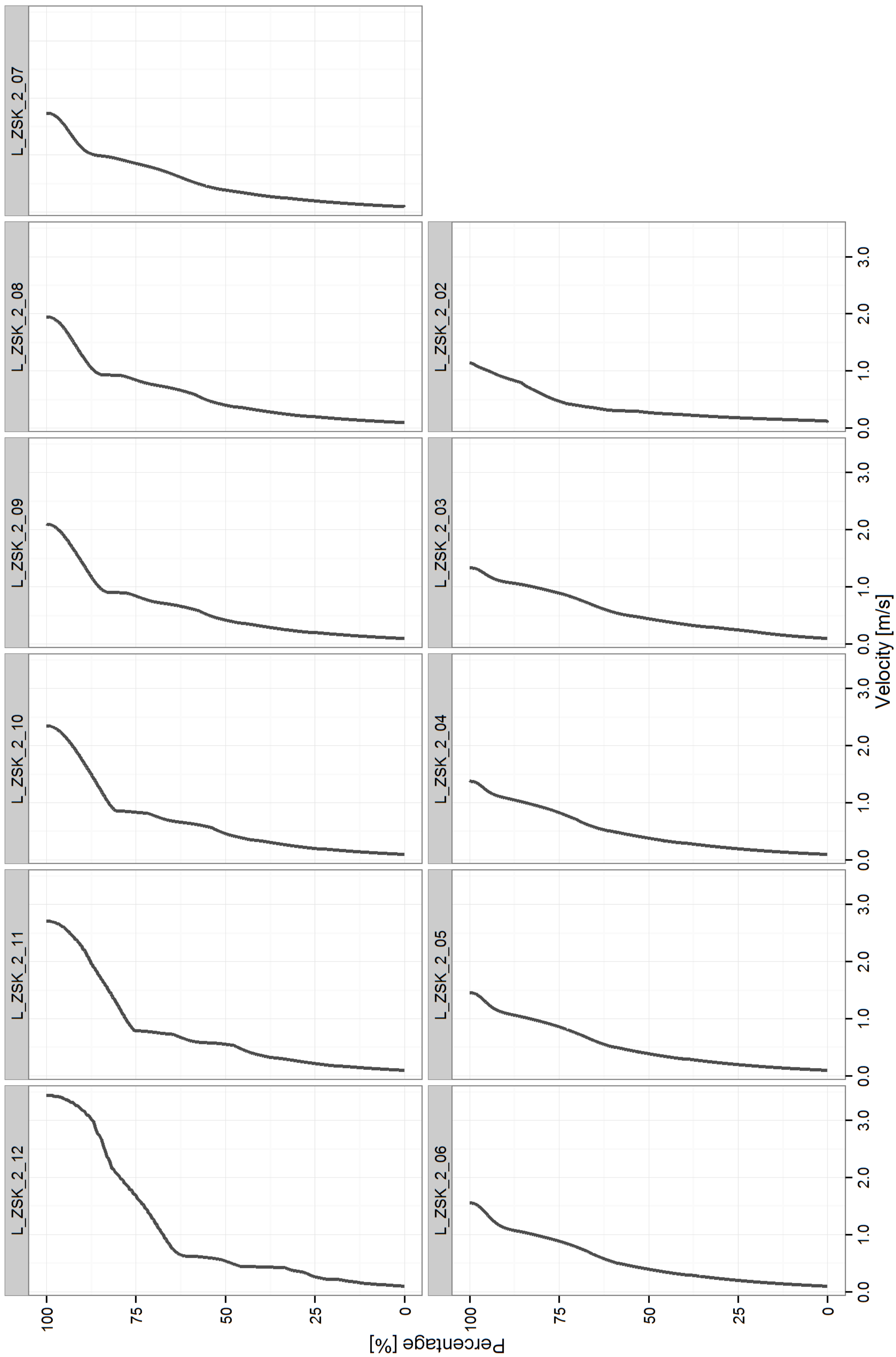
Flushing scenarios

Overview

Name	Description	v_{maxKS2} [m/s]	v_{maxKS1} [m/s]
1W	1 single wave	0.915	0.907
2W	2 consecutive waves	1.060	1.062
3W	3 consecutive waves	1.086	1.111
3W6M	3 consecutive waves with 6-minute intervals	1.060	1.106
3W8M6M	3 consecutive waves with 8- and 6-minute intervals	1.053	1.079
3W10M6M	3 consecutive waves with 10- and 6-minute intervals	1.045	1.061
2WSt	2 consecutive waves Stored in ZSK2 and then released together	1.528	1.102
3WSt	3 consecutive waves Stored in ZSK2 and then released together	1.335	1.159
4WSt	4 consecutive waves Stored in ZSK2 and then released together	1.438	1.210
2WSt1	2 consecutive waves Stored in ZSK2, released together and caught by a 3rd right before KS1	1.358	0.922

One single wave

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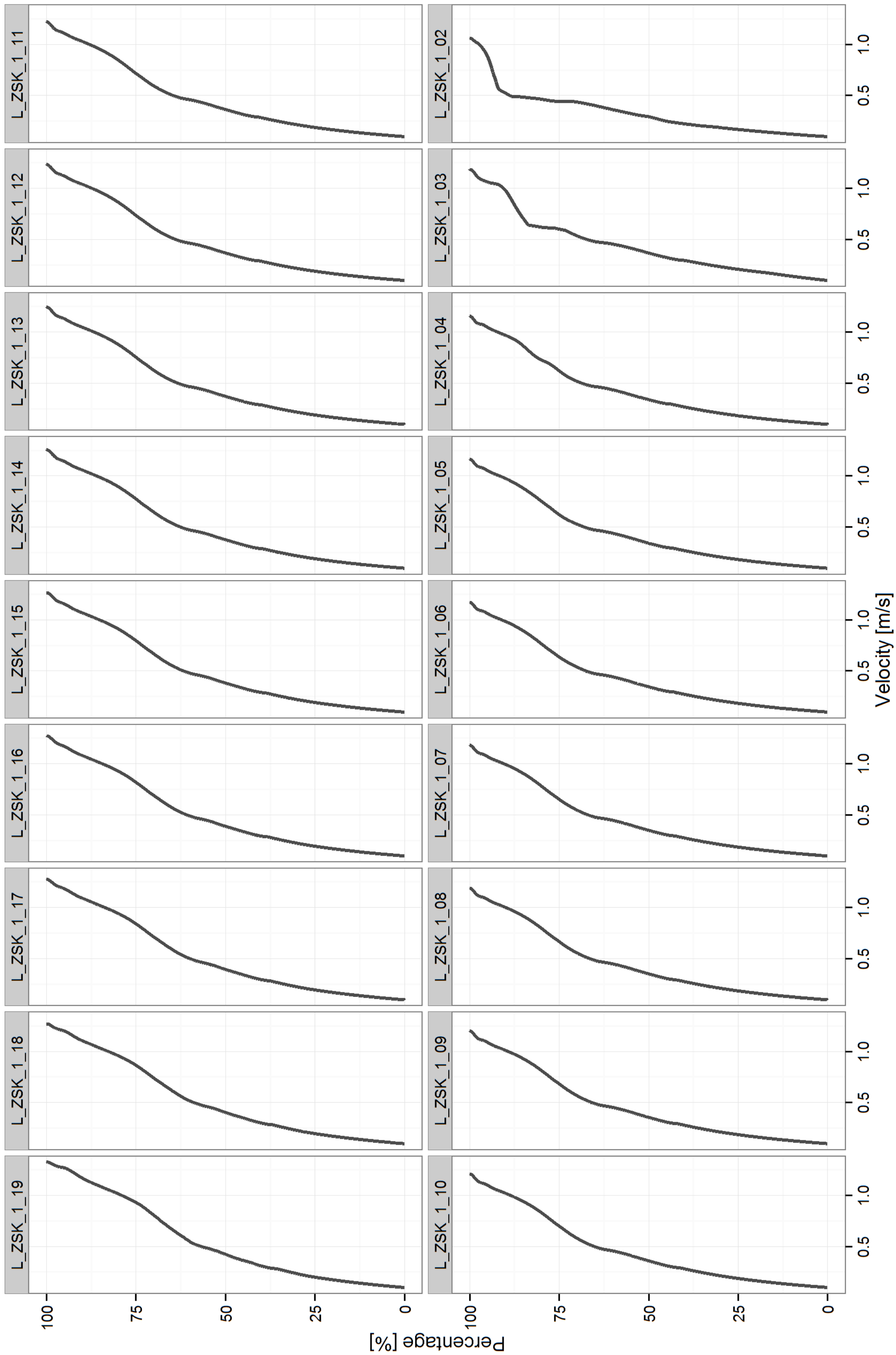
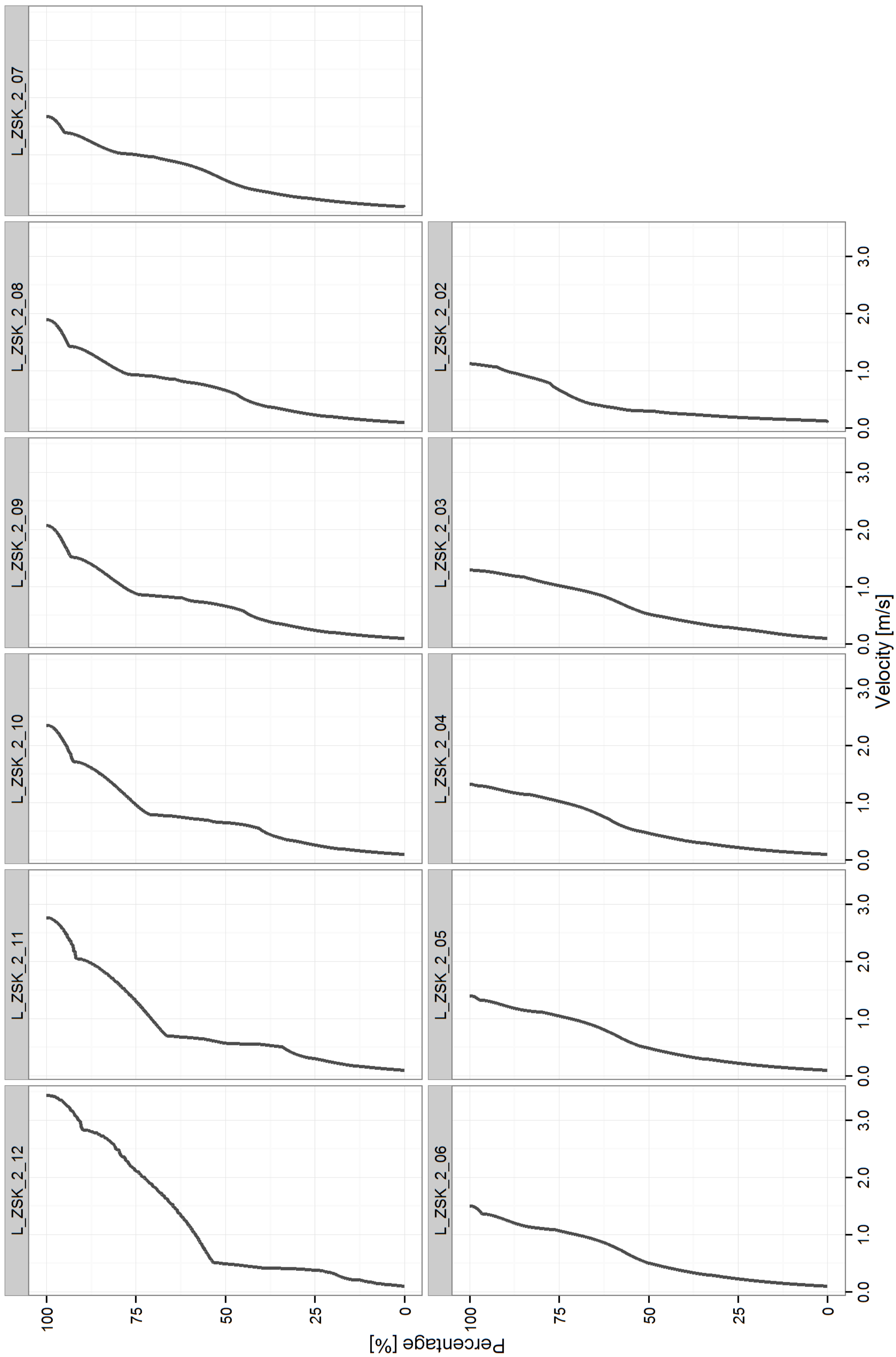
L_ZSK_2_11
L_ZSK_2_12
L_ZSK_2_02
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
Percentage [%]
Velocity [m/s]

Two consecutive waves

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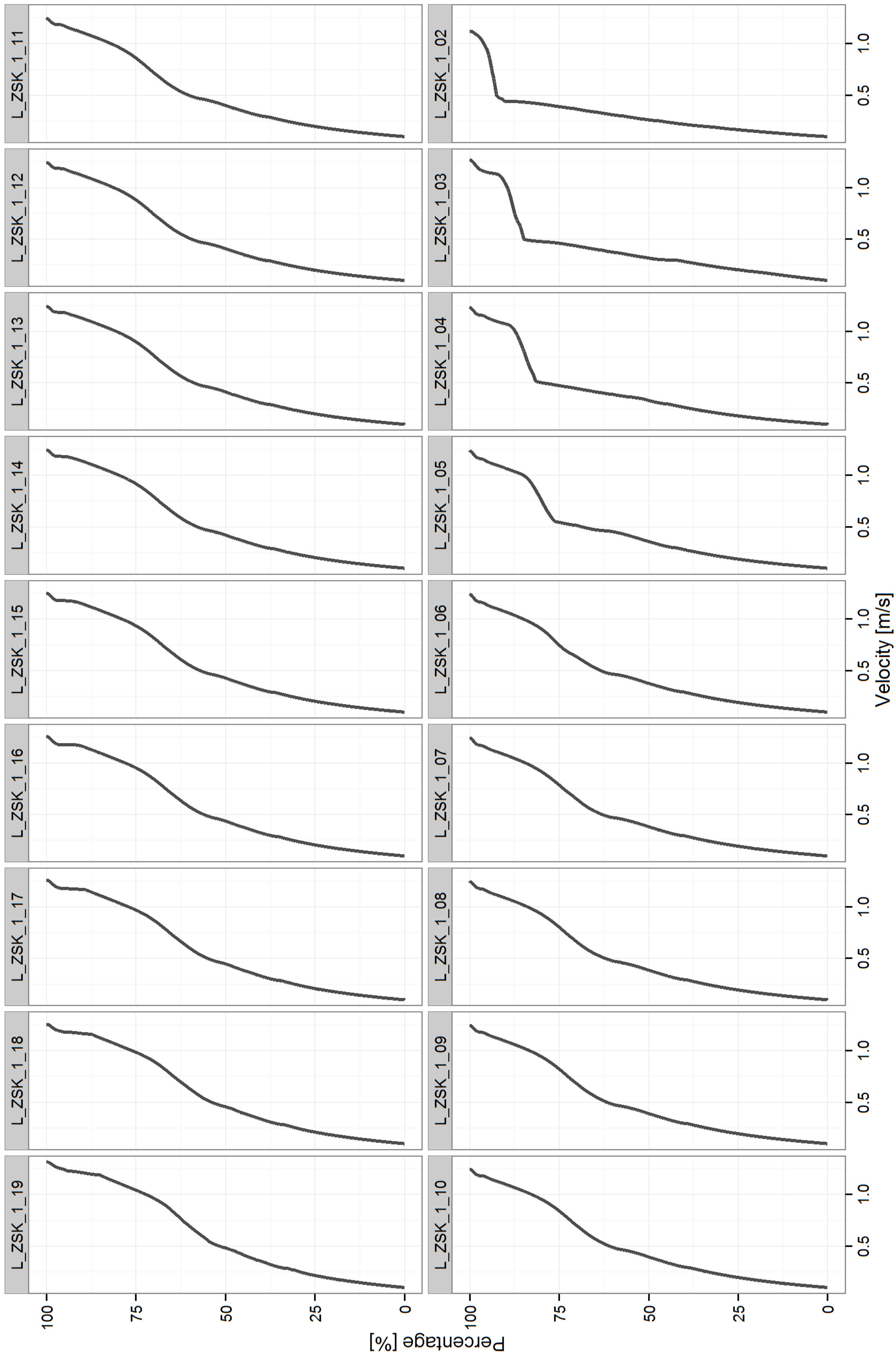
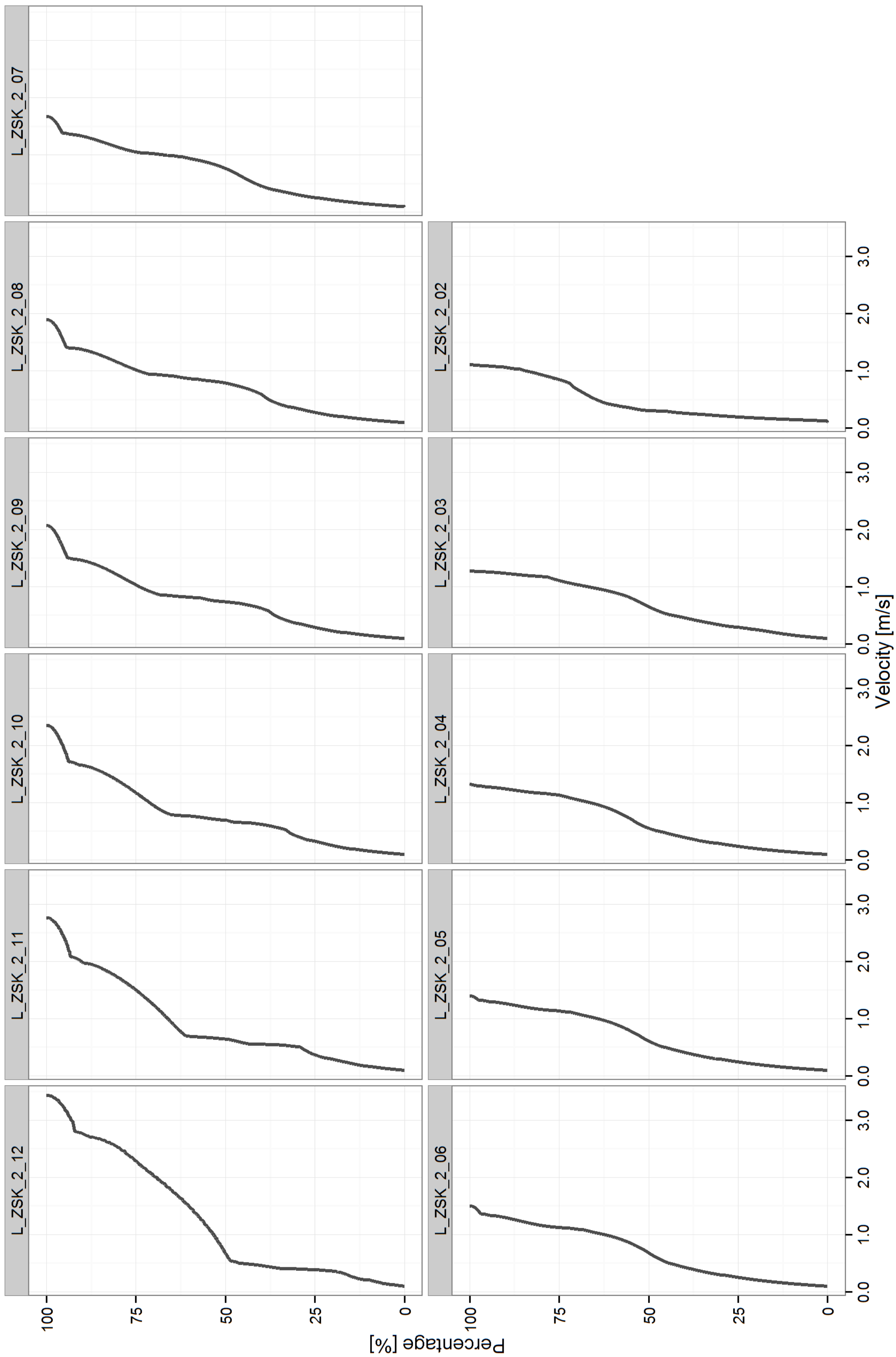
L_ZSK_2_11
L_ZSK_2_12
L_ZSK_2_02
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
Velocity [m/s]
Percentage [%]

Three consecutive waves

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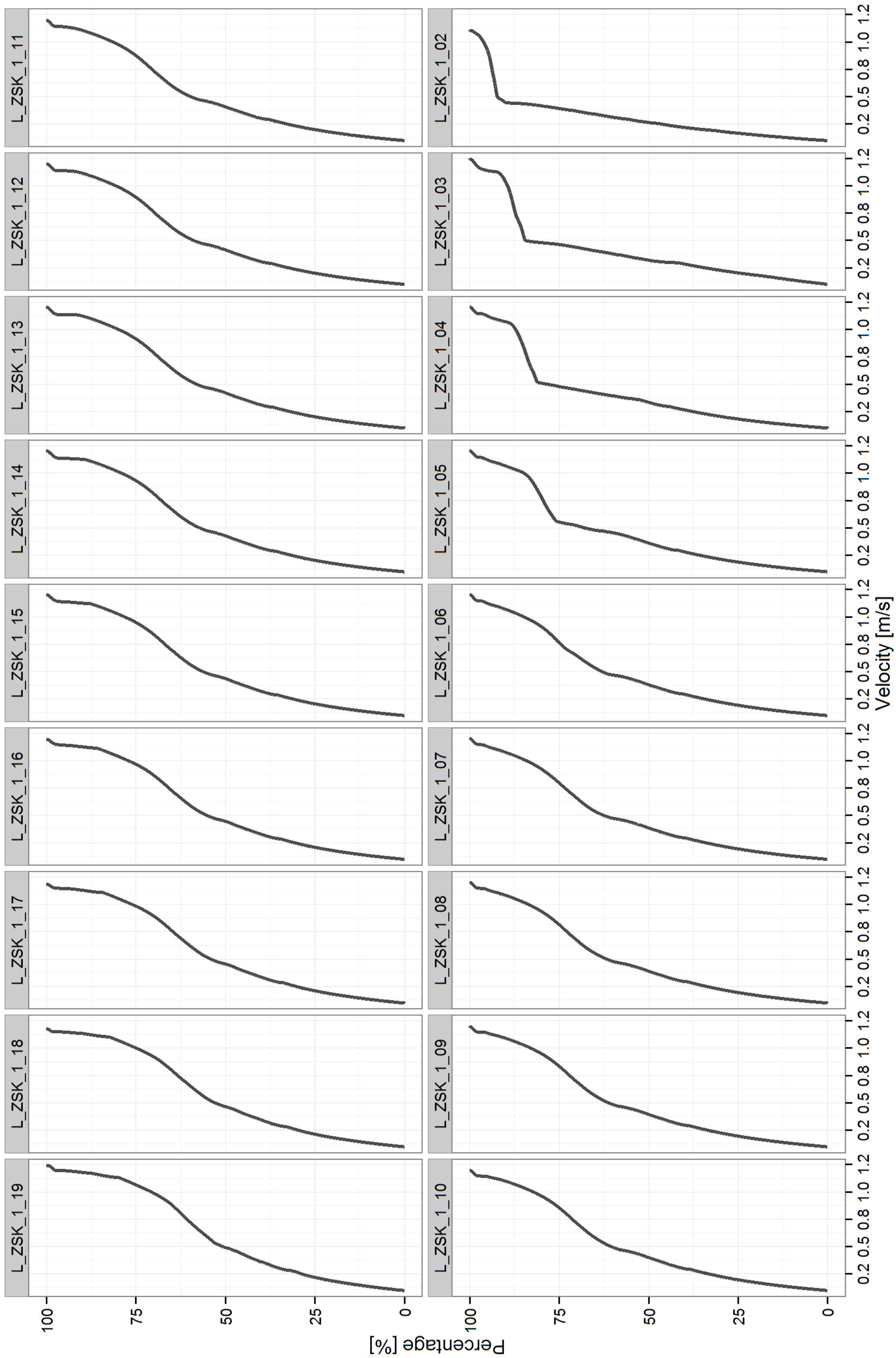
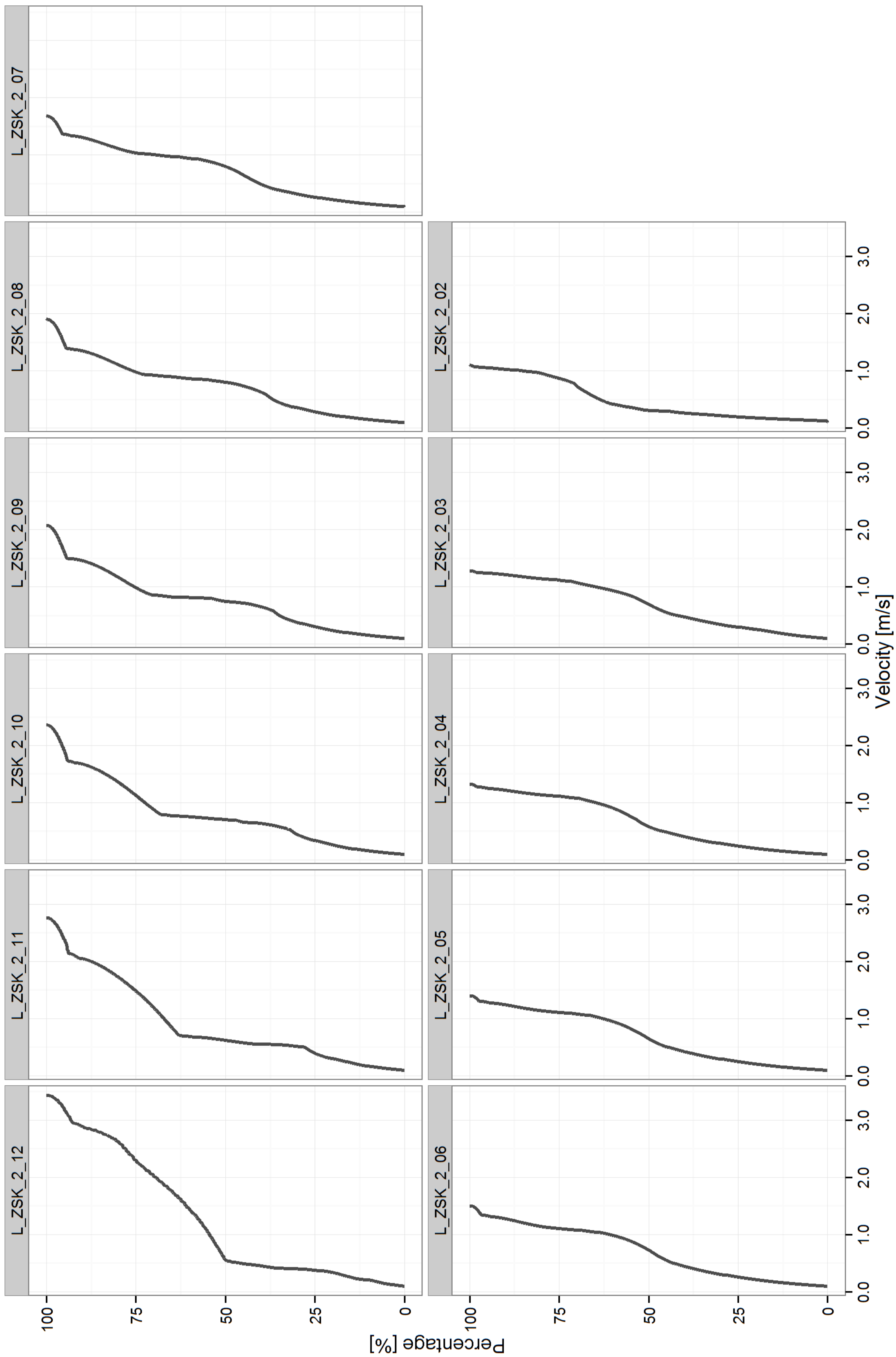
L_ZSK_2_10
L_ZSK_2_11
L_ZSK_2_12
L_ZSK_2_02
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
Percentage [%]
Velocity [m/s]
100
75
50
25
0
0.0
1.0
2.0
3.0

Three waves with a six-minute interval

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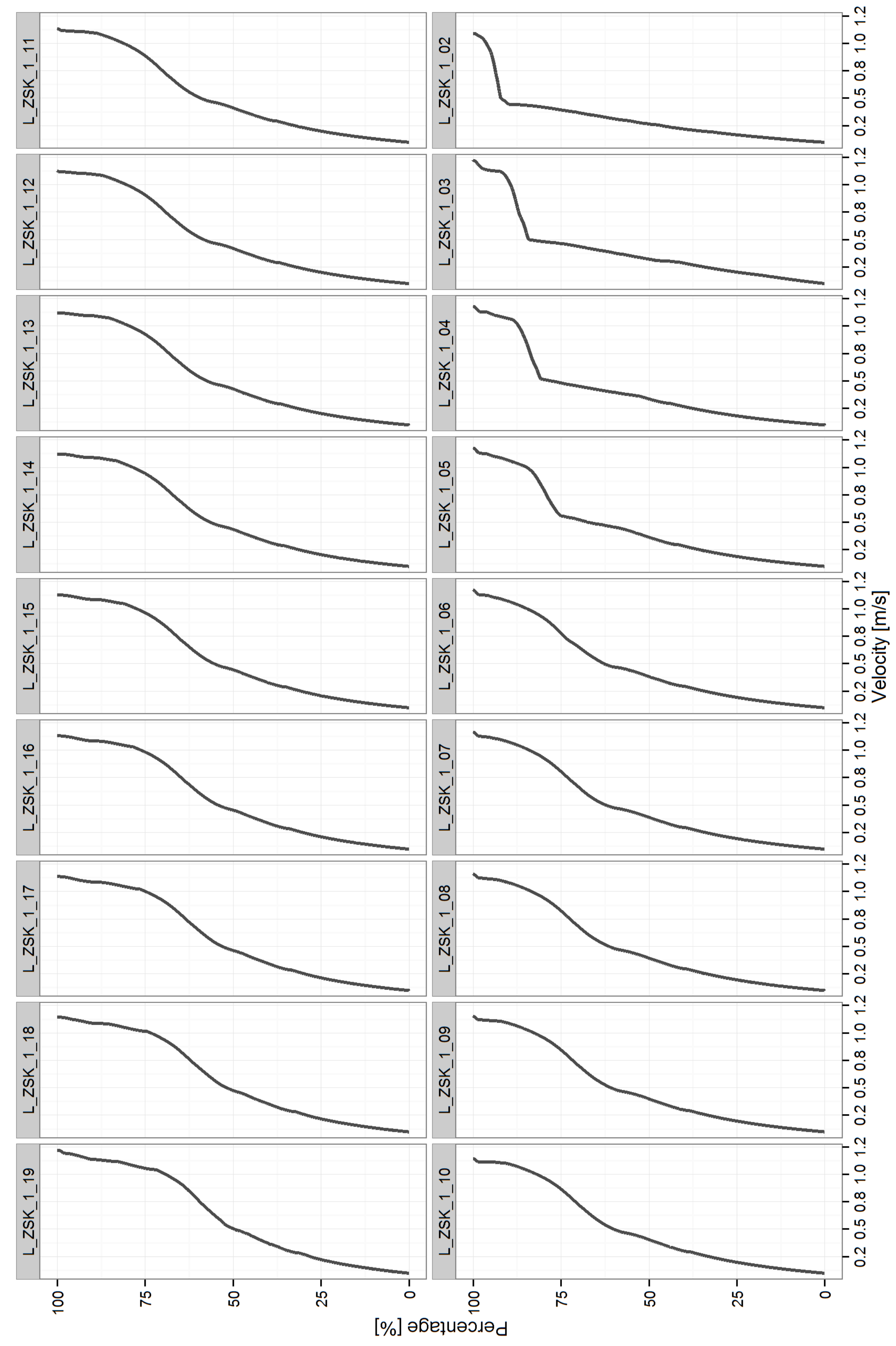
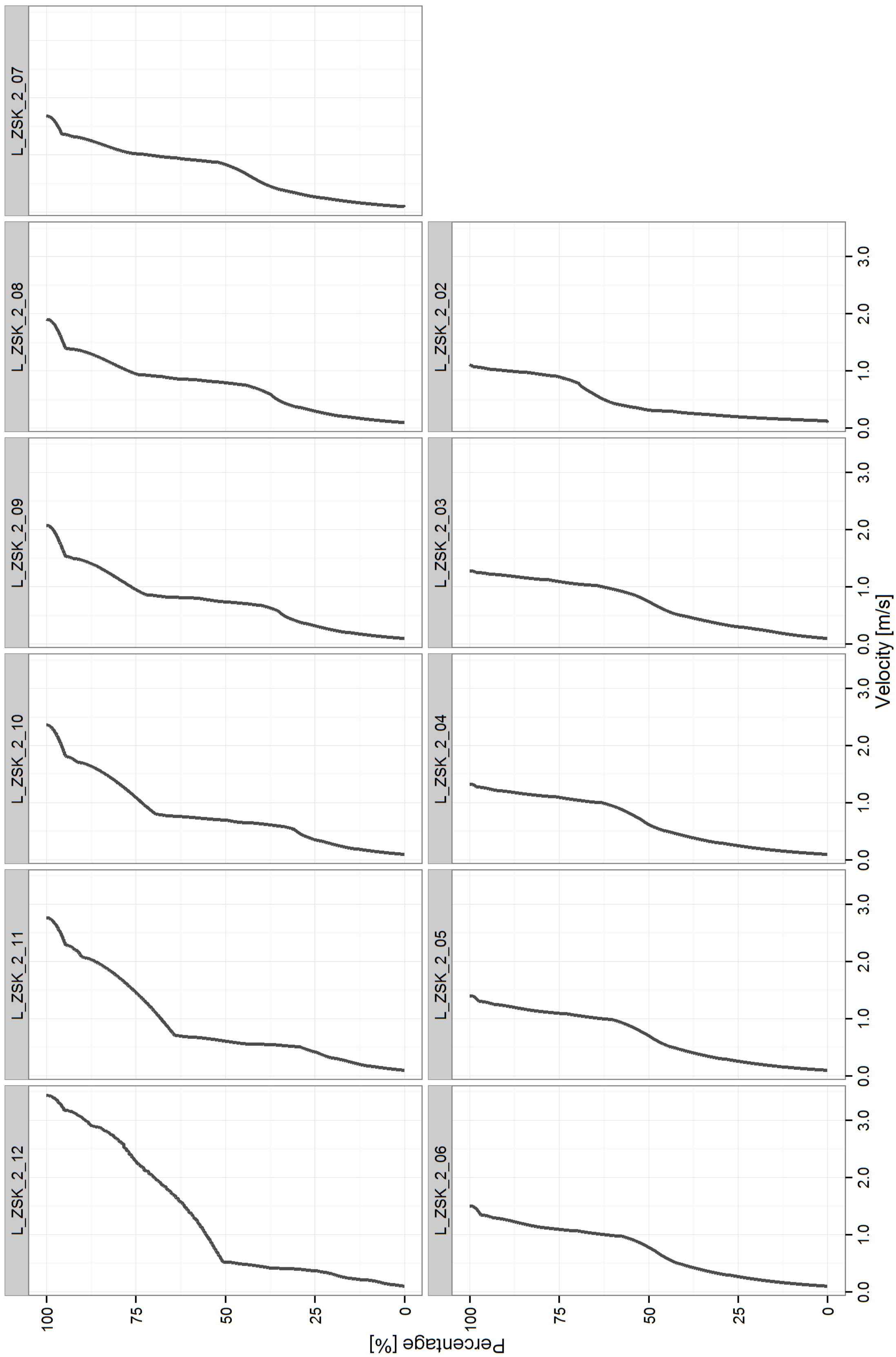
L_ZSK_2_08
L_ZSK_2_09
L_ZSK_2_10
L_ZSK_2_11
L_ZSK_2_12
L_ZSK_2_02
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
Velocity [m/s]
Percentage [%]
0.0
1.0
2.0
3.0
100
75
50
25
0

Three waves with an eight- and a six-minute interval

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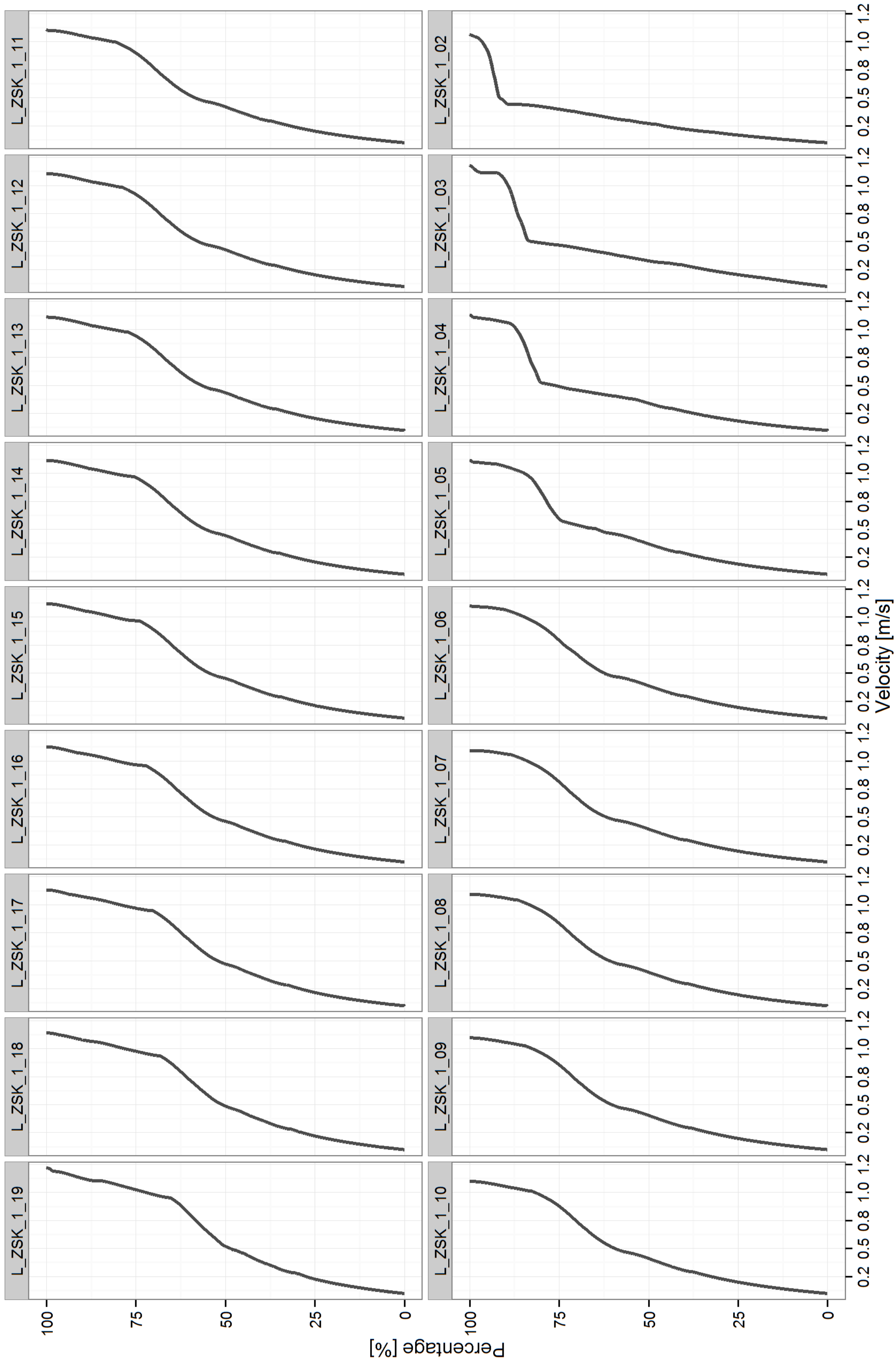
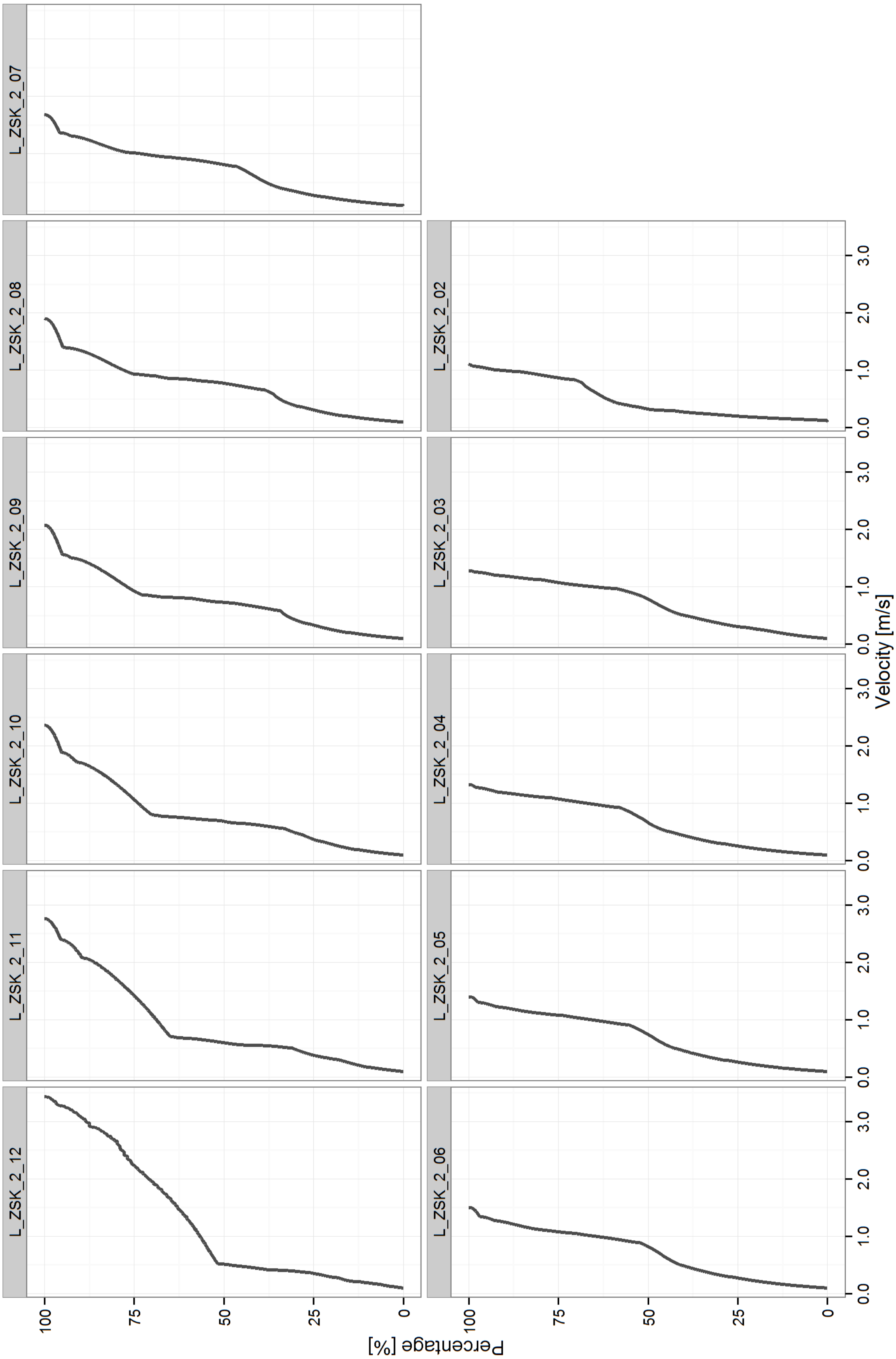
L_ZSK_2_09
L_ZSK_2_10
L_ZSK_2_11
L_ZSK_2_12
L_ZSK_2_02
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
Velocity [m/s]
Percentage [%]
0.0
1.0
2.0
3.0
100
75
50
25
0

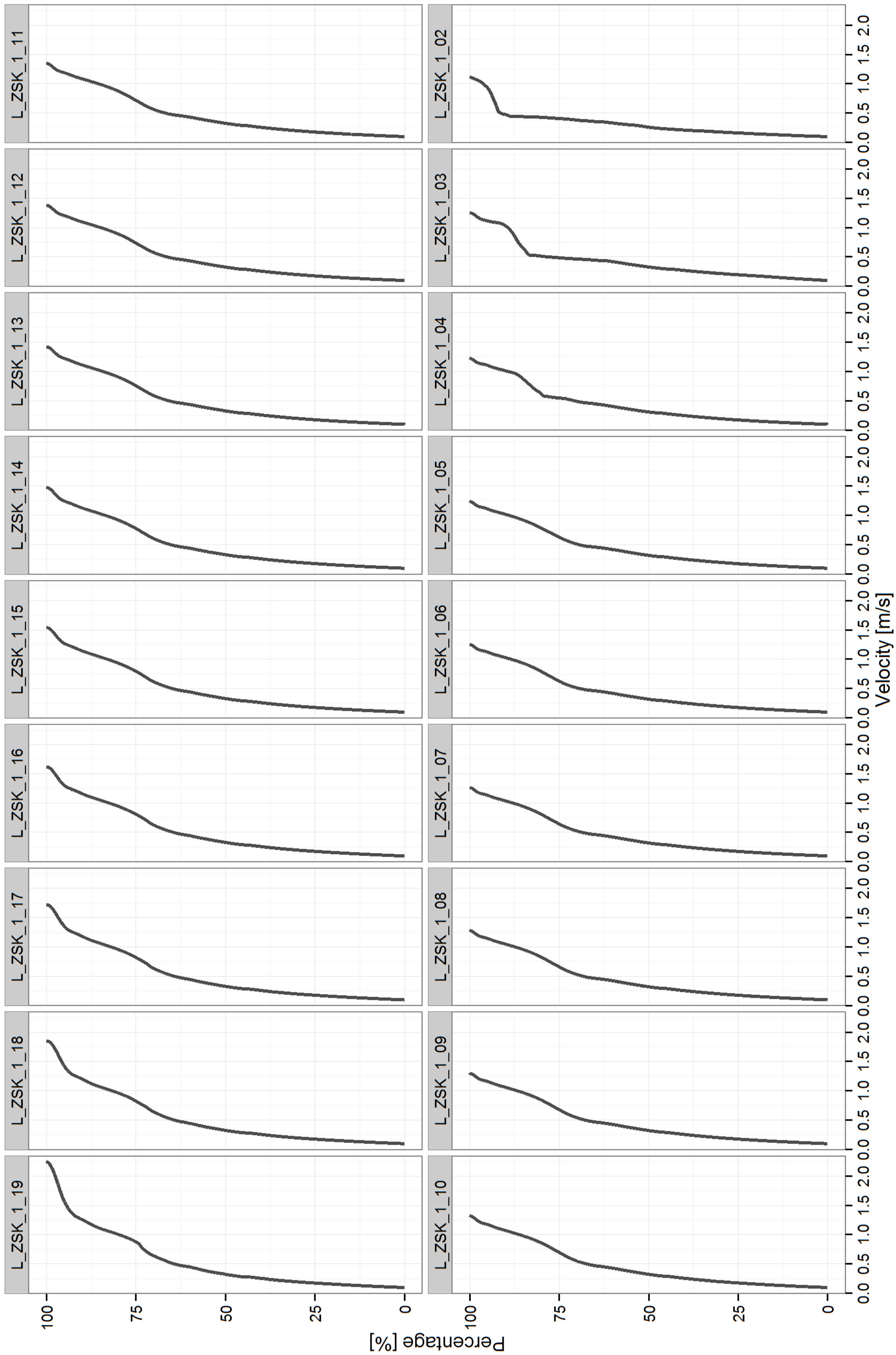
Three waves with a ten- and a six-minute interval

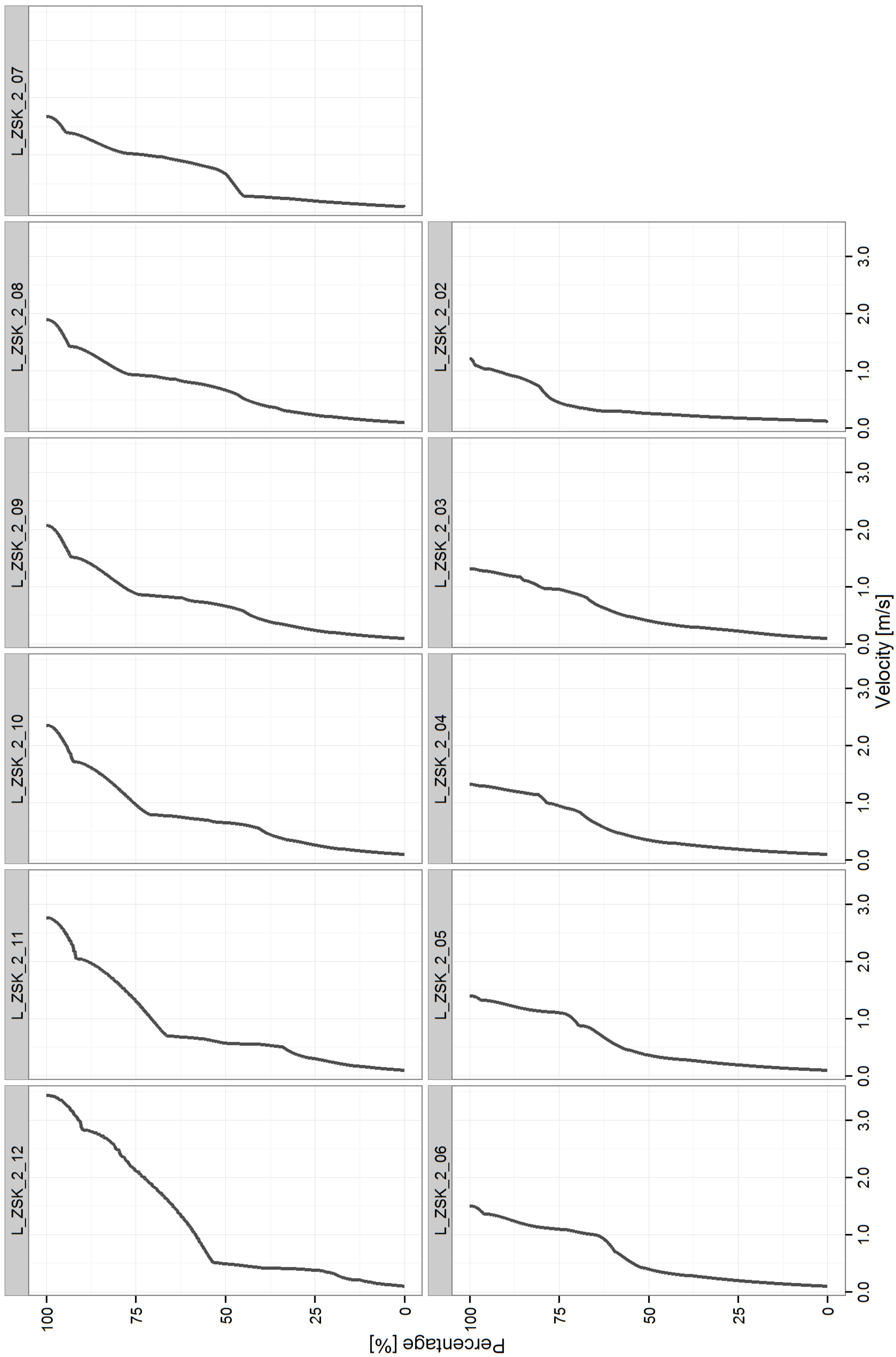




L_ZSK_2_07
L_ZSK_2_08
L_ZSK_2_09
L_ZSK_2_10
L_ZSK_2_11
L_ZSK_2_12
L_ZSK_2_02
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
Percentage [%]
100
75
50
25
0
Velocity [m/s]
0.0
1.0
2.0
3.0

Two waves intercepted at KS2 and then released together

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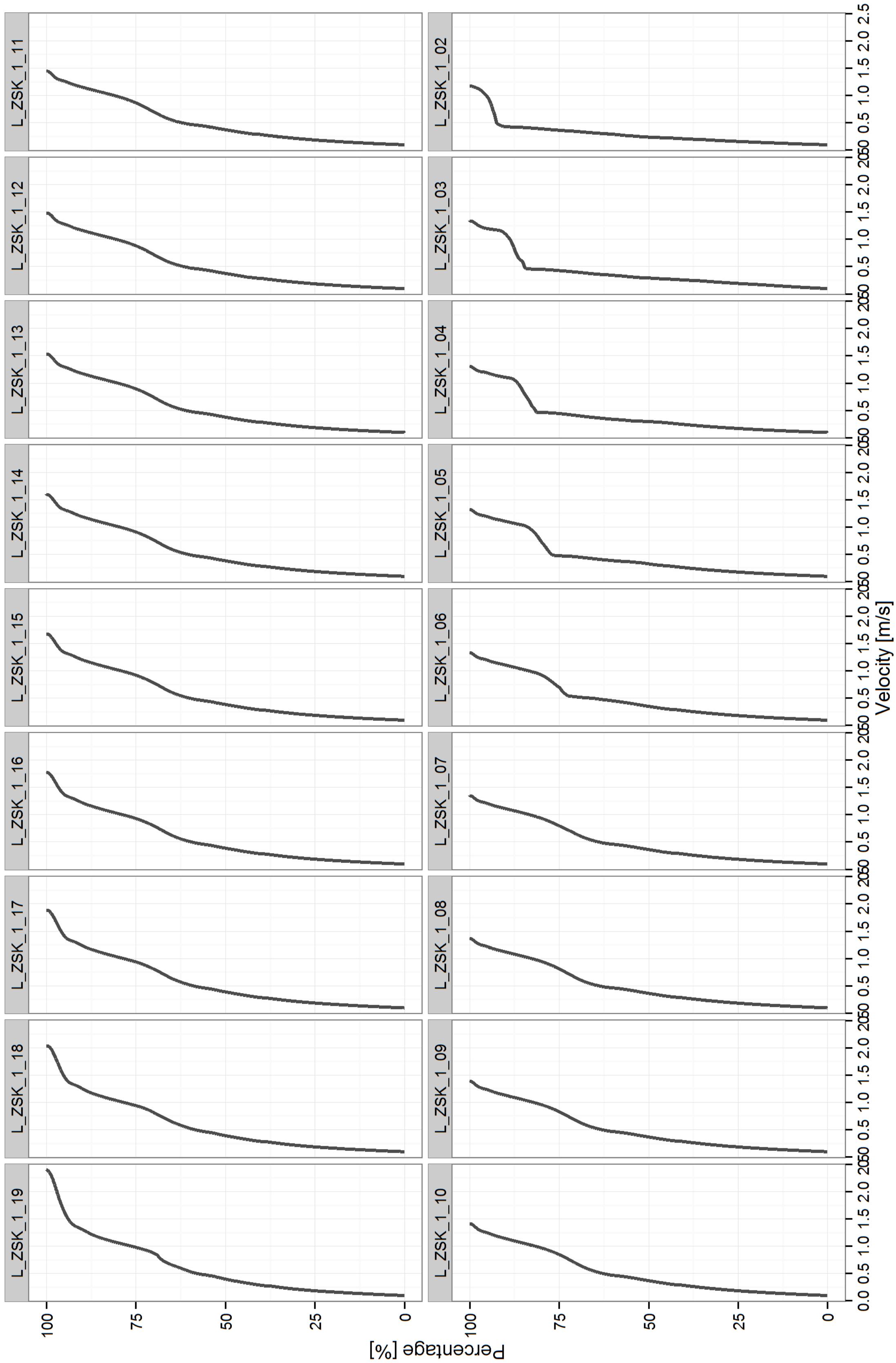
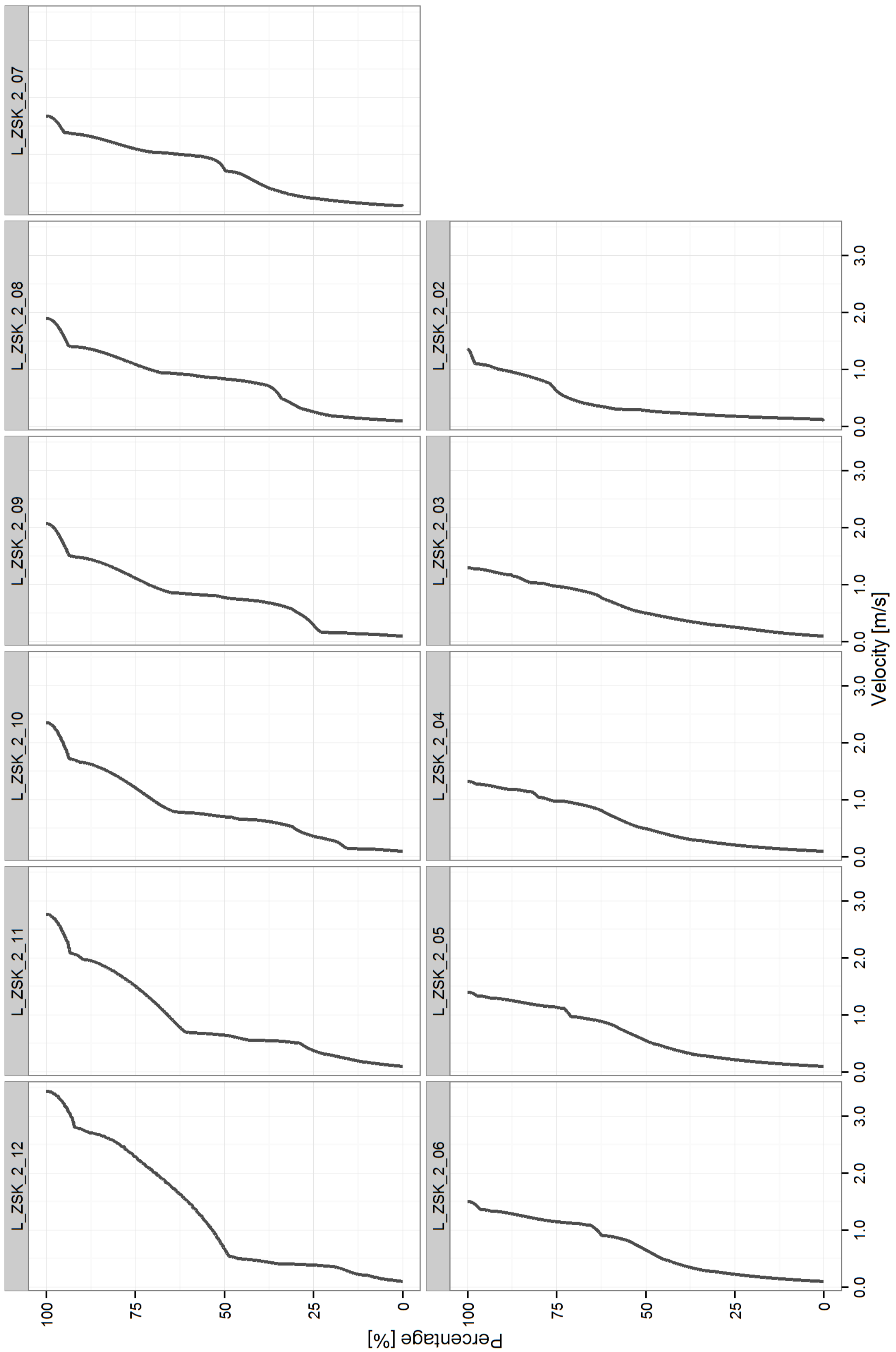
L_ZSK_2_10
L_ZSK_2_11
L_ZSK_2_12
L_ZSK_2_02
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
Velocity [m/s]
0.0
1.0
2.0
3.0
Percentage [%]
100
75
50
25
0

Three waves intercepted at KS2 and then released together

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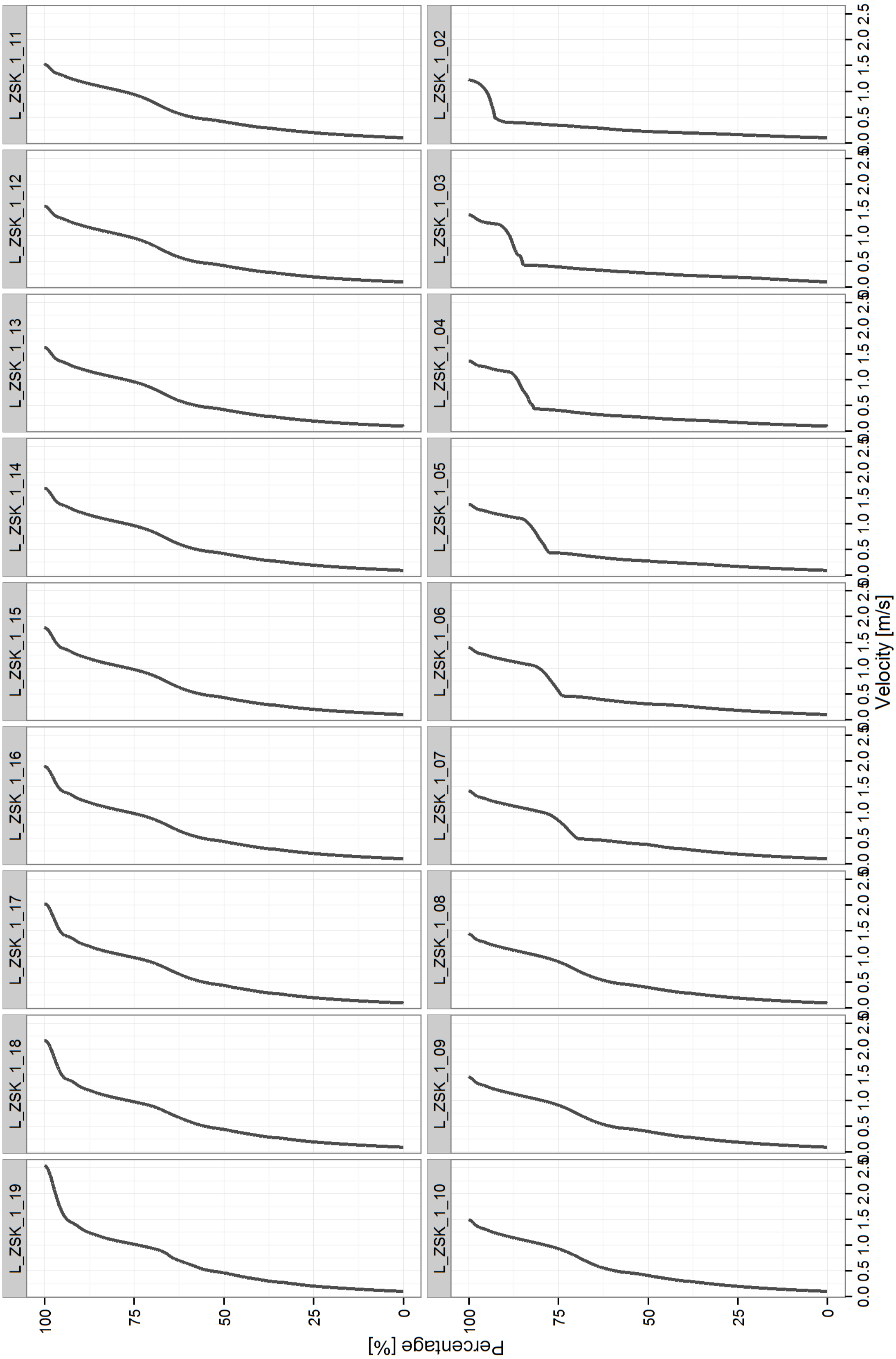
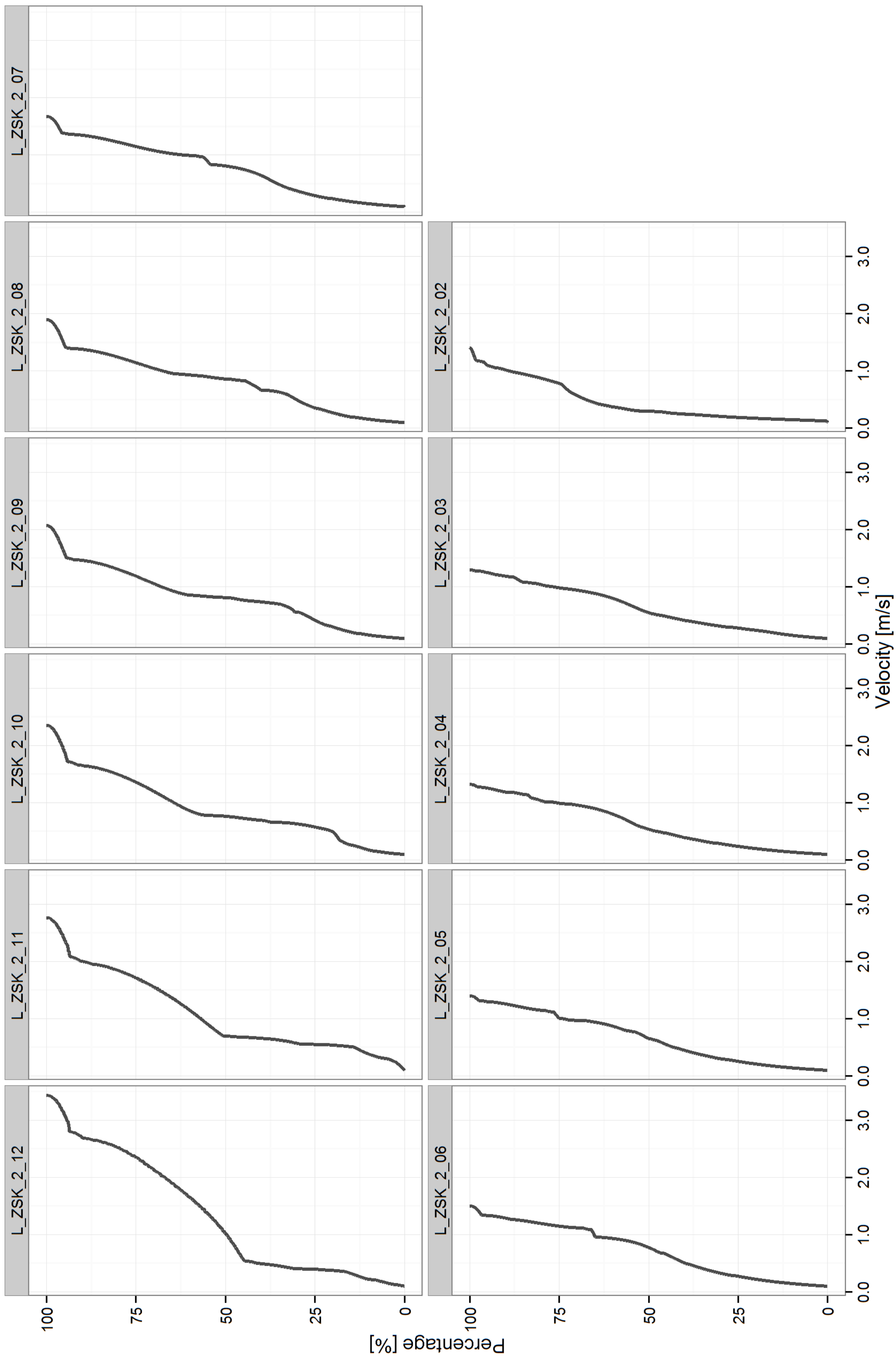
L_ZSK_2_10
L_ZSK_2_11
L_ZSK_2_12
L_ZSK_2_02
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
Velocity [m/s]
Percentage [%]
0.0
1.0
2.0
3.0
100
75
50
25
0

Four waves intercepted at KS2 and then released together

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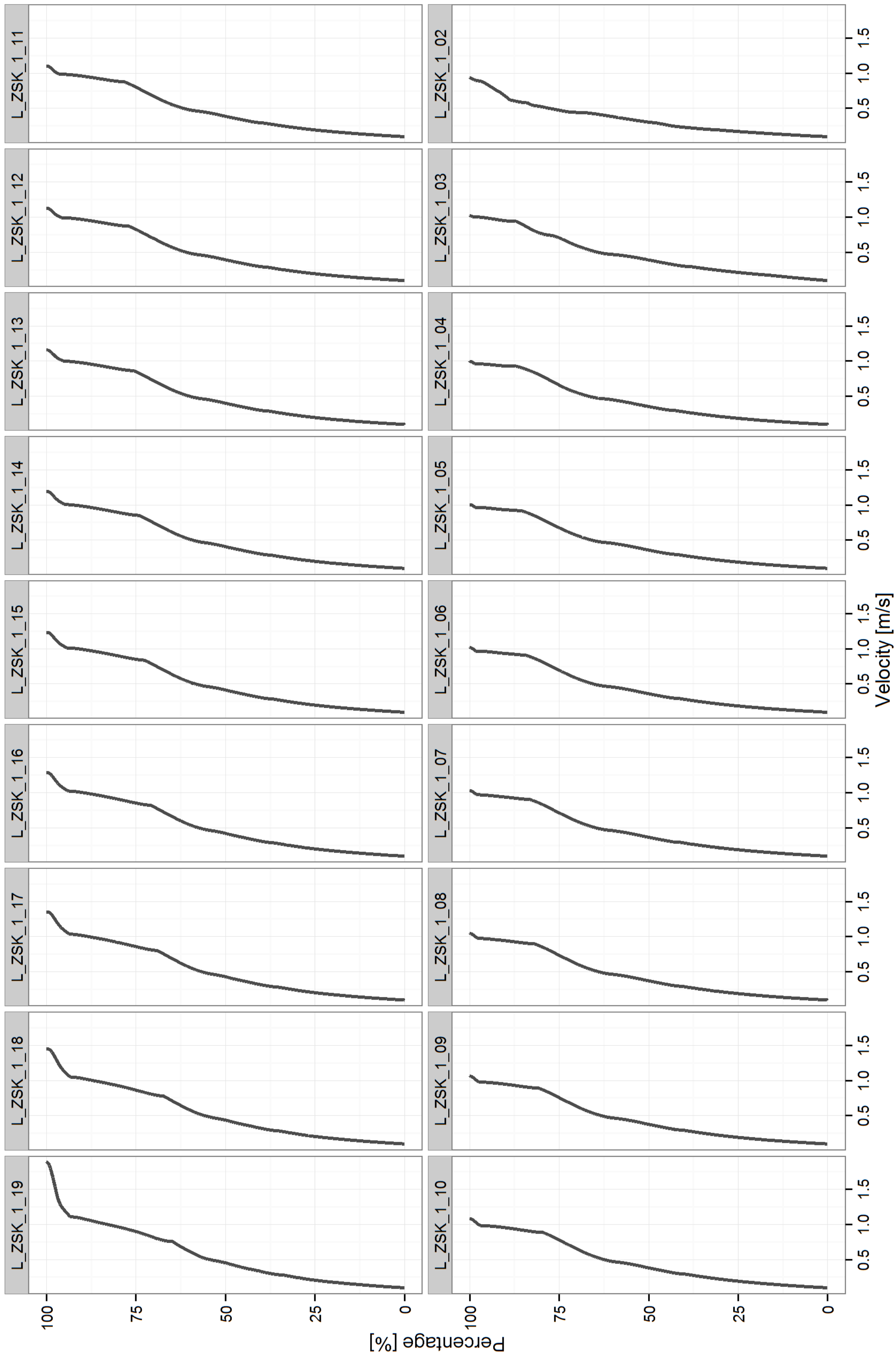
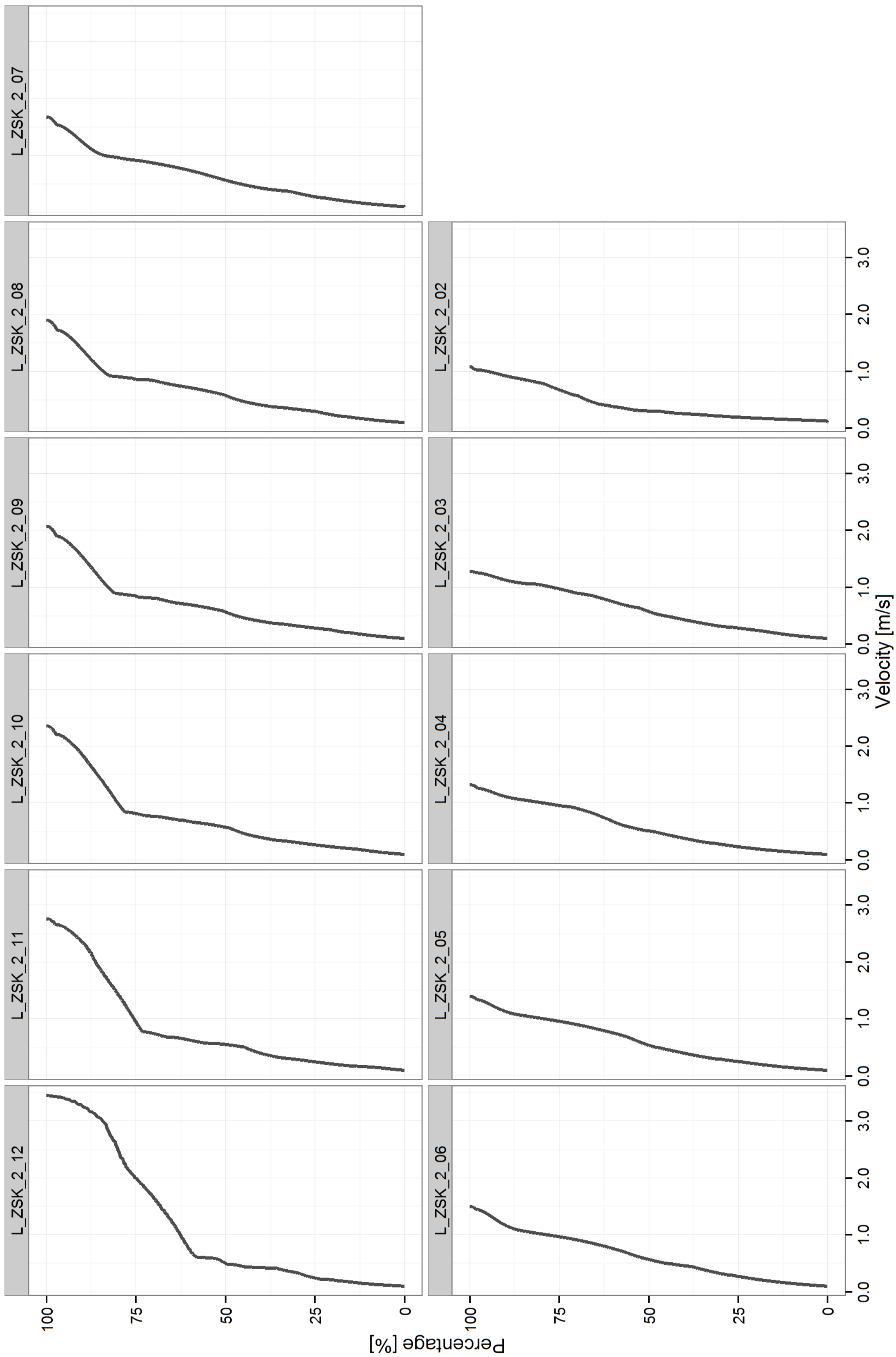
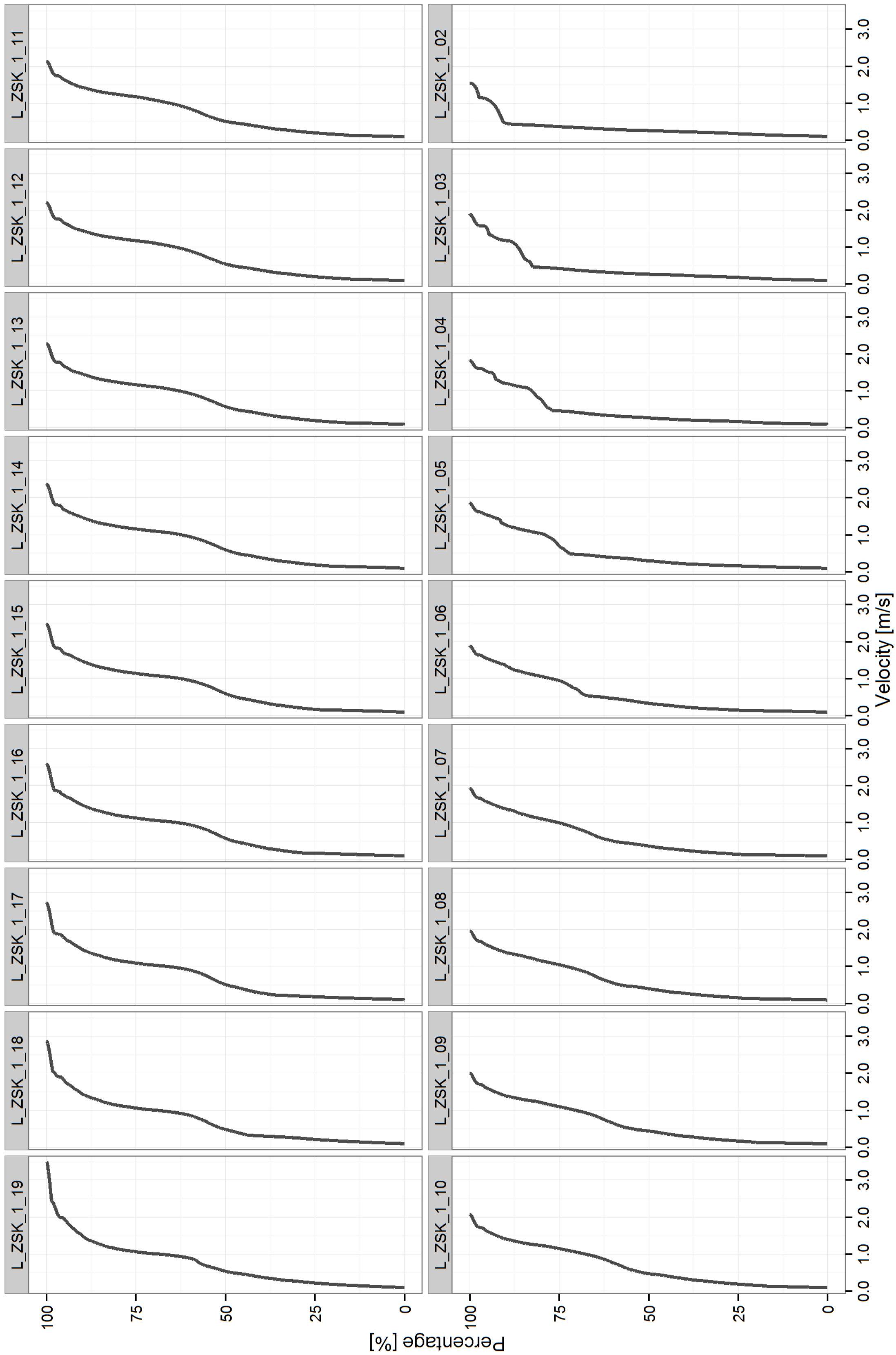
L_ZSK_2_09
L_ZSK_2_10
L_ZSK_2_11
L_ZSK_2_12
L_ZSK_2_02
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
100
75
50
25
0
Percentage [%]
0.0
1.0
2.0
3.0
Velocity [m/s]

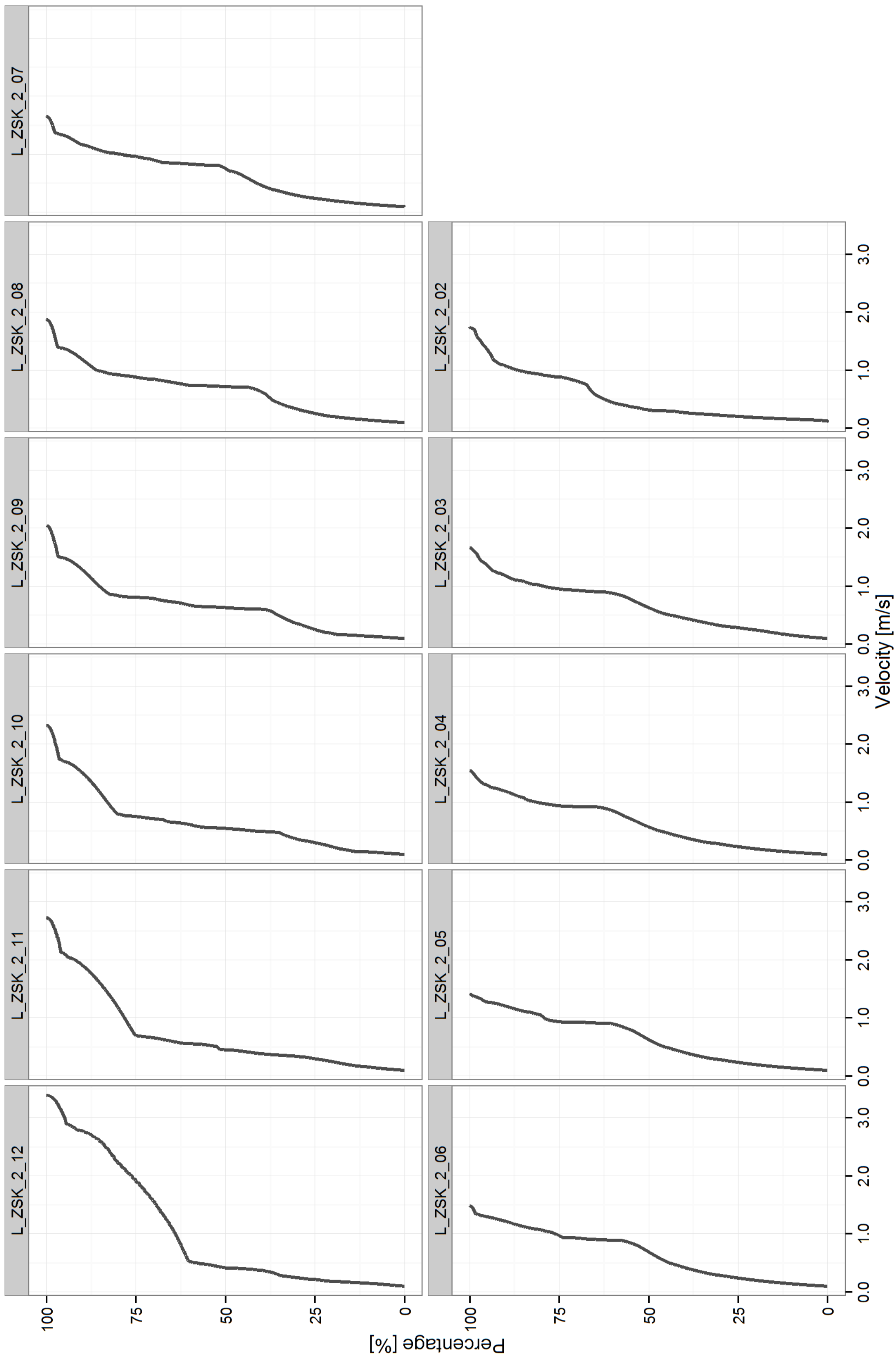
Two waves intercepted at KS2, then released together and caught by a third wave




L_ZSK_2_07
L_ZSK_2_08
L_ZSK_2_09
L_ZSK_2_10
L_ZSK_2_11
L_ZSK_2_12
L_ZSK_2_02
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
Percentage [%]
Velocity [m/s]

Optimal Strategy – Combination of refill-on-empty and 3WSt




L_ZSK_2_07
L_ZSK_2_08
L_ZSK_2_09
L_ZSK_2_10
L_ZSK_2_11
L_ZSK_2_12
L_ZSK_2_02
L_ZSK_2_03
L_ZSK_2_04
L_ZSK_2_05
L_ZSK_2_06
0.0
1.0
2.0
3.0
Velocity [m/s]
100
75
50
25
0
Percentage [%]