**RIGA TECHNICAL UNIVERSITY**

Civil Engineering Faculty

Institute of Heat, Gas and Water technology

**Oskars LAUVA**

Student of doctorate program of Heat, Gas and Water technology

# EQUILIBRIUM TIME OF SCOUR NEAR WATER INTAKES ON RIVER FLOODPLAINS

**Promotional work**

Scientific supervisor

Professor, *Dr. sc. ing.*

B. GJUNSBURGS

**Riga 2016**

## ACKNOWLEDGEMENT

The completion of this promotional work could not have been possible without the participation and assistance of so many people whose names may not all be enumerated. Their contributions are sincerely appreciated and gratefully acknowledged. However, I would like to express my deep appreciation and indebtedness particularly to the following:

Professor Boriss Gjunsburgs for his full support, expert guidance, understanding and encouragement throughout my study and research. Without his incredible patience and timely wisdom and counsel, my thesis work would have been a frustrating and overwhelming pursuit.

To Italian Colleagues Associated Professor Alessio Radice and Professor Francesco Ballio for their endless support, kind and understanding spirit during our research projects at Politecnico di Milano.

To my parents, grandparents, all relatives, friends and others who in one way or another shared their support, either morally, financially and physically.

Thank you very much, everyone.

Oskars Lauva

# ABSTRACT

The study of equilibrium time of scour calculation methods at steady-flow and clear-water conditions revealed that the most common parameters used are: approach flow depth; approach flow velocity; critical flow velocity; structure size parameters (abutment length and width, or pier diameter); median size of the sand; and sand density; however, several parameters are left out or avoided, such as: contraction rate of the flow; local flow velocity at the structure; bed stratification; flood duration; flood sequence; flood probability; and flood frequency. The differential equation of the bed sediment movement in clear-water scour conditions was used and a new method for equilibrium time of scour calculation at water intakes with and without flow separation at the structure in clear-water scour conditions was elaborated. Data from previously conducted and published experiments were used for equilibrium time of scour development calculations and computer modeling. The proposed threshold criteria for equilibrium time of scour known from the literature are only depending on the size of the hydraulic structure, and not on hydraulic parameters of the flow. Ratio of the recalculated critical flow velocity to the local one at the head of the water intake was proposed as the hydraulic threshold criterion in equilibrium time of scour calculation. Different values of the new hydraulic threshold criterion were presumed to find the best fit between computer modeled and calculated equilibrium time of scour values. As a result, the proposed equilibrium time of scour calculation threshold criterion for water intakes that showed the best agreement is equal to $\beta V_{0t}/V_{lt} = 0.985$. Using the new hydraulic threshold criterion values $h_{equil}$, $A_{equil}$, $D_{equil}$, $x_{equil}$, $N_{equil}$ and finally time to equilibrium scour $t_{equil}$ can be calculated. An electronic time to equilibrium scour calculation model was created. Previously conducted flow-altering method against scour at abutments experiment results show that using flow-altering method against scour at abutments, results in equilibrium depth of scour and time of scour reduction. Since the flow-altering method mostly affects the wall, scour depth reduction at the wall was almost three times more effective, than at the abutments nose. Recently carried out live-bed pier scour experiment results revealed that as the ratio of approach flow velocity to critical flow velocity increases, equilibrium time of scour is reached faster with decreasing scour depth value. The duration of water intake laboratory tests of 7 hours was prolonged by using computer program "RoBo", until the scour depth development reached the equilibrium stage. Calculated water intake test data revealed that with an increase in flow contraction rate and with an increase in approach flow Froude number, equilibrium time of scour increases as well. To verify the developed equilibrium time of scour evaluation method, calculated time of scour values were

compared to computer modelled ones; a percent relative error was determined for each calculated and computer modeled time to equilibrium scour test values; using the determined percent relative errors a line of agreement was drawn for each set of water intake experiment data, the results showed good agreement. The developed method for equilibrium time of scour calculation at water intakes with flow separation at the structure was compared with other equilibrium time of scour evaluation methods at abutments available in the literature, and it showed that other calculation methods give great over and under-predicted equilibrium time of scour values, when the approach flow velocity is much lower than the critical flow velocity, ignoring the factor of flow contraction rate. Theoretical analysis of the developed equilibrium time of scour calculation method was made and it showed that equilibrium time of scour depends on: flow contraction rate; kinetic parameter of flow in contraction in open-flow conditions; kinetic parameter of the open flow; ratio of the Froude number to the river slope; dimensionless sand grain size; ratio of the recalculated critical flow velocity to the local flow velocity; relative flow depth; and relative scour depth. Graphical hydraulic and riverbed parameter dependence analysis of the proposed equilibrium time of scour calculation results was made and it showed that equilibrium time of scour depends on: flow contraction rate; relative depth of scour; Froude number; and relative velocity of the flow. This calculation method can be applied to river water intakes with and without flow separation at the structure at steady-flow and clear-water conditions, as well as, other water engineering and hydraulic structures like bridge abutments, piers, guide banks, dams and roads located on river floodplain area. Equilibrium time of scour values are used worldwide in equilibrium scour depth calculation methods, where it is essential to use the most precise predictors of time. A reliable time of scour prediction method can give an advantage to engineers, to know when the equilibrium depth of scour has been achieved, to understand the stability of a water engineering structure.

Promotional work consists of: introduction, 5 chapters, conclusion, 3 appendixes, 91 references, 25 figures, 17 tables, and together 90 pages.

# ANOTĀCIJA

Līdzsvara izskalojuma laika aprēķinu metožu literatūras analīze atklāja, ka visbiežāk lietotie parametri ir: pienākošās plūsmas dziļums; pienākošās plūsmas ātrums; kritiskais plūsmas ātrums; būves izmēru parametri (garums, platums vai diametrs); smilšu daļiņu izmērs; un smilšu blīvums; tomēr vairāki parametri netiek ņemti vērā, tādi kā: plūsmas saspiestība; vietējais plūsmas ātrums pie būves; grunts slāņainība; plūdu ilgums; plūdu secība; plūdu varbūtība; un plūdu biežums. Tika izmantots sanešu kustības diferenciālais vienādojums skaidras plūsmas apstākļos, un izstrādāta jauna līdzsvara izskalojuma laika aprēķinu metode skaidras plūsmas apstākļos pie ūdens ņemšanas būvēm ar un bez plūsmas atdalīšanos pie būves. Dati no iepriekš veiktiem un publicētiem eksperimentiem tika izmantoti līdzsvara izskalojuma laika aprēķinos un dator-modelēšanā. Ierosinātie robežnosacījumi, kas atrodami literatūrā līdzsvara izskalojuma laika aprēķināšanai ir balstīti uz fiziskiem būves parametriem, nevis uz hidrauliskiem plūsmas parametriem. Kritiskā plūsmas ātruma attiecība pret vietējo plūsmas ātrumu pie ūdens ņemšanas būves tika ierosināts kā hidrauliskais robež-nosacījums līdzsvara izskalojuma laika aprēķinos. Tika pieņemtas dažādas jaunā hidrauliskā robež-nosacījuma vērtības, lai atrastu labāko sakarību starp dator-modelētajiem un aprēķinātajiem līdzsvara izskalojuma laikiem. Kā rezultātā ierosinātais robež-nosacījums līdzsvara izskalojuma laika aprēķināšanai pie ūdens ņemšanas būvēm, kurš uzrādīja vislabākos rezultātus ir vienāds ar $\beta V_{0t}/V_{lt} = 0.985$. Izmantojot jauno hidraulisko robežnosacījumu var aprēķināt parametru $h_{equil}$, $A_{equil}$, $D_{equil}$, $x_{equil}$, $N_{equil}$ vērtības un visbeidzot līdzsvara izskalojuma laika $t_{equil}$ vērtību. Tika izstrādāts līdzsvara izskalojuma laika aprēķinu modelis elektroniskajā vidē. Iepriekš veiktu plūsmas vājināšanas metodes izskalojuma samazināšanai pie krasta balstiem eksperimentu rezultāti parāda, ka izmantojot plūsmas vājināšanas metodi samazinās gan līdzsvara izskalojuma dziļums, gan laiks. Izmantojot plūsmas vājināšanas metodi izskalojuma dziļuma samazinājums pie būves sienas ir trīskāršs, salīdzinājumā ar izskalojuma dziļuma samazinājumu pie būves stūra. Nesen veiktu nevienmērīgas plūsmas tilta balsta eksperimentu rezultāti atklāja, ka palielinoties plūsmas ātruma un kritiskā plūsmas ātruma attiecībai, līdzsvara izskalojuma laiks tiek sasniegts ātrāk, samazinoties izskalojuma dziļuma vērtībai. Ūdens ņemšanas būvju eksperimentu ilgums ar datorprogrammas "RoBo" palīdzību no 7 stundām tika pagarināts līdz izskalojuma dziļums sasniedza līdzsvara stāvokli. No ūdens ņemšanas būvju eksperimentu aprēķinu datiem atklājās, ka palielinoties plūsmas saspiestības pakāpei un palielinoties pienākošās plūsmas Fruda skaitlim, palielinās arī līdzsvara izskalojuma laiks. Lai pārbaudītu izstrādāto līdzsvara izskalojuma laika aprēķinu metodi, aprēķinātās līdzsvara izskalojuma laika vērtības tika salīdzinātas ar dator-modelēšanā iegūtajām vērtībām; katrām aprēķinātajām un dator-modelētajām līdzsvara izskalojuma laika vērtībām tika aprēķināta procentuālā relatīvā kļūda; izmantojot aprēķinātās procentuālās relatīvās kļūdas katrai eksperimentu kopai tika izveidota atbilstības līkne, rezultāti parādīja labu sakritību. Izstrādātā aprēķinu metode pie ūdens ņemšanas būvēm ar plūsmas atdalīšanos pie būves tika salīdzināta ar citām līdzsvara izskalojuma laika aprēķinu metodēm pie krasta balstiem, kas pieejamas literatūrā, un rezultāti parādīja, ka citas aprēķinu metodes dod rezultātus, kas stipri atšķiras no reālajām līdzsvara izskalojuma laika vērtībām, kad pienākošās plūsmas ātrums ir stipri mazāks par kritisko plūsmas ātrumu, ignorējot plūsmas saspiestību. Tika veikta izstrādātās līdzsvara izskalojuma laika aprēķinu metodes teorētiskā analīze atkarībā no hidrauliskiem un grunts parametriem, un tā parādīja, ka līdzsvara izskalojuma laiks ir atkarīgs no: plūsmas saspiestības; brīvas plūsmas kinētiskā parametra saspiestajā daļā; nesaspiestas plūsmas kinētiskā parametra; Fruda skaitļa attiecības pret grunts slīpumu; relatīvā plūsmas dziļuma; relatīvā grunts daļiņu izmēra; un kritiskā plūsmas ātruma attiecības pret vietējo plūsmas ātrumu. Tika veikta līdzsvara izskalojuma laika aprēķinu metodes rezultātu grafiskā analīze atkarībā no hidrauliskiem un grunts parametriem, un tā parādīja, ka līdzsvara izskalojuma laiks ir atkarīgs no: plūsmas saspiestības; relatīvā plūsmas dziļuma; Fruda skaitļa; un relatīvā plūsmas ātruma. Šo aprēķinu metodi var pielietot līdzsvara izskalojuma laika aprēķināšanai pie ūdens ņemšanas būvēm ar un bez plūsmas atdalīšanos pie būves, kā arī pie citām ūdens inženierbūvēm kā tiltu/krasta balsti, dambji un ceļa daļas, kas atrodas uz palienas. Līdzsvara izskalojuma laika vērtības lieto visā pasaulē, ievietojot tās dažādās līdzsvara izskalojuma dziļuma aprēķinu metodēs, kur ir būtiska nozīme lietot visprecīzākās laika noteikšanas metodes. Uzticama un droša līdzsvara izskalojuma laika noteikšanas metode sniedz priekšrocības inženieriem jau laicīgi saprast, kad ir sasniegts līdzsvara izskalojuma dziļums, lai novērtētu būves stabilitāti.

Promocijas darbs satur: ievadu, 5 nodaļas, secinājumus, 3 pielikumus, 91 literatūras atsauces, 25 attēlus, 17 tabulas, un kopā 90 lappuses.

# TABLE OF CONTENTS

## LIST OF FIGURES

# LIST OF TABLES

## LIST OF SYMBOLS

| | | |
|---|---|---|
| $A$ | = | a parameter in the Levi (1969) [53] formula; |
| $A_1$ | = | coefficient; |
| $b$ | = | abutment/pier width / section width, m; |
| $B$ | = | width of the scour hole, m; |
| $C_c$ | = | clay content, %; |
| $C_{omp}$ | = | compaction degree related to optimum value; |
| $d$ | = | diameter of sediment particle, m; |
| $D_{50}$ | = | median size of the sand, m; |
| $D_i$ | = | constant parameter in a steady-flow time step; |
| $d_i$ | = | grain size of the bed material, m; |
| $D_p$ | = | pier diameter, m; |
| $d_s$ | = | equilibrium scour depth, m; |
| $Fr$ | = | approach flow Froude number; |
| $Fr_d$ | = | densimetric Froude number; |
| $Fr_f$ | = | Froude number for the flume; |
| $Fr_R$ | = | Froude number for the plain river; |
| $g$ | = | acceleration of gravity, m/s$^2$; |
| $h$ | = | approach flow depth, m; |
| $h_{equil}$ | = | equilibrium scour depth, m; |
| $h_f$ | = | water depth in floodplain, m; |
| $h_m$ | = | mean depth of the scour hole, m; |
| $h_s$ | = | scour depth, m; |
| $k$ | = | coefficient of changes in discharge because of scour, which depends on the flow contraction (Gjunsburgs & Neilands, 2001) [24]; |
| $K_1$ | = | coefficient; |
| $k_m$ | = | coefficient depending on the side-wall slope of the abutment; |
| $k_\alpha$ | = | coefficient depending on the angle of flow crossing; |
| $l$ | = | distance between sediment particles, m; |
| $L$ | = | length of the abutment/ flume width, m; |
| $LI$ | = | liquidity index; |
| $m$ | = | steepness of the scour hole/ dynamic coefficient of continuity; |
| $N$ | = | number of particles passing the cross section on width $b$ in time $t$; |
| $N_0$ | = | parameter to calculate scour formed during the previous time step; |
| $p$ | = | porosity of riverbed material; |
| $P_k$ | = | kinetic parameter of flow in contraction in open-flow conditions; |
| $P_{kb}$ | = | kinetic parameter of the open flow in natural conditions; |
| $q$ | = | relative flow discharge, m$^3$/s; |
| $Q$ | = | flow discharge, m$^3$/s; |
| $Q_b$ | = | discharge in the bridge opening under open-flow conditions, m$^3$/s; |
| $Q_c$ | = | threshold discharge for incipient motion of the bed sediment, m$^3$/s; |
| $Q_f$ | = | discharge across the width of the scour hole with a plain bed, m$^3$/s; |
| $q_s$ | = | sediment discharge in the unit width (in weight units), m$^3$/s; |
| $Q_s$ | = | sediment discharge out of the scour hole, m$^3$/s; |
| $Q_{sc}$ | = | discharge across the scour hole with a scour depth $h_s$, m$^3$/s; |
| $Re_c$ | = | Reynolds number in the channel; |
| $Re_f$ | = | Reynolds number in the floodplain; |
| $T$ | = | relative time/ test duration, s; |

| | | |
|---|---|---|
| $t$ | = | time, s; |
| $t_{comp}$ | = | computer modeled equilibrium time of scour, days; |
| $T_D$ | = | mean period of the bedform, s; |
| $t_e$ | = | time to equilibrium scour depth, s; |
| $t_{equil}$ | = | time to equilibrium scour depth, days; |
| $t_{form}$ | = | calculated equilibrium time of scour by the suggested method, days; |
| $t_i$ | = | time interval, s; |
| $U$ | = | mean velocity of the approach flow, m/s; |
| $U_0$ | = | critical flow velocity, m/s; |
| $U_c$ | = | critical flow velocity for the beginning of sediment motion, m/s; |
| $U_s$ | = | sediment particle velocity, m/s; |
| $V$ | = | voltage read from the device, V / approach flow velocity, m/s / volume of the particles, $m^3$; |
| $V_0$ | = | flow velocity of the beginning of sediment movement, m/s; |
| $V_{0t}$ | = | critical flow velocity for any depth of scour $h_s$, m/s; |
| $V_k$ | = | flow velocity in contraction, m/s; |
| $V_l$ | = | local flow velocity, m/s; |
| $V_{lt}$ | = | local flow velocity at a scour depth $h_s$, m/s; |
| $W$ | = | volume of the scour hole, $m^3$; |
| $x$ | = | relative depth of scour; |
| $\alpha$ | = | coefficient depending on the shape of sediment particle/ constant; |
| $\beta$ | = | reduction coefficient of the critical flow velocity at the bended flow determined by using the Rozovskyi (1956) [82] approach; |
| $\Delta_D$ | = | mean height of the bedform, m; |
| $\Delta h$ | = | maximum backwater value at bridge crossing determined by the Rotenburg (1969) [81] formula, m; |
| $\varepsilon$ | = | relative error; |
| $\gamma$ | = | specific weight of sediments, $t/m^3$ / specific weight of water, $t/m^3$; |
| $\gamma_1$ | = | specific weight of particle, $t/m^3$; |
| $\varphi$ | = | shear stress; |
| $\lambda$ | = | bedform wavelength, m; |
| $\lambda_D$ | = | mean length of the bedform, m; |
| $\mu_c$ | = | relative contents of sediments in flow; |
| $\rho$ | = | water density, $kg/m^3$; |
| $\rho_s$ | = | sand density, $kg/m^3$; |
| $\sigma$ | = | sediment uniformity parameter. |

# INTRODUCTION

The European Environment Agency (EEA) identifies river flooding together with wind related storms as the most important natural hazards in the EU in terms of economic loss. The spatial distribution of high and very high flood hazard risk did not change significantly from 2002 to 2012. However, flood occurrence is projected to increase even further with climate change. The main reason for high flood occurrence is the general increase in winter precipitation, apparent in almost all regions of Europe except in the Mediterranean [90].

From year 1997 up to nowadays, Europe has suffered over 100 major damaging floods. Economic losses as a consequence of extreme flood events have been dramatic. The 1997 floods in Poland and Czech Republic were responsible for losses of about EUR 5.2 billion. In 2000, Italy, France and Switzerland experienced losses of EUR 9.2 billion. In 2002 the material flood damage recorded in Germany, Czech Republic and Austria of EUR 17.4 billion has been higher than in any single year before. And the cost of floods in the UK in summer 2007 has been estimated at around EUR 4.3 billion. The annual average flood damage in Europe in the last few decades is about EUR 4 billion per year (Barredo, 2007) [5].

The highest average values for floods per year measured for each river basin in Europe are found in the low-lying areas along the Rhine and the Danube rivers. The other river basins with high flood risks are the Po river in Northern Italy and all river systems in England [90].

Without efforts to reduce emissions, resulting in changes to rainfall and streamflow mean that across Europe, extreme floods are likely to double in frequency within the next three decades. For example, floods that used to happen about every 100 years will start to occur every 50 years instead. A doubling in frequency of these extreme events corresponds to a tripling in the expected damage by the end of the century in Europe (Alfieri et al., 2015) [2].

The amount of water flowing in large European rivers will increase in 73% of the study area by 2080. Taking into account the size of the rivers and the projected changes, this corresponds to an average increase in water flow of 8% by 2080 compared with 1990 (Alfieri et al., 2015) [2].

Despite the significant investment of researchers in local scour investigation, hydraulic structures in rivers still fail due to scouring processes. It is believed that this is partially a consequence of scouring processes simplification, application of empirical methods, inadequacies between laboratory conditions and the reality in nature, and also the present state of knowledge about some aspects of hydraulic and scouring complexity.

Equilibrium time of scour at clear-water conditions reflects time, when equilibrium scour depth at a hydraulic structure has been achieved.

Equilibrium time of scour evaluation methods designed for time of scour estimation at river hydraulic structures with vertical walls, e.g., abutments can be used for equilibrium time of scour calculations at water intakes on river floodplains, because of the similarity of the shape, location and influence on the flow.

According to literature analysis no method for equilibrium time of scour calculation at water intakes can be found at steady-flow and clear-water conditions, where the following parameters are being taken into consideration: contraction rate of the flow, local flow velocity near the structure, different flood parameters, and bed stratification.

The objective of this research is to develop a new equilibrium time of scour evaluation method for water intakes with and without flow separation at the structure on river floodplains at steady-flow and clear-water scour conditions, by combining into methodology the following scour-control parameters: flow contraction, local flow velocity, bed stratification, flood duration, flood sequence, flood probability, flood frequency, relative depth of scour, densimetric Froude number and median size of the sand.

Influence of flow contraction rate and local flow velocity on equilibrium time of scour at water intakes will be researched. A new equilibrium time of scour calculation method will be developed based on: flow continuity relation before and after the scour; and the rule that clear-water scour reaches the equilibrium stage and ceases when the local flow velocity at the water intake becomes equal to the critical flow velocity. Calculation method of scour depth development at abutments in time during multiple floods of Gjunsburgs et al. (2004, 2008) [27], [30], laboratory data of abutment scour experiments (Gjunsburgs et al., 2004, 2005) [29 – 30], and computer program "RoBo" (Gjunsburgs et al., 2006) [32] will be used for computer modelling of scour development in time to investigate the experimental, computer modeled equilibrium time of scour. Calculated and computer modeled equilibrium time of scour values will be compared, to verify the developed equilibrium time of scour evaluation method.

## Scientific novelty and application

A new equilibrium time of scour evaluation method for water intakes at steady-flow and clear-water conditions was worked out. Suggested method can be applied to river water intakes with and without flow separation at the structure at steady-flow and clear-water conditions, as well as, other water engineering and hydraulic structures like bridge abutments, piers, guide banks, dams and roads located on river floodplain area. Ratio of the recalculated critical flow velocity to the local one at the head of the water intake was proposed as the hydraulic threshold criterion in equilibrium time of scour calculation. The proposed equilibrium time of scour calculation threshold criterion is equal to $\beta V_{0t}/V_{lt} = 0.985$. Equilibrium time of scour values are used worldwide in equilibrium scour depth calculation methods, where it is essential to use the most precise predictors of time. A reliable time of scour prediction method can give an advantage to engineers, to know when the equilibrium depth of scour has been achieved, to understand the stability of a water engineering structure.

Previously conducted flow-altering method against scour at abutments experiment results show that using flow-altering method against scour at abutments, results in equilibrium depth of scour and time of scour reduction. Since the flow-altering method mostly affects the wall, scour depth reduction at the wall was almost three times more effective, than at the upstream edge of the abutment.

Recently carried out live-bed pier scour experiment results revealed that as the ratio of approach flow velocity to critical flow velocity increases, equilibrium time of scour is reached faster with decreasing scour depth value.

The duration of water intake laboratory tests of 7 hours was prolonged by using computer program “RoBo” (Gjunsburgs et al., 2006) [32], until the scour depth development reached the equilibrium stage.

Calculated results of water intake test data (Gjunsburgs et al., 2004, 2005) [29 – 30] revealed that with an increase in flow contraction rate and with an increase in local flow velocity at the upstream edge of the water intake, equilibrium time of scour increases as well.

To verify the developed equilibrium time of scour evaluation method, calculated time of scour values were compared to computer modelled ones, results showed close agreement.

The developed method for equilibrium time of scour calculation at water intakes with flow separation at the structure was compared with other equilibrium time of scour evaluation methods at abutments available in the literature, and it showed that other calculation methods give great over and under-predicted equilibrium time of scour values, when the approach flow velocity is much lower than the critical flow velocity, and ignoring the factor of flow contraction rate.

Theoretical hydraulic and riverbed parameter dependence analysis of the developed equilibrium time of scour evaluation method was made and it showed that equilibrium time of scour depends on: flow contraction rate, kinetic parameter of flow in contraction in open-flow conditions, kinetic parameter of the open flow, ratio of the Froude number to the river slope, relative flow depth, dimensionless sand grain size, ratio of the recalculated critical flow velocity to the local flow velocity.

Graphical hydraulic and riverbed parameter dependence analysis of the proposed equilibrium time of scour calculation results was made and it showed that equilibrium time of scour depends on: flow contraction rate, relative depth of scour, Froude number, and relative velocity of the flow.

This work is a continuation of previous studies in the field of fluvial hydraulics developed by researchers of the Institute of Heat, Gas and Water technology of Riga Technical University: B. Gjunsburgs [23 – 24], R.R. Neilands [71], R. Neilands [67 – 70] and G. Jaudzems [26]. Further studies on riverbed layering impact on scour development in time are in process and studied by E. Govsha [25].

**Research objective and tasks**

The objective of this research is to develop a new equilibrium time of scour evaluation method for water intakes with and without flow separation at the structure on river floodplains at steady-flow and clear-water scour conditions, by combining into methodology the following scour-control parameters: flow contraction, local flow velocity, bed stratification, flood duration, flood sequence, flood probability, flood frequency, relative depth of scour, densimetric Froude number and median size of the sand. To achieve the research objective, the following tasks are defined:

1. Analyze literature for time of scour calculation methods at piers and abutments to find out what kind of threshold criteria and which parameters are being used in equilibrium time of scour calculations;
2. Work out a calculation method for equilibrium time of scour at steady-flow and clear-water conditions for water intakes with and without flow separation at the structure;
3. Propose a new hydraulic threshold condition for equilibrium time of scour calculation, considering hydraulic parameters of the flow;
4. Develop a simple approach sequence how to calculate equilibrium time of scour in practice; create an electronic equilibrium time of scour calculation model in computer environment;
5. Research how flow-altering method affects equilibrium time of scour and depth of scour at abutments;
6. Carry out live-bed pier scour experiments to investigate how live-bed conditions and bedforms affect equilibrium time of scour and depth of scour at a pier;

7. Using computer modeling prolong the duration of water intake experiments until the equilibrium stage; collect computer modelled and calculated equilibrium time of scour data; analyze equilibrium time of scour by approach flow Froude number and contraction rate of the flow;
8. Compare calculated equilibrium time of scour values with computer modeled time of scour values, calculate the percent relative error, and draw a line of agreement to verify the developed equilibrium time of scour evaluation method;
9. Compute time to equilibrium scour values using different author calculation methods found in the literature with data from water intake tests; compare and analyze the results;
10. Theoretically analyze hydraulic and riverbed parameter impact in the proposed equilibrium time of scour calculation method for water intakes;
11. Graphically analyze hydraulic and riverbed parameter influence on equilibrium time of scour;
12. Explore local scour evaluation methods in Latvian legislation.

# 1 BACKGROUND AND LITERATURE REVIEW

## 1.1 Scour at water intakes

The basic function of water intakes is to safely withdraw water from the source (river, lake, etc.) and then to discharge this water into the withdrawal conduit (normally called intake conduit), through which it flows up to water treatment plant. There are two types of river water intakes mostly used in practice: (1) water intakes, located at the riverbank, at the edge of the floodplain, exposed to flow; and (2) submerged water intakes with pumping station located ashore, mostly on river floodplains. River water intakes in common with other hydraulic structures located on river floodplains, as abutments, piers, guide banks, spur dikes, and others, are under constant action of flow during floods.

Water intakes are exposed to flow; therefore, risk of local scoring should be considered as one of the significant causes of possible failure. Obstructed structures considerably disturb the flow regime, inducing flow contraction and local flow conditions. Local scour can expose fundaments of water intakes in floods, which can lead to considerable damage. Failure of water intakes damaged by flood can lead to substantial adverse economic, sanitary, and environmental consequences. Therefore, equilibrium scour should be calculated at water intakes in design stage to ensure their reliability during maintenance period.

Sediment transport processes in rivers have been studied for many decades by researchers like Laursen & Toch (1956) [47], Laursen (1960, 1963, 1980) [44 – 46], Ettema (1980) [16], Melville & Sutherland (1988) [64], Melville (1992, 1995, 1997) [59 – 61], Dongol (1994) [15], Richardson & Davis (1995, 2001) [79 – 80], Kandasamy & Melville (1998) [38], Cardoso & Bettess (1999) [9], Melville & Coleman (2000) [63], Gjunsburgs & Neilands (2001, 2006) [23 – 24], Radice et al. (2002) [75], Coleman et al. (2003) [13], Gjunsburgs et al. (2004, 2005, 2006a, 2006b, 2006c, 2008, 2010, 2014a, 2014b) [25 – 33], Sheppard et al. (2004, 2011, 2014) [86 – 88], Ettema et al. (2006) [17], Fael et al. (2006) [19], Sheppard & Miller (2006) [85], Fael & Cardoso (2008) [18], Cardoso & Fael (2010) [10], Lança et al. (2010, 2013) [40 – 41], Lauva (2012) [48], Radice & Lauva (2013) [74] and many more. Estimation of scour depth at hydraulic structures is a problem that has troubled engineers for many years. In recent years, an extensive research has been aimed at finding methods to efficiently calculate the expected equilibrium scour depth levels at hydraulic structures.

Equilibrium time of scour at water intakes has not been studied yet.

Since the shape, location and influence on the flow of water intakes and hydraulic structures like bridges, bridge piers/abutments are similar, further in the text scour at water intakes will be also referred as pier or abutment scour.

## 1.2 Equilibrium time of scour

For the last two decades many studies, particularly those dealing with clear-water conditions (without general bed sediment motion), report experiments that might not have lasted long enough so as to reach the equilibrium scour holes, whose depth and shape no longer significantly evolve with time. Since then, most studies claim the contrary, i.e., to have reached the equilibrium in long lasting experiments. With few exceptions, such studies refer to steady flows, although in nature, long lasting steady flows seldom exist; unsteadiness is particularly important during floods (Fael & Cardoso, 2008) [18].

Equilibrium time of scour at piers and abutments has been studied by Yanmaz & Altinbaek (1991) [91], Bertoldi & Jones (1998) [6], Cardoso & Bettess (1999) [9], Melville & Chiew (1999) [62], Ballio & Orsi (2001) [3], Gjunsburgs & Neilands (2001) [24], Lauchlan et al. (2001) [43], Neilands & Gjunsburgs (2001) [69], Santos & Cardoso (2001) [83], Oliveto & Hager (2002, 2005) [72 – 73], Coleman et al. (2003) [13], Mia & Nago (2003) [65], Dey & Barbhuiya (2005) [14], Grimaldi et al. (2006) [34], Kothyari et al. (2007) [39], Fael & Cardoso (2008) [18], Setia (2008) [84], Cardoso & Fael (2010) [10], Ghani et al. (2011) [20], Mohammadpour et al. (2011) [66], Abou-Seida et al. (2012) [1], Gjunsburgs et al. (2014) [28], Gjunsburgs & Lauva (2015a, 2015b) [21 – 22], Lauva et al. (2015) [50], and Lauva & Gjunsburgs (2015) [49].

Since the scouring process at hydraulic structures is never ending, threshold criteria are used for equilibrium time of scour evaluation. Threshold criteria, proposed and known from the literature are, when in a 24 hours' period: (i) the depth of scour increases less than 5% of the pier diameter (Melville & Chiew, 1999) [62]; or (ii) less than 5% of the flow depth or abutments length (Coleman et al., 2003) [13]; or (iii) less than 5% of the 1/3 of the pier diameter (Grimaldi et al., 2006) [34]. The proposed threshold criteria for equilibrium time of scour known from the literature are only depending on the size of the hydraulic structure, and not on hydraulic parameters of the flow. Time to equilibrium, $t_e$ is defined as the time corresponding to the end of the principal phase, and the onset of the equilibrium phase.

### 1.2.1 Evaluation methods

Melville & Chiew (1999) [62] prediction of time to equilibrium reads:

$$t_e = 48.26\frac{b}{U}\left(\frac{U}{U_c} - 0.4\right) \quad for \quad \frac{L}{b} > 6, \frac{U}{U_c} > 0.4, \tag{1.1}$$

$$t_e = 30.89\frac{b}{U}\left(\frac{U}{U_c} - 0.4\right)\left(\frac{h}{b}\right)^{0.25} \quad for \quad \frac{L}{b} \le 6, \frac{U}{U_c} > 0.4, \tag{1.2}$$

where $t_e$ – time to equilibrium scour depth, days;
$U$ – approach flow velocity, m/s;
$U_c$ – critical flow velocity, m/s;
$h$ – approach flow depth, m;
$b$ – abutment width, m;
$L$ – abutment length, m.

Eqs. (1.1 – 1.2) are restricted to $0.4 \le U/U_c \le 1$. The lower limit is $U/U_c = 0.4$. The lower limit of 0.4 refers to the condition of scour initiation at bridge piers as suggested by Melville & Chiew (1999) [62]. Melville & Chiew equilibrium time of scour calculations are based on these parameters: abutment length and width, approach flow velocity, critical flow velocity and approach flow depth. The authors do not consider the following parameters in their equilibrium time of scour calculations: densimetric Froude number, median size of the sand, sand density, contraction rate of the flow, local flow velocity at the structure, different flood parameters and bed stratification.

Radice et al. (2002) [75] suggest the following equation for equilibrium time of scour prediction:

$$\frac{t_e \cdot U}{b \cdot h} = 538.11\left(\frac{b}{h}\right)^{0.77}\left(\frac{L}{b}\right)^{0.46}, \tag{1.3}$$

where $t_e$ – time to equilibrium scour depth, h;
$U$ – approach flow velocity, m/s;
$b$ – abutment width, m;
$h$ – approach flow depth, m;
$L$ – abutment length, m.

Radice et al. (2002) [75] equilibrium time of scour prediction Eq. (1.3) is based on approach flow velocity, abutment length and width, and depth of the approach flow, however the following parameters have been left out of consideration: sediment size and density, flow contraction rate, critical flow velocity, local flow velocity, different flood parameters and bed stratification.

Coleman et al. (2003) [13] suggest the following upper bound predictor of time to equilibrium scour:

$$\frac{Ut_e}{L}=10^6\left(\frac{U}{U_c}\right)^3\left(\frac{h}{L}\right)\left[3-1.2\left(\frac{h}{L}\right)\right] \; for \quad \frac{L}{D_{50}}>60, \tag{1.4}$$

where $t_e$ – time to equilibrium scour depth, s;
$U$ – approach flow velocity, m/s;
$U_c$ – critical flow velocity, m/s;
$L$ – length of the abutment, m;
$h$ – approach flow depth, m;
$D_{50}$ – median size of the sand, m.

Fael & Cardoso (2008) [18] state that Coleman et al. (2003) [13] Eq. (1.4) is not a predictor of time to equilibrium scour, but rather an upper bound predictor of $t_e$. As an upper bound predictor of time to equilibrium scour, the contribution of Coleman et. al. (2003) [13] is excellent as soon as $U/U_c$ = 1.0, but it does not seem to properly assess the effect $U/U_c$ for the smaller values of this variable. This shows that there is room for further research regarding the evaluation of time to equilibrium scour.

Coleman et al. (2003) [13] propose three more formulae for time to equilibrium scour:

$$t_e=10^6\left(\frac{U}{U_c}\right)^3\left(\frac{h}{U}\right)\left[3-1.2\left(\frac{h}{U}\right)\right] \; for \quad \frac{h}{L}<1\,. \tag{1.5}$$

Another variation of Coleman et al. (2003) [13] formula for time to equilibrium scour similar to the previous one (Eq. 1.5) is:

$$t_e=10^6\left(\frac{L}{U}\right)^3\left(\frac{h}{L}\right)\left[3-1.2\left(\frac{h}{L}\right)\right] \; for \quad \frac{h}{L}<1\,, \tag{1.6}$$

$$t_e=1.8\cdot10^6\left(\frac{L}{U}\right)\left(\frac{U}{U_c}\right)^3 \; for \quad \frac{h}{L}\geq 1\,, \tag{1.7}$$

where $t_e$ – time to equilibrium scour depth, s;
$h$ – depth of the approach flow, m;
$U$ – mean velocity of the approach flow, m/s;
$U_c$ – critical flow velocity, m/s;
$L$ – abutment length, m.

Coleman et al. (2003) [13] Eqs. (1.6 – 1.7) are based on extensive laboratory and field data, and are meant for clear-water scour and for uniform sediment size. Coleman et al. time to equilibrium scour calculations are based on parameters such as: flow depth, approach flow velocity, critical flow velocity, and abutment length; though ignoring the influence of: flow contraction rate, bed stratification, different flood parameters, and local flow velocity.

Grimaldi et al. (2006) [34] propose the following equation for time to equilibrium scour:

$$t_e = 1.1 \cdot 10^6 \left(\frac{L}{U}\right)\left(\frac{h}{L}\right)^{0.75\frac{U_c}{U}}\left(\frac{U}{U_c}\right)^3, \tag{1.8}$$

where $t_e$ – time to equilibrium scour depth, h;
$L$ – abutment length, m;
$h$ – approach flow depth, m;
$U$ – approach flow velocity, m/s;
$U_c$ – critical flow velocity, m/s.

Grimaldi et al. (2006) [34] Eq. (1.8) parameters for time to equilibrium scour calculation are similar with Coleman et al. (2003) [13] Eq. (1.5 – 1.7) parameters, namely: approach flow depth, approach flow velocity and critical flow velocity, and abutments length. Yet, Grimaldi et al. method does not depend from: sand density and the median size of the sand, flow contraction rate, local flow velocity, different flood parameters, and bed stratification.

The predictor of time to equilibrium scour suggested by Kothyari et al. (2007) [39] is:

$$\log T = 4.8 Fr_d^{0.2}, \tag{1.9}$$

where $T = t/t_R$ – relative time;
$t$ – time, s;
$t_R = z_R/[\sigma^{1/3}(\Delta g D_{50})^{1/2}]$;
$z_R = (hD_p^2)^{1/3}$;
$\sigma = (d_{84}/d_{16})^{1/2}$ – sediment uniformity parameter;
$h$ – flow depth, m;
$D_p$ – pier diameter, m;
$Fr_d = U/(\Delta g D_{50})^{1/2}$ – densimetric Froude number;
$U$ – approach flow velocity, m/s;
$\Delta = \rho_s/\rho - 1$;
$\rho_s$ – sand density, kg/m$^3$;
$\rho$ – water density, kg/m$^3$;
$g$ – acceleration of gravity, m/s$^2$;
$D_{50}$ – median size of the sand, m.

Kothyari et al. (2007) [39] evaluation method (Eq. 1.9) of time to equilibrium scour depends on: approach flow velocity, approach flow depth, pier diameter, densimetric Froude number, median size of the sand and sand density. However, Kothyari et al. evaluation method (Eq. 1.9) disregards: critical flow velocity, local flow velocity, flow contraction rate, different flood parameters, and bed stratification.

As infinitely long experiments are not feasible, Cardoso & Fael (2010) [10] used the results of an important number of long-lasting experiments run under clear-water flow conditions for large abutment lengths (Fael et al., 2006) [19] in order to propose a predictor of time to equilibrium scour:

$$\frac{UD_{50}t_e}{L^2} = 3136\left(\frac{h}{L}\right)^{-1.318}, \tag{1.10}$$

where $U$ – approach flow velocity, m/s;
$D_{50}$ – median size of the bed material, m;
$t_e$ – time to equilibrium scour depth, s;
$L$ – abutment length, m;
$h$ – approach flow depth, m.

Cardoso & Fael (2010) [10] have concluded that time to equilibrium scour at thin vertical wall abutments, protruding at right angles from the side wall of fully developed, uniform flows in wide rectangular channels on flat bed of uniform non-ripple forming sand, depends mostly on relative abutments length; and that time to equilibrium scour may be rather large in the field. Eq. (1.10) is based on these parameters: approach flow velocity, median sand grain size, abutment length and approach flow depth; disregarding the influence of: critical flow velocity, local flow velocity, flow contraction rate, different flood parameters, and bed stratification.

Ghani et al. (2011) [20] using the Genetic Programming (GP) method proposed the following formula for time to equilibrium scour:

$$\frac{Ut_e}{L} = 7.13\cdot10^6\left(\frac{h}{L}\right)^{0.6}\left(\frac{U}{U_c}\right)^{2.97}\left(\frac{L}{D_{50}}\right)^{-0.3}, \tag{1.11}$$

where $t_e$ – time to equilibrium scour, s;
$U$ – approach flow velocity, m/s;
$U_c$ – critical flow velocity, m/s;
$L$ – abutment length, m;
$h$ – approach flow depth, m;
$D_{50}$ – median size of the sand, m.

Ghani et al. (2011) [20] used computer modeling to acquire equilibrium time of scour. The evaluation method that Ghani et al. propose (Eq. 1.11) depends on: approach flow depth, approach flow velocity, critical flow velocity, median size of the sand, and abutments length; however, Ghani et al. evaluation method does not depend on: density of the sand, densimetric Froude number, flow contraction rate, local flow velocity, different flood parameters, and bed stratification.

GP method was developed to predict the values of equilibrium time of scour from laboratory measurements, but this new approach gives the chance to estimate equilibrium time of scouring around long abutments ($h/L < 1$) with the GP modeling techniques. All experiments were run under clear-water flow conditions and different sediment sizes. GP equation was obtained using comparisons of performance, based on error statistics and scatter plots. GP model showed that it has lower absolute error as compared to other equations (Coleman et al., 2003 [13] and Cardoso & Fael, 2010 [10]) and that it is the best fit for time to equilibrium scour value calculation. GP can predict non-dimensional equilibrium time of scour with more accuracy for conditions without any limitations in sediment size or flow velocity.

Abou-Seida et al. (2012) [1] used the least squares approach and proposed the following equilibrium time of scour equation at clear-water scour conditions for soils containing clay:

$$\frac{Ut_e}{h} = 27000 \; Fr^{0.58} \cdot C_c^{0.12} \cdot C_{comp}^{0.82} \cdot LI^{-0.05}, \tag{1.12}$$

where $U$ – approach flow velocity, m/s;
$t_e$ – equilibrium time of scour, s;
$h$ – approach flow depth, m;
$Fr$ – approach flow Froude number;
$C_c$ – clay content, %;
$C_{omp}$ – compaction degree related to optimum value;
$LI$ – liquidity index.

Eq. (1.12) was validated using experiment data, indicating a reasonable prediction of $t_e$ (87 % were in good agreement with only ± 10 % over-prediction/under-prediction). Abou-Seida et al. (2012) [1] equilibrium time of scour predictor is based on approach flow depth and approach flow velocity, Froude number, liquidity index, content of clay in the sand, and compaction degree related to optimum value. This is a rather unique approach, considering clay as a part of the sand contents, although Abou-Seida et al. evaluation method disregards the influence of: critical flow velocity, any parameters of the structure, flow contraction rate, local flow velocity, different flood parameters, and bed stratification.

### 1.2.2 Analysis of equilibrium time of scour evaluation methods

Most of the equilibrium time of scour estimation methods available in the literature are derived from empirical approaches and dimensional analyses. Typically, the equations for equilibrium time of scour calculations were developed using regression analyses from data acquired from local scour experiments, usually conducted in flumes with idealized conditions.

To have a deeper understanding about the parameters being used in equilibrium time of scour estimation formulas, an analysis of parameters is needed. All of the analyzed equations can be found in Chapter 1.2. All equations were analyzed by hydraulic and riverbed parameters as well as structure size parameters used in equilibrium time of scour calculations at piers and abutments (see Table 1.1).

Literature analysis of equilibrium time of scour evaluation methods showed that there is no unified opinion between the authors about equilibrium time of scour calculation (Table 1.1). For instance: approach flow depth, approach flow velocity, as well as abutment length/pier width are the most common factors that have influence on equilibrium time of scour prediction by most of the authors in Table 1.1.

However, taking a closer look: Abou-Seida et al. (2012) [1] do not consider structure size parameters (abutment length or pier width) at all; these authors believe that abutment or pier size parameters have no significant influence on equilibrium time of scour value and therefore they consider these parameters unsubstantial.

Approach flow velocity and approach flow depth are parameters on which all authors agree upon, that have an important influence on equilibrium time of scour (see Table 1.1).

In most of the studies critical flow velocity is considered as an important parameter, since it is the threshold value at which sediment particles start to move, nevertheless only half of the authors mentioned in Table 1.1 use this parameter in their equilibrium time of scour calculations (Melville & Chiew, 1999 [62], Coleman et al., 2003 [13], Grimaldi et al., 2006 [34], Ghani et al., 2011 [20]).

Kothyari et al. (2007) [39] are the only authors, who consider the influence of densimetric Froude number. Likewise, Kothyari et al. are considering median grain size of the bed material and sand density along with his colleagues Coleman et al. (2003) [13], Cardoso & Fael (2010) [10], Ghani et al. (2011) [20] and Abou-Seida et al. (2012) [1], who also agree that riverbed material parameters are significant in equilibrium time of scour calculations.

Most of the equilibrium time of scour evaluation methods compiled in Table 1.1 are restricted to certain preconditions (approach flow velocity, structure size, bed material), which have to be met, otherwise the method cannot be used for objective time of scour prediction.

Some methods are valid for time to equilibrium prediction and give the best results when the approach flow velocity $U$ is close to the critical flow velocity $U_c$, i.e. $0.9 \leq U/U_c \leq 0.99$ (Radice et al., 2002 [75], Coleman et al., 2003 [13]). Other methods work in a much broader range of $U/U_c$, for instance Cardoso & Fael (2010) [10] equilibrium time of scour evaluation method works in a range of $0.57 \leq U/U_c \leq 1.2$, which means this method works also for lower approach flow velocities in compare with the methods of Radice et al. (2002) [75] and Coleman et al. (2003) [13]; in addition it is also valid for equilibrium time of scour prediction in minor live-bed conditions, when the approach flow velocity exceeds the critical flow velocity, which means that the whole riverbed is in motion. Melville & Chiew (1999) [62] time to equilibrium scour prediction method is not valid for live-bed scour conditions ($0.4 \leq U/U_c \leq 1$), however it has an even lower approach flow velocity limit of $U/U_c = 0.4$, which refers to the condition of scour initiation at bridge piers and gives the advantage to predict equilibrium time of scour at low approach flow velocities.

The ratio of approach flow velocity to the critical flow velocity $U/U_c$ is not the only restriction precondition for equilibrium time of scour evaluation methods in Table 1.1. Other restriction preconditions are related to the ratio of abutment length $L$ to sand grain size $D_{50}$; Coleman et al. (2003) [13] time of scour prediction method is limited for $L/D_{50} > 60$, while Cardoso & Fael (2010) [10] equilibrium time of scour prediction method has a higher limit of $L/D_{50} > 100$.

The ratio of approach flow depth $h$ to abutments length $L$ is another precondition that has to be met ($h/L < 1$) in order to use the following equilibrium time of scour prediction methods of Coleman et al. (2003) [13], Cardoso & Fael (2010) [10], and Ghani et al. (2011) [20]. Equilibrium time of scour evaluation methods of Coleman et al. (2003) [13], Cardoso & Fael (2010) [10], and Ghani et al. (2011) [20] are meant for shallow flows and long abutments.

Abou-Seida et al. (2012) [1] time to equilibrium scour predictor is a rather unique approach in the field of study since it is restricted for rivers with soils containing clay and thereby it cannot be used in equilibrium time of scour calculations for rivers with sand, gravel or rock riverbeds.

Table 1.1

Comparison of parameters used in equilibrium time of scour calculations

| Parameters used in equilibrium time of scour calculation equations → | | | Abutment length | Approach flow depth | Approach flow velocity | Critical flow velocity | Densimetric Froude number | Median size of the sand | Pier diameter | Sand density | Contraction rate of the flow | Local flow velocity | Flood duration | Flood sequence | Flood probability | Flood frequency | Bed stratification | Equation restrictions |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| No. | Year | Authors | **1** | **2** | **3** | **4** | **5** | **6** | **7** | **8** | **9** | **10** | **11** | **12** | **13** | **14** | **15** | **16** |
| 1 | 1999 | Melville & Chiew | X | X | X | X | | | | | | | | | | | | $0.4 \leq U/U_c \leq 1$ |
| 2 | 2002 | Radice et al. | X | X | X | | | | | | | | | | | | | $0.9 \leq U/U_c \leq 0.99$ |
| 3 | 2003 | Coleman et al. | X | X | X | X | | X | | X | | | | | | | | $0.9 \leq U/U_c \leq 0.99$<br>$L/D_{50} > 60$, $h/L < 1$ |
| 4 | 2006 | Grimaldi et al. | X | X | X | X | | | | | | | | | | | | |
| 5 | 2007 | Kothyari et al. | | X | X | | X | X | X | X | | | | | | | | |
| 6 | 2010 | Cardoso & Fael | X | X | X | | | X | | X | | | | | | | | $0.57 \leq U/U_c \leq 1.2$<br>$h/L < 1$, $L/D_{50} > 100$ |
| 7 | 2011 | Ghani et al. | X | X | X | X | | X | | X | | | | | | | | $h/L < 1$ |
| 8 | 2012 | Abou-Seida | | X | X | | | X | | X | | | | | | | | Riverbed containing clay |
| 9 | 2015 | Presented method | X | X | X | X | X | X | | X | X | X | X | X | X | X | X | |

In general, literature analysis of equilibrium time of scour evaluation methods at clear-water conditions revealed that no equilibrium time of scour evaluation method can be found in the literature, where the following parameters are being considered: contraction rate of the flow, local flow velocity near the structure, different flood parameters, and bed stratification (see Table 1.1).

## 1.3 Conclusion

The proposed threshold criteria for equilibrium time of scour known from the literature are only depending on the size of the structure, and not on hydraulic parameters of the flow.

Although many studies on local scour around bridge piers and abutments have been found in the literature, investigations dealing with time of scour at piers and abutments under steady flow conditions are still limited.

The study of equilibrium time of scour calculation methods shows that the most common parameters used are approach flow depth, approach flow velocity, critical flow velocity, structure size parameters, as well as median size of the sand and sand density. However, several parameters are left out or avoided, such as densimetric Froude number, contraction rate of the flow, local flow velocity at the structure, bed stratification, flood duration, flood sequence, flood probability, and flood frequency (see Table 1.1).

Most of the equilibrium time of scour evaluation methods are restricted to certain flow, soil and structures conditions, which makes them unusable outside the defined boundary conditions. The most common boundary condition is the ratio of the approach flow velocity to the critical flow velocity, nevertheless the ratio of approach flow depth to abutments length and the ratio of abutments length to median size of the sand are also being used as boundary conditions in equilibrium time of scour evaluation methods (see Table 1.1).

Literature review revealed the following:

1) Approach flow depth, approach flow velocity, critical flow velocity, abutment length/pier diameter, sand grain size and sand density are considered as the basic parameters in equilibrium time of scour calculation (see Table 1.1);
2) According to literature analysis no method for equilibrium time of scour calculation at water intakes can be found, where the following parameters are being taken into consideration: contraction rate of the flow, local flow velocity near the structure, flood duration, flood sequence, flood probability, flood frequency, and bed stratification.

# 2 QUASIANALYTICAL METHOD FOR EQUILIBRIUM TIME OF SCOUR EVALUATION AT WATER INTAKES

The differential equation of equilibrium for the bed sediment movement in clear-water conditions has the form:

$$(1-p)\frac{dW}{dt} = Q_s, \tag{2.1}$$

where $p$ – porosity of riverbed material;
$W$ – volume of the scour hole at a water intake with flow separation at the structure, according to the test results, $(1 - p)\cdot W = 1/6\ \pi m^2 h_s^3$ (at a water intake without flow separation at the structure $(1 - p)\cdot W = 1/5\ \pi m^2 h_s^3$), m$^3$ (Gjunsburgs et al., 2006a, 2006b) [31 – 32];
$m$ – steepness of the scour hole;
$h_s$ – scour depth, m;
$t$ – time, s;
$Q_s$ – sediment discharge out of the scour hole, m$^3$/s.

Volume and shape of the scour hole are independent of the contraction rate of the flow (Gjunsburgs et al., 2006a, 2006b) [31 – 32].

For water intakes with flow separation at the structure the left-hand part of Eq. (2.1) can be written as:

$$(1-p)\frac{dW}{dt} = \frac{1}{2}\pi m^2 h_s^2 \frac{dh_s}{dt} = a h_s^2 \frac{dh_s}{dt}, \tag{2.2}$$

where $h_s$ – scour depth, m;
$m$ – steepness of the scour hole;
$a = 1/2\ \pi m^2$.

For water intakes without flow separation at the structure the left-hand part of Eq. (2.1) can be written as:

$$(1-p)\frac{dW}{dt} = \frac{3}{5}\pi m^2 h_s^2 \frac{dh_s}{dt} = a h_s^2 \frac{dh_s}{dt}, \tag{2.3}$$

where $a = 3/5\ \pi m^2$.

The sediment discharge was determined by the Levi (1969) [53] formula:

$$Q_S = AB\cdot V_l^4, \tag{2.4}$$

where $A$ – a parameter in the Levi (1969) [53] formula;
$B = mh_s$ describes the width of the scour hole, m;

$V_l$ – local flow velocity at the water intake, m/s.

Description of the Levi (1969) [53] sediment discharge concept and explanations of the method can be found in Appendix 2.

The discharge across the width of a scour hole before and after the scour at water intakes with flow separation at the structure is determined as follows (Gjunsburgs & Neilands, 2001) [24]:

$$Q_f = Q_{sc} \cdot k, \tag{2.5}$$

where $Q_f$ – discharge across the width of the scour hole with a plain bed, $m^3/s$;
$Q_{sc}$ – discharge across the scour hole with a scour depth $h_s$, $m^3/s$;
$k$ – coefficient of changes in discharge because of scour, which depends on the flow contraction (Gjunsburgs & Neilands, 2001) [24].

The discharge across the width of a scour hole before and after the scour at water intakes without flow separation at the structure is determined in this way (Gjunsburgs & Neilands, 2001) [24]:

$$Q_f = Q_{sc}. \tag{2.6}$$

From Eq. (2.5) we have:

$$mh_s h_f V_l = k\left(mh_s h_f + \frac{mh_s}{2} h_s\right) \cdot V_{lt}, \tag{2.7}$$

where $m$ – steepness of the scour hole;
$mh_s$ – width of the scour hole, m;
$h_f$ – water depth in the floodplain, m;
$V_l$ – local flow velocity at the water intake, m/s;
$h_s$ – scour depth, m;
$V_{lt}$ – local flow velocity at the water intake at a scour depth $h_s$, m.

From Eq. (2.6) we have:

$$mh_s h_f V_l = \left(mh_s h_f + \frac{mh_s}{2} h_s\right) \cdot V_{lt}. \tag{2.8}$$

Now from Eq. (2.7) we find the local flow velocity $V_{lt}$ at the water intake with flow separation at the structure for any depth of scour (Gjunsburgs & Neilands, 2001) [24]:

$$V_{lt} = \frac{V_l}{k\left(1 + \frac{h_s}{2h_f}\right)}, \tag{2.9}$$

where $k$ – coefficient of changes in discharge because of scour, which depends on the flow contraction (Gjunsburgs & Neilands, 2001) [24];
$V_l$ – local flow velocity at the water intake, m/s.

And from Eq. (2.8) we find the local flow velocity $V_{lt}$ at the water intake without flow separation at the structure for any depth of scour (Gjunsburgs & Neilands, 2001) [24]:

$$V_{lt} = \frac{V_l}{1 + \frac{h_s}{2h_f}}, \tag{2.10}$$

where $V_l$ – local flow velocity at the water intake, m/s.

The critical flow velocity $V_0$ at the water intake at plain bed can be determined by the Studenitcnikov (1964) [89] formula:

$$V_0 = 1.15\sqrt{g} \cdot d_i^{0.25} \cdot h_f^{0.25}, \tag{2.11}$$

where $g$ – acceleration due to gravity, m/s$^2$;
$d_i$ – grain size of the bed material, m;
$h_f$ – water depth in the floodplain, m.

Description of the Studenitcnikov (1964) [89] critical velocity concept and explanations of the method can be found in Appendix 3.

From Eq. (2.11) calculating the square root of $g$ and multiplying it by 1.15 we get 3.6 m$^{0.5}$/s, afterwards this is inserted into Eq. (2.11) and $V_0 = 3.6d^{0.25}h_f^{0.25}$. The critical flow velocity $V_{0t}$ at the water intake for any depth of scour $h_s$ and for the flow bended by the structure is:

$$V_{0t} = \beta \cdot 3.6 \cdot d_i^{0.25} \cdot h_f^{0.25}\left(1 + \frac{h_s}{2h_f}\right)^{0.25}, \tag{2.12}$$

where β – reduction coefficient of the critical flow velocity at the bended flow determined by using the Rozovskyi (1956) [82] approach;
$h_m = h^{0.25}(1 + h_s/2h_f)^{0.25}$ – mean depth of the scour hole, m.

At a plain riverbed the formula for $A = A_1$ is presented as (Eq. 2.4):

$$A = \frac{5.62}{\gamma}\left(1 - \frac{\beta V_0}{V_l}\right)\frac{1}{d_i^{0.25} \cdot h_f^{0.25}}, \tag{2.13}$$

where $\gamma$ – specific weight of sediments, t/m$^3$;
$A$ – a parameter in the Levi (1969) [53] formula.

The parameter $A$ depends on scour, local flow velocity $V_l$, and recalculated critical flow velocity $\beta V_0$, and grain size of the bed material, which changes during floods.

For water intakes with flow separation at the structure $A_i$ at any depth of scour is:

$$A_i = \frac{5.62}{\gamma}\left[1 - \frac{k\beta V_0}{V_l}\left(1+\frac{h_s}{2h_f}\right)^{1.25}\right]\cdot\frac{1}{d_i^{0.25}\cdot h_f^{0.25}\left(1+\frac{h_s}{2h_f}\right)^{0.25}}, \qquad (2.14)$$

where $\dfrac{\beta V_{0t}}{V_{lt}} = \dfrac{\beta V_0}{V_l}\left(1+\dfrac{h_s}{2h_f}\right)^{1.25}$.

For water intakes without flow separation at the structure $A_i$ at any depth of scour is:

$$A_i = \frac{5.62}{\gamma}\left[1 - \frac{\beta V_0}{V_l}\left(1+\frac{h_s}{2h_f}\right)^{1.25}\right]\cdot\frac{1}{d_i^{0.25}\cdot h_f^{0.25}\left(1+\frac{h_s}{2h_f}\right)^{0.25}}. \qquad (2.15)$$

Then we replace $V_l$ in Eq. (2.4) with the local flow velocity at any depth of scour $V_{lt}$ from Eq. (2.9). The parameter $A$ in Eq. (2.4) is replaced with the parameter $A_i$ from Eq. (2.14). The sediment discharge upon the development of scour for water intakes with flow separation at the structure is:

$$Q_s = A_i \cdot mh_s \cdot V_{lt}^4 = b\frac{h_s}{k^4\left(1+\frac{h_s}{2h_f}\right)^4}, \qquad (2.16)$$

where $m$ – steepness of the scour hole;
$h_s$ – scour depth, m;
$b = A_i m V_l^4$.

A similar procedure is done for water intakes without flow separation at the structure, where $V_l$ in Eq. (2.4) is being replaced with the local flow velocity at any depth of scour $V_{lt}$ from Eq. (2.10) and the parameter $A$ in Eq. (2.4) is replaced with the parameter $A_i$ from Eq. (2.15). The sediment discharge upon the development of scour for water intakes without flow separation at the structure is:

$$Q_s = A_i \cdot mh_s \cdot V_{lt}^4 = b\frac{h_s}{\left(1+\frac{h_s}{2h_f}\right)^4}. \tag{2.17}$$

The hydraulic characteristics, such as the contraction rate of the flow, flow velocities $\beta V_0$ and $V_l$, grain size in different bed layers, sediment discharge, and the depth, width and volume of the scour hole, varied during floods.

Taking into account Eq. (2.2) and Eq. (2.16), the differential Eq. (2.1) can be written in the form:

$$ah_s^2\frac{dh_s}{dt} = b\frac{h_s}{k^4\left(1+\frac{h_s}{2h_f}\right)^4}. \tag{2.18}$$

And taking into account Eq. (2.3) and Eq. (2.17), the differential Eq. (2.1) can be written in the form:

$$ah_s^2\frac{dh_s}{dt} = b\frac{h_s}{\left(1+\frac{h_s}{2h_f}\right)^4}. \tag{2.19}$$

After separating the variables and integrating Eq. (2.18), we have:

$$t = D_i\int_{x_1}^{x_2} h_s\left(1+\frac{h_s}{2h_f}\right)^4 dh_s\,, \tag{2.20}$$

$$D_i = \frac{k^4 a}{b} = \frac{1}{2}\frac{\pi \cdot m \cdot k^4}{A_i \cdot V_l^4}\,, \tag{2.21}$$

where $D_i$ – constant parameter in short time interval;

$x_1 = 1 + h_{s1}/2h_f$ and $x_2 = 1 + h_{s2}/2h_f$ are relative depths of scour;

$A_i$ – a parameter in the Levi (1969) [53] formula.

After separating the variables in Eq. (2.19) and integrating it, we have:

$$t = D_i\int_{x_1}^{x_2} h_s\left(1+\frac{h_s}{2h_f}\right)^4 dh_s\,, \tag{2.22}$$

$$D_i = \frac{a}{b} = \frac{3}{5}\frac{\pi \cdot m}{A_i \cdot V_l^4}, \tag{2.23}$$

where $D_i$ – constant parameter in short time interval.

After integration of Eqs. (2.20) and (2.22) with new variables x=1 + $h_s/2h_f$, $h_s=2h_f(x-1)$, and $dh_s=2h_f dx$ we obtain:

$$t = 4D_i h_f^2 (N_i - N_0), \tag{2.24}$$

where $D_i$ – constant parameter in short time interval;
$N_i = 1/6 \cdot x_i^6 - 1/5 \cdot x_i^5$;
$N_0 = 1/6 \cdot x_0^6 - 1/5 \cdot x_0^5 = -0.033$ – parameter to calculate scour formed during the previous time step;
$x_i = 1 + h_s/2h_f$ – relative depth of scour.

Using Eq. (2.14) and Eq. (2.21) for water intakes with flow separation at the structure or Eq. (2.23) for water intakes without flow separation at the structure, and Eq. (2.24), which contain equilibrium depth of scour, it is therefore possible to find the equilibrium time of scour near water intakes:

$$t_{equil} = 4D_{equil.} \cdot h_f^2 (N_{equil.} - N_0), \tag{2.25}$$

where $D_{equil} = 1/2 \cdot (\pi \cdot m \cdot k^4)/(A_{equil} \cdot V_l^4)$ for water intakes with flow separation ($= 3/5 \cdot (\pi \cdot m)/(A_{equil} \cdot V_l^4)$ for water intakes without flow separation);
$h_f$ – water depth in floodplain, m;
$N_{equil} = 1/6 \cdot x_{equil}^6 - 1/5 \cdot x_{equil}^5$;
$x_{equil} = 1 + h_{equil}/2h_f$;
$h_{equil}$ – equilibrium scour depth, m.

The value of $h_{equil}$ for water intakes with flow separation at the structure is determined by Gjunsburgs & Neilands (2001) [24]:

$$h_{equil} = 2h_f \left[ \left( \frac{V_l}{k\beta V_0} \right)^{0.8} - 1 \right] \cdot k_\alpha \cdot k_m, \tag{2.26}$$

where $k$ – coefficient of changes in discharge because of scour, which depends on the flow contraction (Gjunsburgs & Neilands, 2001) [24];
$k_\alpha$ – a coefficient depending on the flow crossing angle (when $\alpha = 90°$, $k_\alpha = 1$);
$k_m$ – a coefficient depending on the side-wall slope of the water intake ($k_m = 1$);
$h_{equil}$ – equilibrium depth of scour, m.

Equilibrium depth of scour at water intakes without flow separation at the structure is found by (Gjunsburgs et al., 2006a, 2006b) [31 – 32]:

$$h_{equil} = 2h_f\left[\left(\frac{V_l}{\beta V_o}\right)^{0.8} - 1\right] \cdot k_\alpha \cdot k_m \,. \tag{2.27}$$

where $k_\alpha$ – a coefficient depending on the flow crossing angle (when $\alpha = 90°$, $k_\alpha = 1$);
$k_m$ – a coefficient depending on the side-wall slope of the water intake ($k_m = 1$);

Using value $h_{equil}$, it is possible to find values $A_{equil}$, $D_{equil}$, $N_{equil}$ and finally $t_{equil}$.

When the local flow velocity $V_{lt}$ becomes equal to the recalculated critical flow velocity $\beta V_{0t}$, then $A_{equil} = 0$, $D_{equil} = \infty$ and $t_{equil} = \infty$. Criteria to evaluate the threshold are needed to appoint to calculate equilibrium time of scour near water intakes.

## 2.1 Threshold condition

Since the scouring process at hydraulic structures is never ending, threshold criteria are used for equilibrium time of scour calculation. The proposed threshold criteria for equilibrium time of scour known from the literature are only depending on the size of the structure, and not on hydraulic parameters of the flow (see Chapter 1.2). A new threshold criterion is needed to be proposed, depending on hydraulic parameters of the flow in order to calculate the equilibrium time of scour values for water intake structures.

Ratio of the recalculated critical flow velocity to the local flow velocity at the head of the water intake is proposed as the hydraulic threshold criterion in equilibrium time of scour calculations for water intakes.

According to the computer-modeling results the scour stops when the local flow velocity $V_{lt}$ becomes equal to the recalculated critical flow velocity $\beta V_{0t}$ or the ratio of those velocities becomes equal to 1, and the equilibrium is equal to infinity. Different values of the ratio of recalculated critical flow velocity to local flow velocity at the head of the water intake were presumed for equilibrium time of scour calculation in order to find the best fit between computer modeled and calculated equilibrium time of scour values. As a result, the proposed equilibrium time of scour calculation threshold criterion for water intakes that showed the best agreement is equal to:

$$\frac{\beta V_{0t}}{V_{lt}} = \frac{\beta V_0}{V_l}\left(1 + \frac{h_{equil.}}{2h_f}\right)^{1.25} = 0.985 \,. \tag{2.28}$$

## 2.2 Equilibrium time of scour calculation in practice

After acquiring the threshold criterion from Eq. (2.28), it is possible to calculate $t_{equil}$ and the sequence of that is the following:

1) Calculate equilibrium depth of scour $h_{equil}$ (Eq. 2.26 for water intakes with flow separation at the structure, or Eq. 2.27 for water intakes without flow separation at the structure);
2) Calculate parameter from the Levi (1969) [53] formula $A_{equil}$ using the calculated value of $h_{equil}$ from previous step (Eqs. 2.14 for water intakes with flow separation at the structure, or Eq. 2.15 for water intakes without flow separation at the structure);
3) Calculate parameter $D_{equil}$ using the calculated $A_{equil}$ value from previous step (Eqs. 2.21 for water intakes with flow separation at the structure, or Eq. 2.23 for water intakes without flow separation at the structure);
4) Calculate the relative depth of scour $x_{equil}$ using the calculated value of $h_{equil}$ from first step ($x_{equil} = 1 + h_{equil}/2h_f$);
5) Calculate parameter $N_{equil}$ using the calculated value of $x_{equil}$ from previous step ($N_{equil} = 1/6 \cdot x_{equil}^6 - 1/5 \cdot x_{equil}^5$)
6) And finally calculate equilibrium time of scour $t_{equil}$ using values $h_{equil}$, $D_{equil}$, and $N_{equil}$ from steps 1, 3 and 5.

Equilibrium time of scour calculation model with all the additional parameters needed for the calculations can be found in Appendix 1.

# 3 EXPERIMENT SET-UP

## 3.1 Experiment set-up for flow-altering method against scour at abutments

Figure 3.1 Layout of the experiment channel
*Note:* Flow direction is from right to left.

The flow-altering method against scour experiments were run at the Hydraulic Engineering Laboratory of the Politecnico di Milano, Italy. A rectangular channel 5.8 m long with a cross section 0.40 m wide and 0.16 m deep was used in the experiments. The channel consists of several rectangular transparent plexiglass sections, which are connected with bolts (Figure 3.1).

The sediment tank was filled with nearly uniform sediments with mean diameter $D_{50}$ = 3.6 mm and sediment density $\rho_s$ = 1.43 kg/dm$^3$. The critical discharge $Q_c$ for the sediments was 19 l/s.

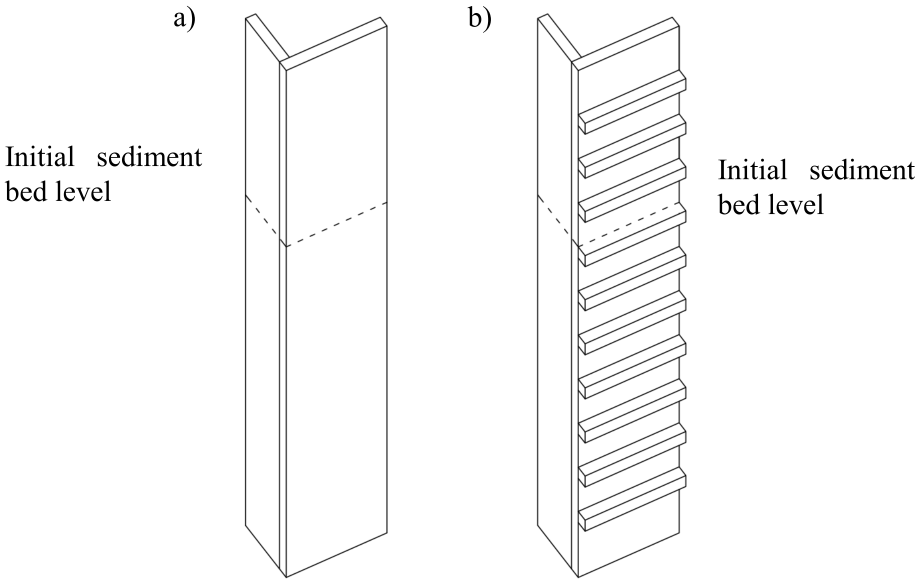



Figure 3.2. Abutment models used in flow-altering experiments
*Note*: (a) unprotected abutment; (b) threaded abutment.

The vertical wall abutments were made of transparent PVC; the dashed line represents the trace of the non-scoured sediment level (see Figure 3.2). Their dimensions were 10 cm long and 1 cm thick. The vertical wall with threads had a total of 10 threads attached to the wall (see Figure 3.2b). The dimensions of the threads were 10 cm long, 1 cm thick, and 1 cm high. The first thread was situated 3 cm from the top of the wall and each next thread was located 3 cm from the previous one.  Each abutment was constructed in a 90-degree angle, so it could withstand the impact from the current and not move from its place. The abutment was dipped into the sediment layer so that the top of the abutment was at the same level as the flow and the lid. The abutment was positioned roughly in the middle of the channel in longitudinal distance and by the left wall of the experiment tank (X = 100; Y = 0).

The water discharge was progressively increased from approximately 6.9 – 7.0 l/s to the desired value for the steady test condition (18.5 l/s). Therefore, the earliest instants for the experiments were not stationary but in a transitory stage, with the duration of the transient being shorter than 30 s. The starting time of the scour process ($T = 0$ s) was chosen as the moment when the discharge was increased from 6.9 – 7.0 l/s to the desired value of 18.5 l/s.

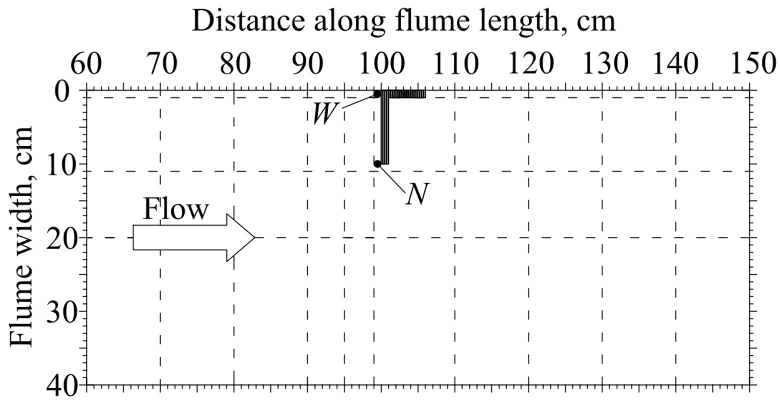



Figure 3.3. Map of sections and measurement points

Scour hole depth was measured at two points near the abutment. The two measurement points are given in the map of sections (Fig. 3.3). First measurement point is located at the nose of the abutment (*N*); the second measurement point is at the wall of the abutment (*W*). The scour depth at the nose was measured with the laser sensor, but the scour depth at the wall was measured manually using a meter attached to the channels side wall. The meter was attached so that the “0” mark was at the initial level of the sediment bed. After specific times like 1 hour, 3 hours etc. surveys were made when cross sections of the sediment bed were measured (the dashed lines in Fig. 3.3) and later put into plots for analyze. From these data also a three-dimensional image could be acquired. A complete set of information about the experiments and results can be found in Master thesis of Lauva (2012) [48].

## 3.2 Experiment set-up for live-bed pier scour

The live-bed (LB) pier scour experiments were run at the Hydraulic Engineering Laboratory of the Politecnico di Milano, Italy in a rectangular channel 5.8 m long with a cross section 0.40 m wide and 0.16 m deep. The channel configuration was very similar to the one used in flow-altering method against scour experiments (Figure 3.1). Several modifications were made to the channel for its ability to run live-bed experiments. A sediment feeder was constructed at the beginning of the channel and a sediment catcher was installed in the outlet tank. Also the sediment tank was modified. The lid was equipped with a piezometer, a false floor with a fine screen and a platform for the cylinder were installed in the tank, and then the tank was filled with bags of sand an additional a filling material (see Figure 3.4).

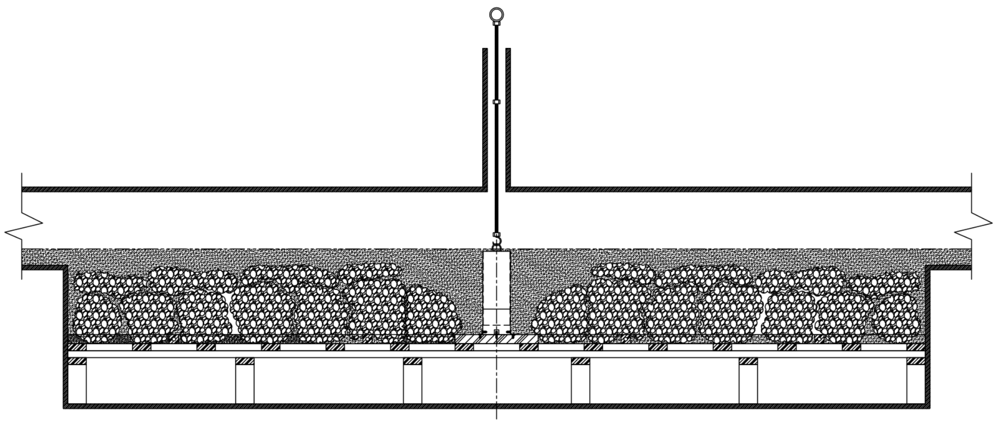


Figure 3.4. Sediment tank sketch with the cylinder sunk under the sediment bed

Afterwards the sediment tank and the channel was filled with PBT grains with a 0.04 m thick bed layer, having mean diameter $D_{50}$ = 3 mm and density $\rho_s$ = 1.2 kg/dm$^3$. During all of the experiments the channel was covered with a plexiglass lid. A telescoping PVC cylinder was placed approximately 1 m from the outlet of the channel, to allow the bed forms to be fully developed throughout the channel, before they reach the section, where the telescopic cylinder is placed. The part of the channel where the cylinder was placed was 0.40 cm deep. The diameter of the cylinder was 6 cm and the flow depth $H$ was 12 cm. Both water and sediment were recirculated. Water came from the main laboratory circuit, while sediment was manually taken out from a sediment catcher in the outlet tank of the channel and placed in a sediment feeder in the upstream part of the channel. Piezometric probes and a magnetic flowmeter were used to measure flow depth and discharge, respectively. Scour depth was measured at an upstream point 1 cm from the cylinder with an optical sensor mounted on and fixed to the lid of the channel. Details about the optical device can be found in Ballio & Radice (2003) [4], and Radice et al. (2004) [76].

Preliminary runs were performed to determine the threshold flow velocity corresponding to the incipient motion of sediment for the flow depth. These results were then used in LB scour experiments.

After the preliminary runs, tests for bedforms were executed with three different discharges ($Q/Q_c$ = 1.2, where $Q$ = flow discharge and $Q_c$ = threshold discharge for incipient motion of the bed sediment, then to $Q/Q_c$ = 1.4, and after that to $Q/Q_c$ = 1.6), to characterize the pattern of the bedforms. In total two cycles of bedform tests were carried out.

The cylinder consisted of two parts. The upper part could slide into the lower part, which formed a jacket for the upper part, so that the cylinder could be completely buried under the sediment bed. Prior to the execution of a LB scour test, uniform flow conditions were achieved with the cylinder sank under the bed surface, to allow for slope adjusting and bedform development. Before the start of the actual scour test, continuous measurements of the bottom surface were taken at a fixed point 1 cm upstream from the face of the cylinder, in order to monitor bedform migration and to determine the reference average non scoured bed level. After that, once a stable initial condition was set, the upper part of the cylinder was rapidly lifted up, out of the sediment bed, which initiated the scour process. During the scour experiment, continuous measurements of water discharge were made in order to verify the maintenance of constant conditions in the channel.

The LB experiments with the cylindrical pier were carried out in three phases, where the flow discharge was first set to $Q/Q_c$ = 1.2, then to $Q/Q_c$ = 1.4, and later to $Q/Q_c$ = 1.6. In total two cycles of live-bed pier scour tests were carried out.

## 3.3 Water intake experiment set-up processing

Tests for water intakes were carried out at the Transport Research Institute (Russia) in a flume 3.5 m wide and 21 m long (see Figure 3.5).

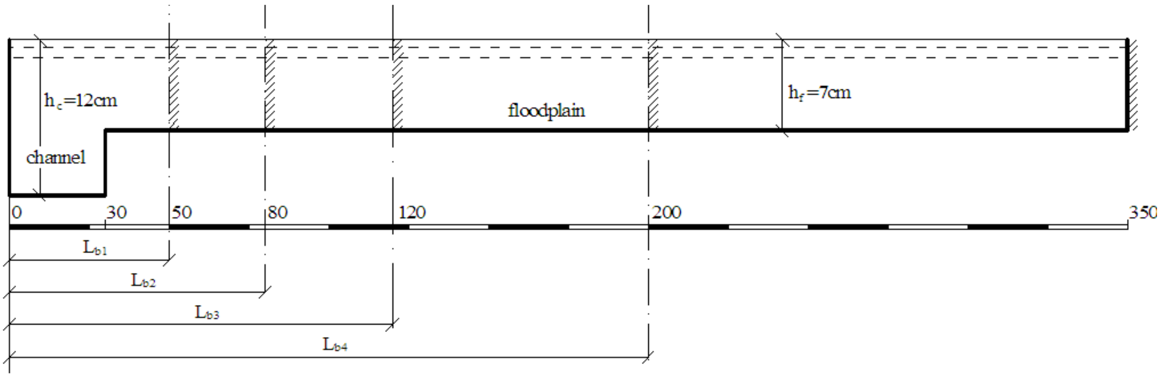


Figure 3.5. Cross-section view of the experiment flume

The tests were carried out under open-flow conditions (Table 3.1), while studying the flow distribution between the channel and the floodplain. Tests were performed with rigid and sand beds.

The fixed bed tests were performed for different flow contractions and Froude numbers (Table 3.1) in order to investigate flow velocity and water level changes near the water intakes with and without flow separation at structure.

The aim of the tests with a sand bed was to study scour processes, changes in flow velocity with time, effect of hydraulic parameters and contraction rate of the flow, grain size of the bed material, and scour development in time.

Table 3.1

Experiment data for open-flow conditions in the flume

| Test | $L$ (cm) | $h_f$ (cm) | $V$ (cm/s) | $Q$ (l/s) | $Fr$ | $Re_c$ | $Re_f$ |
|---|---|---|---|---|---|---|---|
| L1 | 350 | 7 | 6.47 | 16.60 | 0.078 | 7500 | 4390 |
| L2 | 350 | 7 | 8.58 | 22.70 | 0.103 | 10010 | 6060 |
| L3 | 350 | 7 | 10.30 | 23.60 | 0.124 | 12280 | 7190 |
| L4 | 350 | 7 | 8.16 | 20.81 | 0.098 | 10270 | 5590/5660 |
| L5 | 350 | 7 | 9.07 | 23.48 | 0.109 | 11280 | 6140/6410 |
| L6 | 350 | 7 | 11.10 | 28.31 | 0.134 | 13800 | 7550/7840 |

*Note:* $L$ – flume width; $h_f$ – flow depth in the floodplain; $V$ – approach flow velocity; $Q$ – flow discharge; $Fr$ – approach flow Froude number; $Re_c$ – Reynolds number in the channel; $Re_f$ – Reynolds number in the floodplain.

### 3.3.1 Water intake with flow separation

If the shape of a water intake is rectangular, the flow is separated at the upstream edge of the structure; this creates water level changes at the structure and a backwater.

The openings of the water intake model were 50, 80, 120, and 200 cm (see Figure 3.5). Flow contraction rate $Q/Q_b$ (where $Q$ is the flow discharge, and $Q_b$ is the discharge in the bridge opening under open-flow conditions) varied from 1.56 to 5.69 for the floodplain depth of 7 cm, and the Froude numbers varied from 0.078 to 0.124; the slope of the flume was 0.0012.

The sand bed tests were carried out under clear-water conditions. The sand was placed 1 m up and down the contraction of the flume. The mean grain size was $d_1$ = 0.24 mm and $d_2$ = 0.67 mm. The condition that $Fr_R = Fr_f$ was fulfilled, where $Fr_R$ and $Fr_f$ are the Froude numbers for the plain river and for the flume, respectively. The tests in the flume lasted for 7 hours, the length scale was 50 and the time scale was 7. With respect to the real conditions, the test time was equal to 2 days. This was the mean duration of time steps into which the flood hydrograph was divided. Scour development was examined with different flow parameters in time intervals within one 7-hour step and within two steps of the hydrograph, 7 hours each. The tests were carried out with one floodplain model and one side contraction of the flow.

### 3.3.2 Water intake without flow separation

In order for a water intake to have no flow separation with the bypassing flow, the shape of the water intake has to be smooth. In nature such structures with an elliptical and smooth shape are called guide banks.

Different forms of guide banks were studied by Latishenkov (1960) [42], and he recommended using elliptical guide banks as most acceptable in practice. The dimensions of the upper part of an elliptical guide bank, namely the turn and the length, were calculated according to the Latishenkov (1960) [42] method and were found to be dependent on flow contraction and main channel width. The length of the lower part of the guide bank was assumed to be half of the calculated upper part.

The openings of the water intake model were 50, 80, 120, and 200 cm (see Figure 3.5). Flow contraction rate $Q/Q_b$ varied respectively from 1.56 to 5.69 for the floodplain depth of 7 cm, and the Froude numbers varied from 0.078 to 0.124; the slope of the flume was 0.0012.

The sand bed tests were carried out under clear-water conditions. The sand was placed 1 m up and down the contraction of the flume. The mean grain size was $d_1$ = 0.24 mm and $d_2$ = 0.67 mm. The condition that $Fr_R = Fr_f$ was fulfilled. The tests in the flume lasted for 7 hours, the length scale was 50 and the time scale was 7. With respect to the real conditions, the test time was equal to 2 days. This was the mean duration of time steps into which the flood hydrograph was divided.

The development of the scour was examined with different flow parameters in time intervals within one 7-hour step and within two steps of the hydrograph, 7 hours each.

The tests were carried out with one floodplain model and one side contraction of the flow.

## 3.4 Water intake experiment data

Experiment data by Gjunsburgs & Neilands (2001) [24] and method for estimation of scour development in time during floods by Gjunsburgs & Neilands (2006) [23], and Gjunsburgs et al. (2004, 2005) [29 – 30] were used for computer modelling of scour depth development in time. Method for estimation of scour development in time during floods was confirmed by the experiment data of laboratory tests with duration of 7 hours (Gjunsburgs & Neilands, 2001) [24].

By using computer modelling, the duration of water intake laboratory tests of 7 hours was prolonged until the scour depth development reached the equilibrium stage. The computer modelling principle of scour depth development in time is discussed later in Chapter 3.4.1.

Table 3.2

Experiment data for water intakes with flow separation at the structure, at open-flow conditions

| Test | $L$ (cm) | $h_f$ (cm) | $V$ (cm/s) | $Q$ (l/s) | $Fr$ | $Re_c$ | $Re_f$ |
|---|---|---|---|---|---|---|---|
| AL1 | 350 | 7 | 6.47 | 16.60 | 0.078 | 7500 | 4390 |
| AL2 | 350 | 7 | 6.47 | 16.60 | 0.103 | 7500 | 4390 |
| AL3 | 350 | 7 | 6.47 | 16.60 | 0.124 | 7500 | 4390 |
| AL4 | 350 | 7 | 6.47 | 16.60 | 0.078 | 7500 | 4390 |
| AL5 | 350 | 7 | 8.58 | 22.70 | 0.103 | 10010 | 6060 |
| AL6 | 350 | 7 | 8.58 | 22.70 | 0.124 | 10010 | 6060 |
| AL7 | 350 | 7 | 8.58 | 22.70 | 0.078 | 10010 | 6060 |
| AL8 | 350 | 7 | 8.58 | 22.70 | 0.103 | 10010 | 6060 |
| AL9 | 350 | 7 | 10.30 | 23.60 | 0.124 | 12280 | 7190 |
| AL10 | 350 | 7 | 10.30 | 23.60 | 0.078 | 12280 | 7190 |
| AL12 | 350 | 7 | 10.30 | 23.60 | 0.124 | 12280 | 7190 |

*Note:* $L$ – flume width; $h_f$ – flow depth in the floodplain; $V$ – approach flow velocity; $Q$ – flow discharge; $Fr$ – approach flow Froude number; $Re_c$ – Reynolds number in the channel; $Re_f$ – Reynolds number in the floodplain.

Experiment data used for scour depth modelling at water intakes with flow separation at the structure (further in the text will be referred as AL tests) can be found in Table 3.2. Experimental data used for scour depth modelling at water intakes without flow separation at the structure (further in the text will be referred as EL tests) can be found in Table 3.3.

Table 3.3

Experiment data for water intakes without flow separation at the structure, at open-flow conditions

| Test | $L$ (cm) | $h_f$ (cm) | $V$ (cm/s) | $Q$ (l/s) | $Fr$ | $Re_c$ | $Re_f$ |
|---|---|---|---|---|---|---|---|
| EL1 | 350 | 7 | 6.47 | 16.60 | 0.078 | 7500 | 4390 |
| EL2 | 350 | 7 | 6.47 | 16.60 | 0.103 | 7500 | 4390 |
| EL3 | 350 | 7 | 6.47 | 16.60 | 0.124 | 7500 | 4390 |
| EL4 | 350 | 7 | 6.47 | 16.60 | 0.078 | 7500 | 4390 |
| EL5 | 350 | 7 | 8.58 | 22.70 | 0.103 | 10010 | 6060 |
| EL6 | 350 | 7 | 8.58 | 22.70 | 0.124 | 10010 | 6060 |
| EL7 | 350 | 7 | 8.58 | 22.70 | 0.078 | 10010 | 6060 |
| EL8 | 350 | 7 | 8.58 | 22.70 | 0.103 | 10010 | 6060 |
| EL9 | 350 | 7 | 10.30 | 23.60 | 0.124 | 12280 | 7190 |
| EL10 | 350 | 7 | 10.30 | 23.60 | 0.078 | 12280 | 7190 |
| EL11 | 350 | 7 | 10.30 | 23.60 | 0.103 | 12280 | 7190 |
| EL12 | 350 | 7 | 10.30 | 23.60 | 0.124 | 12280 | 7190 |

*Note:* $L$ – flume width; $h_f$ – flow depth in the floodplain; $V$ – approach flow velocity; $Q$ – flow discharge; $Fr$ – approach flow Froude number; $Re_c$ – Reynolds number in the channel; $Re_f$ – Reynolds number in the floodplain.

### 3.4.1 Computer modelling principle

By using computer program "RoBo" (Gjunsburgs et al., 2006) [31], the duration of water intake laboratory tests of 7 hours was prolonged until the scour depth development stopped and the equilibrium stage was reached. Scour computer modelling principle can be seen in Figure 3.6.

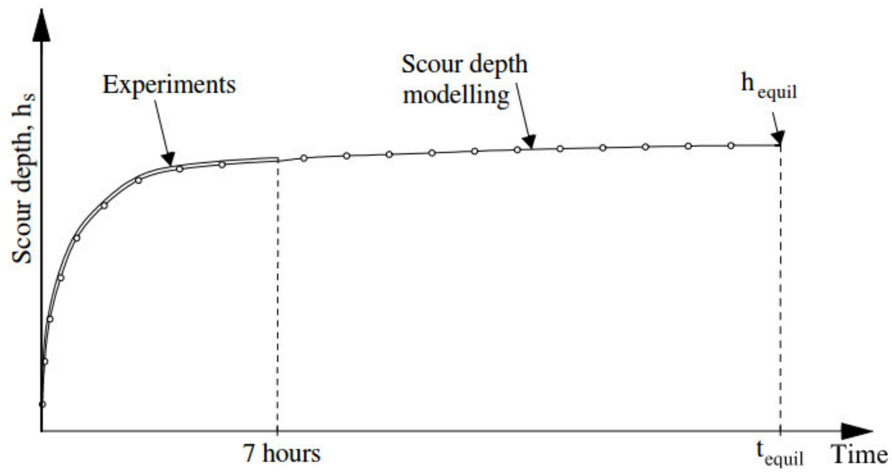



Figure 3.6. Scour computer modelling principle

At the end of each time interval there is a change in local flow velocity and in critical flow velocity, because of the changes in scour hole in the previous time interval. It means that with an increase of scour depth at the end of each time interval, current cross section increases, decreasing local flow velocity $V_{lt}$, on the other hand, critical flow velocity $V_{0t}$ increases because of an increase of the total flow depth – sum of the initial flow depth and the scour depth developed in the previous time interval (see Figure 3.7).

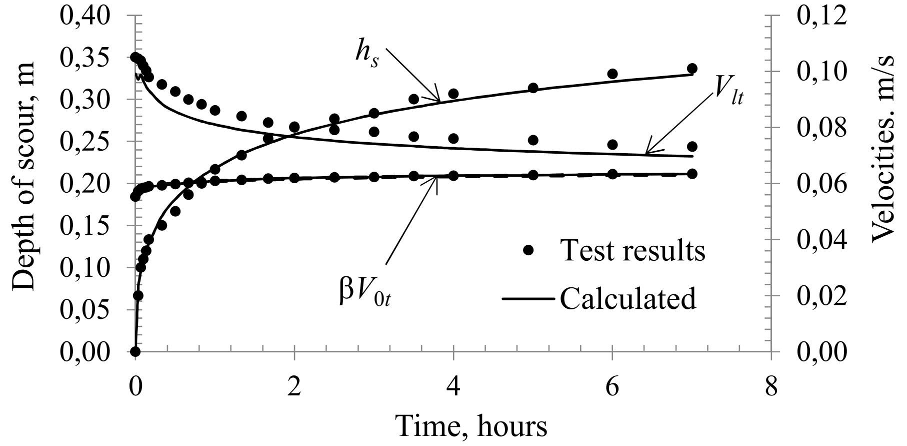



Figure 3.7. Change of local and critical flow velocities during scour development in time
*Note*: Test AL4. $h_s$ – scour depth; $V_{lt}$ – local flow velocity at the water intake; $\beta V_{0t}$ – recalculated critical flow velocity.

Since there is a necessity to re-calculate the scour depth in every time interval, in addition, time to reach the equilibrium stage can be up to a few days, the calculation of the scour depth development by the method of Gjunsburgs et al. (2004, 2005) [29 – 30] is mathematically complicated and long lasting, therefore the program "RoBo" was used (Gjunsburgs et al., 2006) [31]. "RoBo" is a simple, but a powerful tool with a mathematical algorithm written in Microsoft® Excel© program. The following parameters must be inputted: initial flow depth in the floodplain, flow contraction rate, maximum backwater, grain size, specific weight of the bed material, and thickness of the bed layers. After the calculation we have: local flow velocity, critical flow velocity, and scour depth changes at the end of each time interval, as well as, calculated velocity coefficient, and coefficient depending on flow contraction.

By using computer modelling (Gjunsburgs et al., 2006) [31], the number of time intervals and duration of the simulations are not restricted. The key consideration here is to determine an appropriate criterion, that limits the changes of scour depth in time to define the equilibrium stage for each of the experiment tests.

# 4 RESULTS

## 4.1 Flow-altering method against scour at abutments

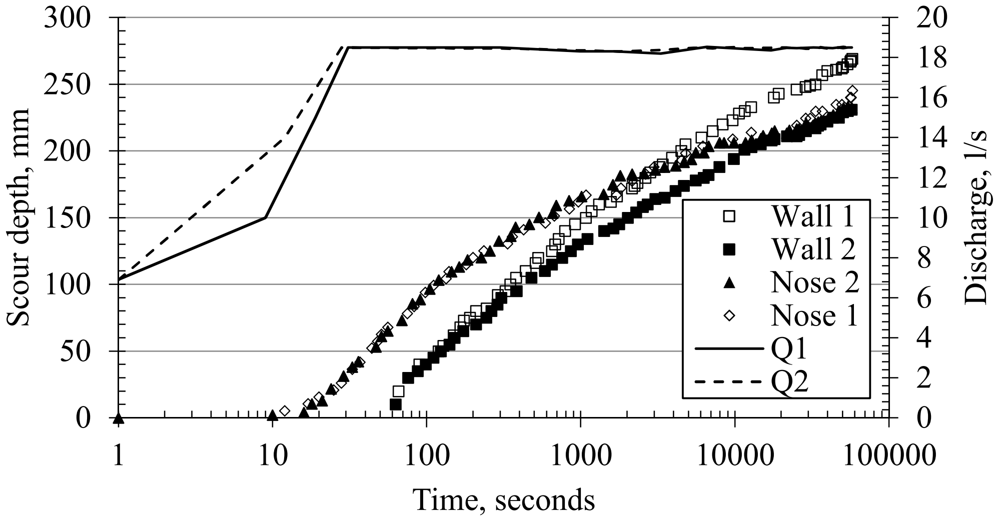



Figure 4.1. Comparison of scour depth evolution in time at the unprotected (Test 1) and the threaded abutments (Test 2)

Comparing the results from Test 1 (unprotected abutment) with Test 2 (threaded abutment) it is clear that the threads protect the wall and not the nose, nevertheless in the end the scour depth in Test 2 was less than in Test 1 (see Fig. 4.1). It looks like the scour hole needed to reach a certain scour depth before the threads started to weaken the flow to protect the wall from scouring, whereas a noticeable difference between Test 1 and Test 2 appears after the scour depth at the wall had reached 100 mm. At the end of Test 2 with the threaded abutment the reduction of scour depth at the wall was considerably good in compare with the scour depth at the unprotected abutment.

Table 4.1

Scour depth reduction rates at abutments

| Time (hours) | Scour Depth (cm) | | | | Scour depth reduction at the wall (%) | Scour depth reduction at the nose (%) |
|---|---|---|---|---|---|---|
| | Unprotected abutment | | Threaded abutment | | | |
| | Wall | Nose | Wall | Nose | | |
| 1 | 19.5 | 19.3 | 16.5 | 18.8 | 15.38 | 2.59 |
| 3 | 22.3 | 20.9 | 18.0 | 19.9 | 19.28 | 4.78 |
| 7 | 24.6 | 21.9 | 21.1 | 21.7 | 14.23 | 0.91 |
| 16 | 26.9 | 24.5 | 23.1 | 23.2 | 14.13 | 5.31 |

The results from Table 4.1 show that the thread protection method weakens the flow which results in scour reduction at the wall. In the beginning of Test 2 the scour depth reduction at the wall was around 15 %, then at one point (Test 2, after 3 hours) the scour depth reduction reached nearly 20 %, after which it stabilized around 14 % which was also the final value of scour depth reduction in Test 2 for the wall. The final scour reduction value in Test 2 for the abutments nose was 5.3 %, which was also a considerably good result.

What is evident in all of the scour reduction cases, that the threaded protection mostly affects only the wall, which is quite logical, since it intercepts the vertical component of the flow – the downflow, which apparently is the major cause of scouring at the wall. Since the protection does not affect the scouring at the abutments nose, the nose is probably affected more by a horizontal flow component so the vertical flow component affects the wall, but the horizontal flow component has an effect on the scouring at the abutments nose.

Flow-altering countermeasures are designed and built to change the structure of the hydrodynamic pattern. The latter, in turn, triggers and controls the kinematics of the sediments on the bottom of the scour hole. As a result, the sediment motion pattern is also expected to undergo some modifications if a flow-altering countermeasure is installed (Radice & Lauva, 2013) [74].

## 4.2 Live-bed pier scour

In total three different live-bed (LB) and one clear-water (CW) experiments were conducted at the pier. The three LB tests are much shorter than the CW test, since it was more difficult to maintain the proper channel conditions for these tests. Our measurements show that the LB experiments are characterized by a similar behavior in the development phase before they reach the equilibrium stage, where the scour depth oscillates around mean scour depth value $d_s$.

The oscillations occur because of the bedforms passing by the scour hole and the pier. As the ratio of $U/U_c$ increases, equilibrium time is reached faster with decreasing scour depth $d_s$.

For the LB experiments with $Q/Q_c$ = 1.2, no actual bedforms were evident at this stage, the sediments were moving along the bed. This is in accordance with the theory found in the literature of Coleman & Melville (1996) [12] for alluvial flow over a plane sediment bed with the sediment in motion, regular trains of small wavelets form on the sediment bed and then develop into ripples or dunes in equilibrium with the flow. Regular trains of small wavelets (wave height was about the size of the sediment diameter) were indeed evident at this stage of the LB experiment, when the flow discharge was enough to initiate sediment bed movement along the whole channel, but not yet high enough to be able to build some larger bedforms like ripples or dunes.

As soon as the flow discharge was increased to $Q/Q_c$ = 1.4, the sediment bed started to evolve and change from smaller bedforms (ripples) to bigger ones (dunes) until stable conditions were reached (see Figure 4.2).

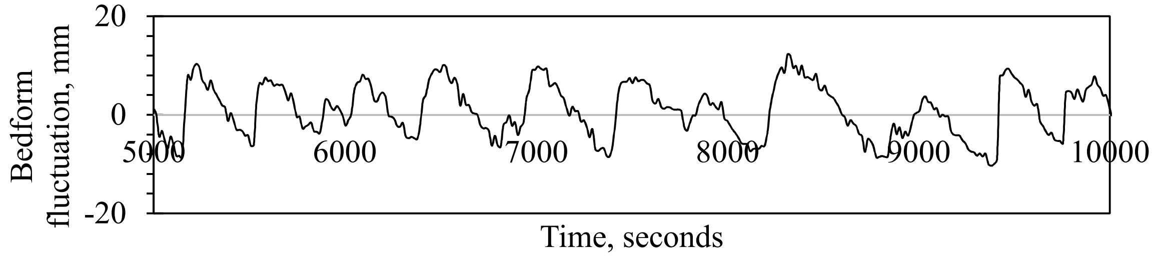



Figure 4.2. Bedforms at flow discharge $Q/Q_c$ = 1.4
*Note*: Bedforms move from right to left.

When the discharge was increased to $Q/Q_c$ = 1.6, dune height and length started to change, namely dunes got higher, shorter and also dune period decreased as the smaller bedforms started to merge with each other and create bigger ones (see Figure 4.3).

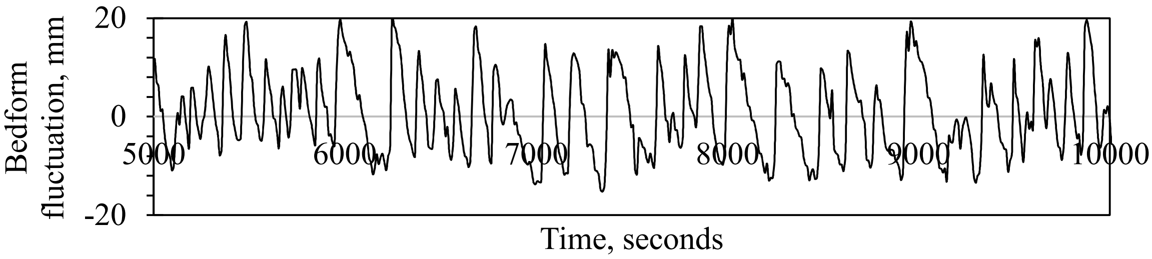



Figure 4.3. Bedforms at flow discharge $Q/Q_c$ = 1.6
*Note*: Bedforms move from right to left.

Wavelets are found to be of a preferred wavelength which is relatively insensitive to the characteristics of the applied flow primarily a function of the size of the sediment, these wavelengths λ for alluvial and laminar open-channel flows over beds of quartz and lightweight sediments of size $d_i$ = 0.2 ÷ 1.6 mm being simply described by $\lambda = 175 d_i^{0.75}$, where λ and $d_i$ are expressed in millimeters (Coleman & Eling, 2000) [11]. As it was already noted before, the sediment size used in the presented experiments is $D_{50}$ = 3 mm, therefore the recommended equation by Coleman & Eling (2000) [11] for dune wavelength prediction is not valid in this case and also the calculated wavelength with the above mentioned equation is two times smaller than the actual wavelength since our grain size is almost twice the size of the equations upper boundary. Data with the actual dune height and length can be found in Table 4.2, where $T$ = duration of the test; $\Delta_D$ = mean height of the bedform; $\lambda_D$ = mean length of the bedform; and $T_D$ = mean period of the bedform.

Table 4.2

Bedform test results

| Test | $Q$ (l/s) | $Q/Q_c$ | $T$ (s) | $\Delta_D$ (mm) | $\lambda_D$ (mm) | $T_\Delta$ (s) |
|---|---|---|---|---|---|---|
| BF1 | 15.78 | 1.2 | 22200 | 3 | -* | -* |
| BF2 | 18.40 | 1.4 | 5700 | 19 | 710 | 500 |
| BF3 | 21.04 | 1.6 | 8820 | 30 | 520 | 125 |
| BF4 | 15.78 | 1.2 | 18080 | 3 | -* | -* |
| BF5 | 18.40 | 1.4 | 18120 | 25 | 830 | 667 |
| BF6 | 21.04 | 1.6 | 18080 | 35 | 520 | 125 |

* No evident bedforms.

The live-bed pier scour test P1 at $Q/Q_c$ = 1.2 was run for approximately 2 hours and 40 minutes, after that the pier was pushed back into the jacket, the discharge was set to $Q/Q_c$ = 1.4 and the channel was let to run for 1 hour in order for the sediment to completely fill the scour hole and allow the bedforms to fully develop to equilibrium, after that the pier was pulled out again; the second live-bed pier scour test P2 lasted for approximately 1 hour and 50 minutes, after which the pier was pushed back into the jacket again, the discharge was set to $Q/Q_c$ = 1.6 and the channel was let to run for another hour for the sediment to completely fill the scour hole and allow the bedforms to fully develop to equilibrium. Test P3 total duration was 2 hours and 4 minutes. The CW test was run for 13 hours; however, the equilibrium was not reached. Results of LB and CW pier scour tests can be seen in Table 4.3, where $d_s$ = equilibrium scour depth; and $t_e$ = equilibrium time of scour.

Table 4.3

Clear-water and live-bed pier scour test results

| Test | $Q$ (l/s) | $Q/Q_c$ | $T$ (s) | $d_s$ (mm) | $t_e$ (s) |
|---|---|---|---|---|---|
| CW | 11.84 | 0.9 | 46880 | 112* | 46880* |
| P1 | 15.78 | 1.2 | 9580 | 98 | 600 |
| P2 | 18.40 | 1.4 | 6670 | 94 | 300 |
| P3 | 21.04 | 1.6 | 7455 | 80 | 180 |
| P4 | 15.78 | 1.2 | 18000 | 106 | 1500 |
| P5 | 18.40 | 1.4 | 18000 | 100 | 300 |
| P6 | 21.04 | 1.6 | 11050 | 80 | 100 |

* The equilibrium was not achieved.

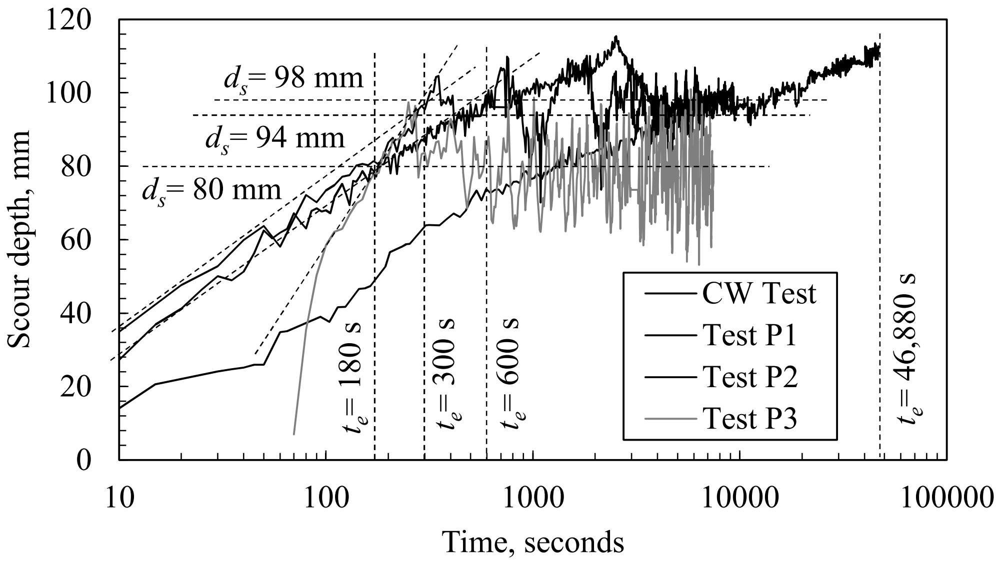



Figure 4.4. Temporal development of scour at pier

Most literature results from CW experiments indicate that the rising phase of the scour process can be approximated with linear trends in a semi logarithmic plot (Cardoso & Bettess, 1999) [9]. Figure 4.4 shows CW scour trend along with LB scour time series that can also be approximated by logarithmic functions, until they reach a condition of dynamic equilibrium wherein the scour depth fluctuates around a mean value.

For all live-bed pier scour tests P1 to P6 the equilibrium time $t_e$ was easily and robustly estimated by the following steps: (1) the equilibrium scour depth is determined by drawing a mean horizontal line representing the mean scour depth in the equilibrium phase and (2) a line is drawn fitting the scour data in the rising phase, and its interception with the line representing equilibrium scour depth is defined as the equilibrium scour time (see Figure 4.4). Since the equilibrium was not achieved in the CW test, time $t_e$ in Table 4.3 corresponds to the total duration of the test, and scour depth $d_s$ is the scour depth at the end of the test.

## 4.3 Equilibrium time of scour at water intakes

Looking at Tables 4.4 – 4.7, where the results of computer modeled and calculated data of water intake tests can be found, a certain parameter trend can be seen, depending on the changes of the contraction rate of the flow and approach flow Froude number, which reflects the changes of mean approach flow velocity. This trend is the same both for water intake tests with and without flow separation at the structure, and both sand grain diameters.

Table 4.4

Computer modeled and calculated results of water intake with flow separation test data, sand grain size $d_1$ = 0.24 mm

| TEST | $Q/Q_b$ | $h_{equil}/h_f$ | $D_i$ | $x_i$ | $N_i$-$N_0$ | $Fr$ | $V_l/\beta V_0$ | $k$ | $A_i$ | $t_{comp}$ (hours) | $t_{form}$ (hours) | $t_{form}/t_{comp}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| AL1 | 5.27 | 2.31 | 29.45 | 2.13 | 6.92 | 0.078 | 2.00 | 0.764 | 1.66 | 99.00 | 95.93 | 0.97 |
| AL2 | 5.69 | 3.04 | 18.97 | 2.49 | 20.58 | 0.103 | 2.34 | 0.739 | 1.23 | 190.00 | 183.66 | 0.97 |
| AL3 | 5.55 | 3.30 | 17.26 | 2.62 | 29.41 | 0.124 | 2.58 | 0.739 | 1.06 | 228.00 | 238.78 | 1.05 |
| AL4 | 3.66 | 1.69 | 119.11 | 1.82 | 2.13 | 0.078 | 1.79 | 0.838 | 0.89 | 103.00 | 119.10 | 1.16 |
| AL5 | 3.87 | 2.33 | 45.44 | 2.14 | 7.11 | 0.103 | 2.15 | 0.838 | 1.07 | 144.00 | 152.02 | 1.06 |
| AL6 | 3.78 | 2.83 | 21.77 | 2.39 | 15.46 | 0.124 | 2.49 | 0.838 | 1.36 | 172.00 | 158.37 | 0.92 |
| AL7 | 2.60 | 0.96 | 306.05 | 1.46 | 0.33 | 0.078 | 1.45 | 0.890 | 1.06 | 48.00 | 46.85 | 0.98 |
| AL8 | 2.69 | 1.58 | 130.67 | 1.77 | 1.68 | 0.103 | 1.83 | 0.890 | 0.94 | 93.00 | 103.12 | 1.11 |
| AL9 | 2.65 | 1.98 | 53.12 | 1.97 | 3.83 | 0.124 | 2.10 | 0.890 | 1.36 | 100.80 | 95.63 | 0.95 |
| AL10 | 1.56 | 0.38 | 963.72 | 1.18 | 0.03 | 0.078 | 1.15 | 0.925 | 0.98 | 12.40 | 11.42 | 0.92 |
| AL12 | 1.67 | 0.77 | 398.77 | 1.37 | 0.17 | 0.124 | 1.38 | 0.925 | 1.12 | 36.00 | 32.26 | 0.90 |

*Note:* $Q/Q_b$ – flow contraction rate; $h_{equil}/h_f$ – relative scour depth; $D_i$, $x_i$, $N_i$-$N_0$, $A_i$ – calculated parameters; $t_{comp}$ – computer modeled equilibrium time of scour; $t_{form}$ – calculated equilibrium time of scour; $Fr$ – approach flow Froude number; $V_l/\beta V_0$ – ratio of local flow velocity to the recalculated critical flow velocity; $k$ – flow contraction coefficient.

Table 4.5

Computer modeled and calculated results of water intake with flow separation test data, sand grain size $d_2 = 0.67$ mm

| TEST | $Q/Q_b$ | $h_{equil}/h_f$ | $D_i$ | $x_i$ | $N_i$-$N_0$ | $Fr$ | $V_l/\beta V_0$ | $k$ | $A_i$ | $t_{comp}$ (hours) | $t_{form}$ (hours) | $t_{form}/t_{comp}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| AL1 | 5.27 | 1.51 | 63.53 | 1.74 | 1.44 | 0.078 | 1.54 | 0.764 | 0.784 | 42.00 | 43.17 | 1.03 |
| AL2 | 5.69 | 2.10 | 31.22 | 2.03 | 4.79 | 0.103 | 1.81 | 0.739 | 0.723 | 63.00 | 70.37 | 1.12 |
| AL3 | 5.55 | 2.32 | 25.61 | 2.13 | 2.13 | 0.124 | 1.96 | 0.747 | 0.688 | 72.00 | 84.25 | 1.17 |
| AL4 | 3.66 | 1.00 | 144.02 | 1.48 | 0.38 | 0.078 | 1.39 | 0.834 | 0.733 | 24.00 | 25.53 | 1.06 |
| AL5 | 3.87 | 1.53 | 62.00 | 1.74 | 1.48 | 0.103 | 1.66 | 0.819 | 0.789 | 42.00 | 43.29 | 1.03 |
| AL6 | 3.78 | 1.94 | 33.23 | 1.94 | 1.94 | 0.124 | 1.92 | 0.825 | 0.828 | 57.00 | 54.55 | 0.96 |
| AL7 | 2.60 | 0.41 | 408.46 | 1.19 | 0.03 | 0.078 | 1.12 | 0.890 | 0.787 | 6.00 | 5.75 | 0.96 |
| AL8 | 2.69 | 0.91 | 158.53 | 1.44 | 0.28 | 0.103 | 1.42 | 0.890 | 0.781 | 21.00 | 21.08 | 1.00 |
| AL9 | 2.65 | 1.24 | 96.12 | 1.60 | 1.60 | 0.124 | 1.63 | 0.890 | 0.757 | 33.00 | 33.68 | 1.02 |
| AL10 | 1.56 | NO SCOUR | | | | | | | | | | |
| AL12 | 1.67 | 0.26 | 584.43 | 1.12 | 0.01 | 0.124 | 1.07 | 0.925 | 0.756 | 3.00 | 2.64 | 0.88 |

*Note:* $Q/Q_b$ – flow contraction rate; $h_{equil}/h_f$ – relative scour depth; $D_i$, $x_i$, $N_i$-$N_0$, $A_i$ – calculated parameters; $t_{comp}$ – computer modeled equilibrium time of scour; $t_{form}$ – calculated equilibrium time of scour; $Fr$ – approach flow Froude number; $V_l/\beta V_0$ – ratio of local flow velocity to the recalculated critical flow velocity; $k$ – flow contraction coefficient.

Table 4.6

Computer modeled and calculated results of water intake without flow separation test data, sand grain size $d_1$ = 0.24 mm

| TEST | $Q/Q_b$ | $h_{equil}/h_f$ | $D_i$ | $x_i$ | $N_i$-$N_0$ | $Fr$ | $V_l/\beta V_0$ | $t_{comp}$ (hours) | $t_{form}$ (hours) | $t_{form}/t_{comp}$ |
|---|---|---|---|---|---|---|---|---|---|---|
| EL1 | 5.27 | 1.63 | 104.71 | 1.80 | 1.86 | 0.078 | 2.11 | 96.00 | 93.33 | 0.97 |
| EL2 | 5.69 | 2.17 | 52.28 | 1.80 | 5.46 | 0.103 | 2.50 | 132.00 | 134.25 | 1.02 |
| EL3 | 5.55 | 2.41 | 40.54 | 2.18 | 8.14 | 0.124 | 2.69 | 153.60 | 155.17 | 1.01 |
| EL4 | 3.66 | 1.49 | 166.55 | 1.72 | 1.37 | 0.078 | 2.00 | 92.10 | 107.42 | 1.17 |
| EL5 | 3.87 | 2.02 | 47.49 | 1.72 | 4.09 | 0.103 | 2.39 | 100.80 | 91.35 | 0.91 |
| EL6 | 3.78 | 2.52 | 39.32 | 2.23 | 9.72 | 0.124 | 2.77 | 151.20 | 179.82 | 1.19 |
| EL7 | 2.60 | 0.82 | 450.47 | 1.39 | 0.20 | 0.078 | 1.53 | 45.00 | 42.96 | 0.94 |
| EL8 | 2.69 | 1.54 | 130.76 | 1.39 | 1.55 | 0.103 | 2.04 | 90.00 | 95.57 | 1.06 |
| EL9 | 2.65 | 1.93 | 55.08 | 1.94 | 3.47 | 0.124 | 2.33 | 84.00 | 89.86 | 1.07 |
| EL10 | 1.56 | 0.46 | 957.28 | 1.22 | 0.04 | 0.078 | 1.30 | 18.00 | 18.89 | 1.05 |
| EL11 | 1.66 | 0.65 | 619.50 | 1.22 | 0.10 | 0.103 | 1.42 | 30.50 | 29.96 | 0.98 |
| EL12 | 1.67 | 0.81 | 467.54 | 1.39 | 0.20 | 0.124 | 1.53 | 45.00 | 43.75 | 0.97 |

*Note:* $Q/Q_b$ – flow contraction rate; $h_{equil}/h_f$ – relative scour depth; $D_i$, $x_i$, $N_i$-$N_0$ – calculated parameters; $t_{comp}$ – computer modeled equilibrium time of scour; $t_{form}$ – calculated equilibrium time of scour; $Fr$ – approach flow Froude number; $V_l/\beta V_0$ – ratio of local flow velocity to the recalculated critical flow velocity.

Table 4.7

Computer modeled and calculated results of water intake without flow separation test data, sand grain size $d_2 = 0.67$ mm

| TEST | $Q/Q_b$ | $h_{equil}/h_f$ | $D_i$ | $x_i$ | $N_i$-$N_0$ | $Fr$ | $V_l/\beta V_0$ | $t_{comp}$ (hours) | $t_{form}$ (hours) | $t_{form}/t_{comp}$ |
|---|---|---|---|---|---|---|---|---|---|---|
| EL1 | 5.27 | 0.960 | 198.980 | 1.462 | 0.328 | 0.078 | 1.63 | 27.6 | 30.68 | 1.11 |
| EL2 | 5.69 | 1.399 | 89.804 | 1.679 | 1.107 | 0.103 | 1.94 | 42.0 | 46.78 | 1.11 |
| EL3 | 5.55 | 1.594 | 64.303 | 1.776 | 1.735 | 0.1243 | 2.08 | 51.0 | 52.47 | 1.03 |
| EL4 | 3.66 | 0.841 | 232.109 | 1.404 | 0.220 | 0.078 | 1.55 | 23.1 | 23.98 | 1.04 |
| EL5 | 3.87 | 1.273 | 109.584 | 1.617 | 0.808 | 0.103 | 1.85 | 39.0 | 41.63 | 1.07 |
| EL6 | 3.78 | 1.683 | 59.280 | 1.820 | 2.102 | 0.1243 | 2.15 | 51.0 | 58.61 | 1.15 |
| EL7 | 2.60 | 0.291 | 607.703 | 1.132 | 0.013 | 0.078 | 1.19 | 4.5 | 3.68 | 0.82 |
| EL8 | 2.69 | 0.886 | 216.703 | 1.426 | 0.258 | 0.103 | 1.58 | 24.0 | 26.35 | 1.10 |
| EL9 | 2.65 | 1.203 | 113.481 | 1.582 | 0.669 | 0.1243 | 1.80 | 36.0 | 35.74 | 0.99 |
| EL10 | NO SCOUR | | | | | | | | | |
| EL11 | 1.66 | 0.156 | 873.163 | 1.065 | 0.003 | 0.103 | 1.10 | 1.5 | 1.19 | 0.79 |
| EL12 | 1.67 | 0.291 | 607.779 | 1.132 | 0.013 | 0.1243 | 1.19 | 4.3 | 3.68 | 0.86 |

*Note:* $Q/Q_b$ – flow contraction rate; $h_{equil}/h_f$ – relative scour depth; $D_i$, $x_i$, $N_i$-$N_0$ – calculated parameters; $t_{comp}$ – computer modeled equilibrium time of scour; $t_{form}$ – calculated equilibrium time of scour; $Fr$ – approach flow Froude number; $V_l/\beta V_0$ – ratio of local flow velocity to the recalculated critical flow velocity.

For the same approach flow velocity $V$ and approach flow Froude number, but increasing contraction rate of the flow $Q/Q_b$, relative depth of scour $h_s/h_f$ is increasing as the scour depth $h_s$ is increasing as well. Parameter $D_i$, which depends from local flow velocity $V_l$, mean sand grain size $d_{50}$ and relative depth of scour $h_s/h_f$ is significantly decreasing with the increase of contraction rate of the flow $Q/Q_b$ at the same approach flow Froude number. Parameters $X_i$ and $N_i$-$N_0$, which depend from relative depth of scour $h_s/h_f$ are increasing as the scour depth $h_s$ increases as well with the same approach flow Froude number and increasing contraction rate of the flow $Q/Q_b$. Parameter $k$ (for water intake tests with flow separation) decreases as the contraction rate of the flow $Q/Q_b$ increases for the same approach flow Froude number. Relative velocity of the flow $V_l/\beta V_0$ is increasing as the local flow velocity $V_l$ is increasing with increasing contraction rate of the flow $Q/Q_b$ and the same approach flow Froude number. Equilibrium time of scour $t_e$ is increasing as the scour depth $h_s$ keeps increasing as well for the same approach flow Froude number and increasing flow contraction $Q/Q_b$. All of the before mentioned parameter trends when the approach flow Froude number is not changing, but the contraction rate of the flow $Q/Q_b$ is increasing are observed also when the approach flow Froude number is increasing at the same flow contraction rate $Q/Q_b$ (see Tables 4.4 – 4.7).

### 4.3.1 Comparison of calculated and computer modeled equilibrium times of scour

To verify the suggested equilibrium time of scour evaluation methods, calculated time of scour values were compared to computer modeled equilibrium time of scour values, as shown in Tables 4.8 – 4.11.

A percent relative error ε was calculated for each of the tests:

$$\varepsilon(\%) = \frac{\left(t_{form} - t_{comp}\right)}{t_{comp}} \cdot 100 \,, \tag{4.1}$$

where $t_{form}$ – equilibrium time of scour calculated by the developed method, days;
$t_{comp}$ – computer modeled equilibrium time of scour, days.

Calculated values of equilibrium time of scour show a good agreement with the computer modeled equilibrium time of scour values for tests at water intakes with flow separation at the structure, with sand grain size $d_1 = 0.24$ mm (see Table 4.8).

However, taking a closer look at Table 4.8 several estimated equilibrium time of scour values from Tests AL4, AL8 and AL12 seem to be over and under evaluated (by more than 10

%) in compare with the computer modeled equilibrium time of scour values, since their percent relative error is 15.63 %, 10.88 % and -10.39 %, respectively.

Table 4.8

Comparison of calculated and computer modeled equilibrium times of scour at water intakes with flow separation, sand grain size $d_1 = 0.24$ mm

| Tests | $t_{comp}$ (hours) | $t_{form}$ (hours) | $\frac{t_{form}}{t_{comp}}$ | ε (%) |
|---|---|---|---|---|
| AL1 | 99.0 | 95.93 | 0.97 | -3.10 |
| AL2 | 190.0 | 183.66 | 0.97 | -3.34 |
| AL3 | 228.0 | 238.78 | 1.05 | 4.73 |
| AL4 | 103.0 | 119.10 | 1.16 | 15.63 |
| AL5 | 144.0 | 152.02 | 1.06 | 5.57 |
| AL6 | 172.0 | 158.37 | 0.92 | -7.92 |
| AL7 | 48.0 | 46.85 | 0.98 | -2.40 |
| AL8 | 93.0 | 103.12 | 1.11 | 10.88 |
| AL9 | 100.8 | 95.63 | 0.95 | -5.13 |
| AL10 | 12.4 | 11.42 | 0.92 | -7.90 |
| AL12 | 36.0 | 32.26 | 0.90 | -10.39 |

On the other hand, the rest of the calculated equilibrium time of scour values from Table 4.8 show a very close agreement with the computer modeled ones, particularly highlighting estimated equilibrium time of scour values from Tests AL1, AL2 and AL7, where the percent relative error in compare with the experimentally derived ones is just -3.10 %, -3.34 % and -2.40 %, respectively.

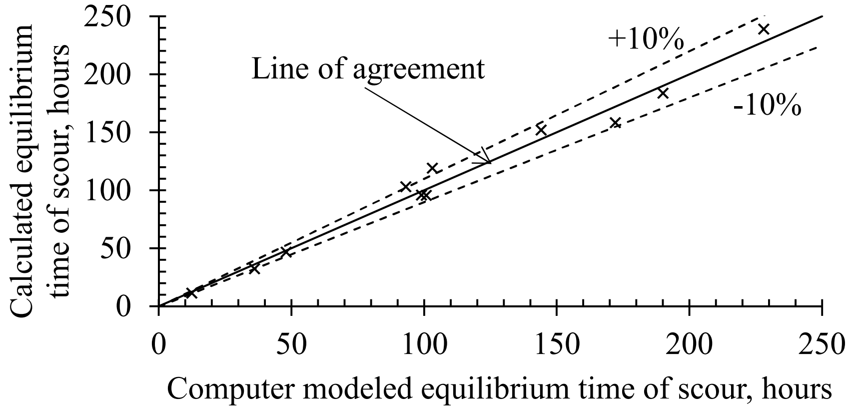



Figure 4.5. Comparison of calculated and computer modeled equilibrium times of scour
*Note*: Tests at water intakes with flow separation, sand grain size $d_1 = 0.24$ mm.

The average percent relative error calculated with Eq. (4.1) for data set of water intake tests with flow separation at the structure, with sand grain size $d_1 = 0.24$ mm is 7.0 % (see Table 4.8 and Figure 4.5).

Comparison of calculated and computer modeled time of scour values for water intake tests with flow separation at the structure, with sand grain size $d_2 = 0.67$ mm can be found in Table 4.9, results show a good agreement as well. In Test AL10 there were no signs of scour at the structure with sand grain size $d_2 = 0.67$ mm, therefore this row in the table is filled in with words "no scour" (see Table 4.9).

The estimated equilibrium time of scour values from Tests AL2, AL3 and AL12 are over and under evaluated (by more than 10 %) in compare with the computer modeled ones, and their percent relative error is 11.70 %, 17.01 % and -12.00 %, respectively (see Table 4.9). Whereas, the rest of the calculated equilibrium time of scour values in Table 4.9 show a close agreement with the computer modeled equilibrium times of scour values, with a percent relative error of under 6.5 %. Here the calculated equilibrium time of scour values for Tests AL1, AL8 and AL9 have to be especially highlighted, since they are in a very close agreement with the computer modeled equilibrium time of scour values, having a percent relative error of 2.79 %, 0.38 % and 2.06 %, respectively (see Table 4.9).

Table 4.9

Comparison of calculated and computer modeled equilibrium times of scour at water intakes with flow separation, sand grain size $d_2 = 0.67$ mm

| Tests | $t_{comp}$ (hours) | $t_{form}$ (hours) | $\frac{t_{form}}{t_{comp}}$ | ε (%) |
|---|---|---|---|---|
| AL1 | 42.0 | 43.17 | 1.03 | 2.79 |
| AL2 | 63.0 | 70.37 | 1.12 | 11.70 |
| AL3 | 72.0 | 84.25 | 1.17 | 17.01 |
| AL4 | 24.0 | 25.53 | 1.06 | 6.38 |
| AL5 | 42.0 | 43.29 | 1.03 | 3.07 |
| AL6 | 57.0 | 54.55 | 0.96 | -4.30 |
| AL7 | 6.0 | 5.75 | 0.96 | -4.17 |
| AL8 | 21.0 | 21.08 | 1.00 | 0.38 |
| AL9 | 33.0 | 33.68 | 1.02 | 2.06 |
| AL10 | NO SCOUR | | | |
| AL12 | 3.0 | 2.64 | 0.88 | -12.0 |

The average percent relative error calculated with Eq. (4.1) for data set of water intake tests with flow separation at the structure and sand grain size $d_2 = 0.67$ mm is 7.4 % (see Table 4.9 and Figure 4.6).

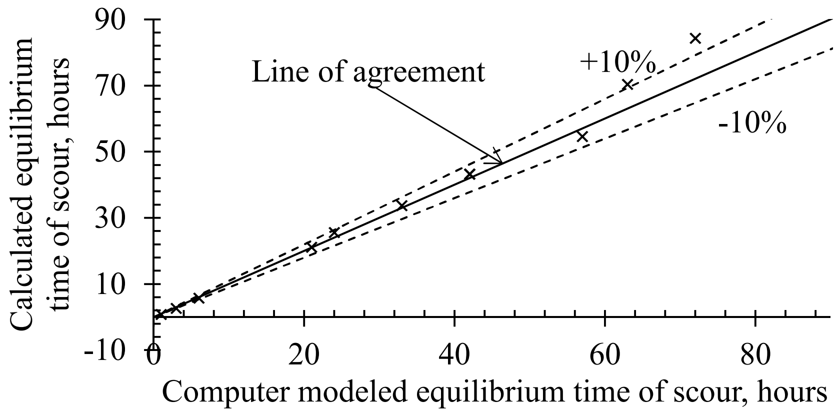



Figure 4.6. Comparison of calculated and computer modeled equilibrium times of scour
*Note*: Tests at water intakes with flow separation, sand grain size $d_2 = 0.67$ mm.

Table 4.10 contains the comparison of calculated and computer modeled time of scour values for water intake tests without flow separation at the structure, with sand grain size $d_1 = 0.24$ mm.

Table 4.10

Comparison of calculated and computer modeled equilibrium times of scour at water intakes without flow separation, sand grain size $d_1 = 0.24$ mm

| Tests | $t_{comp}$ (hours) | $t_{form}$ (hours) | $\frac{t_{form}}{t_{comp}}$ | ε (%) |
|---|---|---|---|---|
| EL1 | 96.0 | 93.33 | 1.03 | -2.78 |
| EL2 | 132.0 | 134.25 | 0.98 | 1.70 |
| EL3 | 153.6 | 155.17 | 0.99 | 1.02 |
| EL4 | 92.1 | 107.42 | 0.86 | 16.63 |
| EL5 | 100.8 | 91.35 | 1.10 | -9.38 |
| EL6 | 151.2 | 179.82 | 0.84 | 18.93 |
| EL7 | 45.0 | 42.96 | 1.05 | -4.53 |
| EL8 | 90.0 | 95.57 | 0.94 | 6.19 |
| EL9 | 84.0 | 89.86 | 0.93 | 6.98 |
| EL10 | 18.0 | 18.89 | 0.95 | 4.94 |
| EL11 | 30.5 | 29.96 | 1.02 | -1.77 |
| EL12 | 45.0 | 43.75 | 1.03 | -2.78 |

The calculated equilibrium time of scour values from Tests EL4, EL5 and EL6 (see Table 4.10) are over and under evaluated in compare with the computer modeled ones, by more than 10 % for Tests EL4 and EL6, where the percent relative error is 16.63 % and 18.93 %, respectively, and slightly under 10 % in Test EL5, where the relative percent error is -9.38 %. As for the rest of the calculated equilibrium time of scour values in Table 4.10, they show a close agreement with the computer modeled equilibrium times of scour values, with a percent relative error of under 7.0 %. Here the calculated equilibrium time of scour values with the closest agreement with the experimentally obtained ones are for Tests EL2, EL3 and EL11, having a percent relative error of 1.70 %, 1.02 % and -1.77 %, respectively (see Table 4.10).

The average percent relative error calculated with Eq. (4.1) for data set of water intake tests without flow separation at the structure, with sand grain size $d_1 = 0.24$ mm is 6.5 % (see Table 4.10 and Figure 4.7).

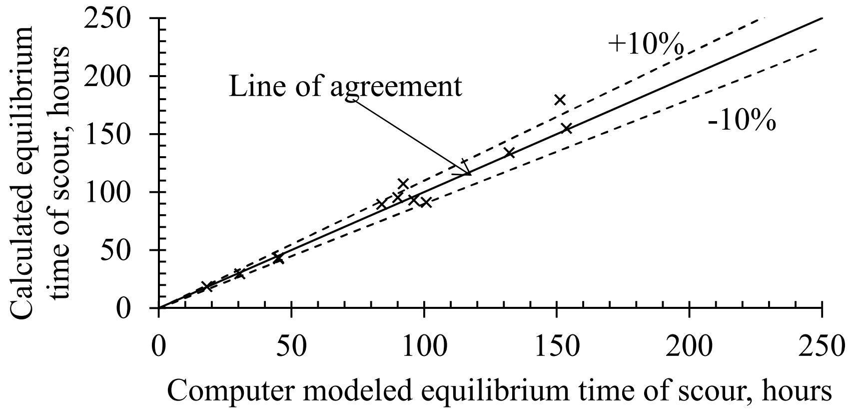



Figure 4.7. Comparison of calculated and computer modeled equilibrium times of scour
*Note*: Tests at water intakes without flow separation, sand grain size $d_1 = 0.24$ mm.

The comparison of calculated and computer modeled time of scour values for water intake tests without flow separation at the structure and sand grain size $d_2 = 0.67$ mm can be found in Table 4.11. In Test EL10 the conditions in the channel and the floodplain were not turbulent enough to create scour at the structure with sand grain size $d_2 = 0.67$ mm, therefore this row in the table is filled in with words "no scour" (see Table 4.11).

Having a closer look at Table 4.11, more than half of the estimated equilibrium time of scour values, in comparison with the computer modeled values, from tests at water intakes without flow separation and sand grain size $d_2 = 0.67$ mm, seem to be over and under evaluated (by more than 10 %). The estimated equilibrium time of scour values from Tests EL6, EL7 and EL11 have the poorest agreement with the computer modeled equilibrium time of scour values, having a percent relative errors of 14.92 %, -18.22 % and -20.67 %, respectively; estimated equilibrium time of scour values from Tests EL1 and EL2 are also outside the 10 % prediction error rate, having percent relative errors of 11.16 % and 11.38 %, respectively. The calculated value of equilibrium time of scour from Test EL8 has a percent relative error of 9.79 %, which is slightly below the 10 % prediction error rate.

Table 4.11

Comparison of calculated and computer modeled equilibrium times of scour at water intakes without flow separation, sand grain size $d_2 = 0.67$ mm

| Tests | $t_{comp}$ (hours) | $t_{form}$ (hours) | $\frac{t_{form}}{t_{comp}}$ | ε (%) |
|---|---|---|---|---|
| EL1 | 27.6 | 30.68 | 1.11 | 11.16 |
| EL2 | 42.0 | 46.78 | 1.11 | 11.38 |
| EL3 | 51.0 | 52.47 | 1.03 | 2.88 |
| EL4 | 23.1 | 23.98 | 1.04 | 3.81 |
| EL5 | 39.0 | 41.63 | 1.07 | 6.74 |
| EL6 | 51.0 | 58.61 | 1.15 | 14.92 |
| EL7 | 4.5 | 3.68 | 0.82 | -18.22 |
| EL8 | 24.0 | 26.35 | 1.10 | 9.79 |
| EL9 | 36.0 | 35.74 | 0.99 | -0.72 |
| EL10 | NO SCOUR | | | |
| EL11 | 1.5 | 1.19 | 0.79 | -20.67 |
| EL12 | 4.3 | 3.68 | 0.86 | -14.42 |

Nevertheless, the rest of the calculated equilibrium time of scour values from Table 4.11 show a good agreement with the computer modeled ones, particularly highlighting equilibrium time of scour values from Tests EL3, EL4 and EL9, where the percent relative error is just 2.88 %, 3.81 % and -0.72 %, respectively.

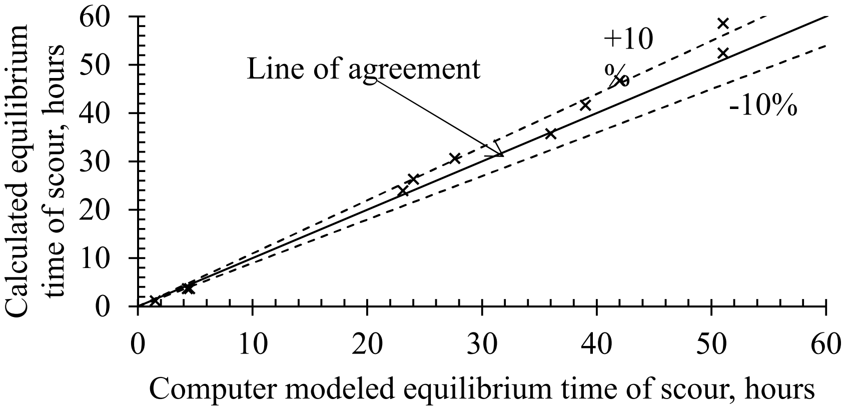



Figure 4.8. Comparison of calculated and computer modeled equilibrium times of scour
*Note*: Tests at water intakes without flow separation, sand grain size $d_2 = 0.67$ mm.

The average percent relative error calculated with Eq. (4.1) for data set of water intake tests without flow separation at the structure and sand grain size $d_2 = 0.67$ mm is 10.4 % (see Table 4.11 and Figure 4.8).

## 4.4 Comparison of time of scour values computed by different author formulas

Calculations of equilibrium time of scour, $t_e$ were made by the formulae from Chapter 1.2 using the data from experiments at water intakes with flow separation at the structure. Unfortunately, Equation (1.9) of Kothyari et al. (2007) [39] could not be used for $t_e$ calculations, since Equation (1.9) is intended for pier, not abutment scour calculations. Also Equation (1.12) of Abou-Seida et al. (2012) [1] could not be used, because the method is developed and restricted for soils containing clay.

The results of different author (discussed in Chapter 1.2) time to equilibrium scour calculation results for median sand size $d_1$ = 0.24 mm can be seen in Table 4.16. Equilibrium time of scour calculation results of authors discussed in Chapter 1.2 for median sand size $d_2$ = 0.67 mm can be found in Table 4.17.

Since the ratio of length and width of the water intake $L/b > 6$ in Melville & Chiew (1999) [62] evaluation method precondition is correct, Equation (1.1) can be used for time to equilibrium scour estimation. Melville & Chiew evaluation method (Eq. 1.1) results show very misleading and mostly negative equilibrium time of scour values (see Table 4.16). Only starting from test AL9 time to equilibrium scour values become positive.

The problem of misleading time to equilibrium scour results is in the ratio of the approach flow velocity and the critical flow velocity. Melville & Chiew (1999) [62] time to equilibrium scour evaluation method is reliable only when the ratio of the approach flow velocity to the critical flow velocity is close to 1 ($U/U_c \sim 1$), moreover it is restricted to certain preconditions ($1 \geq U/U_c \geq 0.4$), which are not fulfilled, since the approach flow velocity is way below the critical flow velocity, the ratio of these velocities is less than 0.4, which creates a negative value in Equation (1.1).

Radice et al. (2002) [75] time to equilibrium scour evaluation method (Eq. 1.3) results show values of $t_e$ that are the same for different test conditions and they are changing with the approach flow velocity and structures length. This method seems to work better, when the ratio of the approach flow velocity and the critical flow velocity is within the range of $0.99 \geq U/U_c \geq 0.9$ (very close to live-bed conditions).

Coleman et al. (2003) [13] propose several equations for time to equilibrium scour calculation, which can be used if they comply with certain preconditions, like the ratio of structures length and median sand grain size $L/D_{50} > 60$, or the ratio of the approach flow depth and structures length $h/L < 1$, or the same ratio $h/L \geq 1$. Since only 3 out of 4 Coleman et al.

(2003) [13] proposed equation preconditions can be fulfilled, Equation (1.7) cannot be used for time to equilibrium scour calculations using the data from experiments at water intakes with flow separation at the structure.

Results from Coleman et al. (2003) "a" [13] Equation (1.4) show that time values are the same for different tests, and they are changing with the approach flow velocity and structures length, neglecting the changes in flow contraction and other factors.

The same trend as in the case of Coleman et al. (2003) "a" [13] Equation (1.4) goes for the estimated time to equilibrium scour results from Coleman et al. (2003) "b" [13] Equation (1.5), where equilibrium time of scour is changing with the approach flow velocity.

Meanwhile the results from Coleman et al. (2003) "c" [13] Equation (1.6) show very high time to equilibrium scour values in all the experiments (see Table 4.16). The cause of this could be the low approach flow velocity and the large length of the structure (abutments length).

Coleman et al. (2003) [13] time to equilibrium scour evaluation methods seem to work better, when the ratio of the approach flow velocity and the critical flow velocity is within the range of $0.99 \geq U/U_c \geq 0.9$, which proves that these evaluation methods are not intended to be used when the ratio of $U/U_c$ is less than 0.9, therefore in this case with the data taken from water intake tests, where the approach flow velocity is low, most of the results are very misleading.

Grimaldi et al. (2006) [34] time to equilibrium scour evaluation method (Eq. 1.8) results are changing with the approach flow velocity and structures length, neglecting the changes in flow contraction and other factors.

Cardoso & Fael (2010) [10] time to equilibrium scour evaluation method (Eq. 1.10) results show enormous time of scour values for all the experiments (see Table 4.16), which basically means that the equilibrium can never be achieved. Taking a closer look at the Eq. (1.10), the reasons of these huge calculated time of scour values for tests AL1 to AL12 can be explained by the very fine sand used in the experiments ($d_1$ = 0.24 mm) and the large length ($L$ = 1.5 ÷ 3 m) of the structure, as well as the shallowness of the flow ($h$ = 0.07 m) in the floodplain.

Ghani et al. (2011) [20] Genetic Programming (GP) method (Eq. 1.11) results are changing with the approach flow velocity and structures length, neglecting the changes in flow contraction and other factors. Ghani et al. (2011) [20] Genetic Programming (GP) method seems to work better with higher approach flow velocities ($0.99 \geq U/U_c \geq 0.9$), which are closer to the critical flow velocity.

Table 4.12

Equilibrium time of scour evaluation method results comparison, sand grain size $d_1 = 0.24$ mm

| Author | SI | AL1 | AL2 | AL3 | AL4 | AL5 | AL6 | AL7 | AL8 | AL9 | AL10 | AL12 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Melville & Chiew (1999) | days | -1.78 | -1.78 | -1.78 | -1.78 | -0.31 | -0.31 | -0.31 | -0.31 | 0.44 | 0.44 | 0.44 |
| Radice et al. (2002) | days | 1.85 | 1.85 | 1.85 | 1.77 | 1.33 | 1.33 | 1.24 | 1.24 | 1.03 | 0.85 | 0.85 |
| Coleman et al. (2003) a | days | 0.82 | 0.82 | 0.82 | 0.82 | 1.45 | 1.45 | 1.44 | 1.44 | 2.08 | 2.07 | 2.07 |
| Coleman et al. (2003) b | days | 0.47 | 0.47 | 0.47 | 0.47 | 0.98 | 0.98 | 0.98 | 0.98 | 1.53 | 1.53 | 1.53 |
| Coleman et al. (2003) c | days | 80013 | 80013 | 80013 | 64743 | 27762 | 27762 | 20109 | 20109 | 11623 | 4911 | 4911 |
| Grimaldi et al. (2006) | days | 2.05 | 2.05 | 2.05 | 2.45 | 47.41 | 47.41 | 55.78 | 55.78 | 260.27 | 347.80 | 347.80 |
| Cardoso & Fael (2010) | days | 2978488 | 2978488 | 2978488 | 2099774 | 1583396 | 1583396 | 930117 | 930117 | 774796 | 187605 | 187605 |
| Ghani et al. (2011) | days | 0.54 | 0.54 | 0.54 | 0.54 | 0.94 | 0.94 | 0.92 | 0.92 | 1.32 | 1.27 | 1.27 |
| Presented method (2015) | days | 4.00 | 7.65 | 9.95 | 4.96 | 6.33 | 6.60 | 1.95 | 4.30 | 3.98 | 0.48 | 1.34 |

Table 4.13

Equilibrium time of scour evaluation method results comparison, sand grain size $d_2 = 0.67$ mm

| Author | SI | AL1 | AL2 | AL3 | AL4 | AL5 | AL6 | AL7 | AL8 | AL9 | AL10 | AL12 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Melville & Chiew (1999) | days | -2.73 | -2.73 | -2.73 | -2.73 | -1.26 | -1.26 | -1.26 | -1.26 | -0.51 | -0.51 | -0.51 |
| Radice et al. (2002) | days | 1.85 | 1.85 | 1.85 | 1.77 | 1.33 | 1.33 | 1.24 | 1.24 | 1.03 | 0.85 | 0.85 |
| Coleman et al. (2003) a | days | 0.38 | 0.38 | 0.38 | 0.38 | 0.67 | 0.67 | 0.67 | 0.67 | 0.96 | 0.96 | 0.96 |
| Coleman et al. (2003) b | days | 0.22 | 0.22 | 0.22 | 0.22 | 0.46 | 0.46 | 0.46 | 0.46 | 0.71 | 0.71 | 0.71 |
| Coleman et al. (2003) c | days | 80013 | 80013 | 80013 | 64743 | 27762 | 27762 | 20109 | 20109 | 11623 | 4911 | 4911 |
| Grimaldi et al. (2006) | days | 0.05 | 0.05 | 0.05 | 0.07 | 2.55 | 2.55 | 3.30 | 3.30 | 21.69 | 35.75 | 35.75 |
| Cardoso & Fael (2010) | days | 1066921 | 1066921 | 1066921 | 752158 | 567187 | 567187 | 333176 | 333176 | 277539 | 67202 | 67202 |
| Ghani et al. (2011) | days | 0.35 | 0.35 | 0.35 | 0.34 | 0.60 | 0.60 | 0.59 | 0.59 | 0.84 | 0.81 | 0.81 |
| Presented method (2015) | days | 1.80 | 2.93 | 3.51 | 1.06 | 1.80 | 2.27 | 0.24 | 0.88 | 1.40 | no scour | 0.11 |

In general, equilibrium time of scour estimation results from different author formulas, using the data from water intake tests with flow separation revealed, that mostly the estimations are reliable, when the approach flow velocity is close to the critical flow velocity; it also revealed that none of these methods take into consideration the changes in flow contraction and the local flow velocity, which is not the same as the mean approach flow velocity and it can exceed the critical flow velocity at the structure in the floodplain, even if the mean approach flow velocity in the main channel is well below the critical flow velocity.

## 4.5 Theoretical analysis of hydraulic and riverbed parameter impact on equilibrium time of scour

To analyze the equilibrium time of scour estimation method for water intakes, Eqs. (2.21), (2.25) and Eqs. (2.23), (2.25) are transformed to a form that shows clearly that they contain dimensionless parameters and characteristics of the flow and riverbed:

$$N_{equil} = \frac{t_{equil}}{4D_{equil}h_f^2} + N_0, \tag{4.2}$$

where $N_{equil} = 1/6x^6_{equil} - 1/5x^5_{equil}$;
$N_0 = 1/6x_0^6 - 1/5x_0^5 = -0.033$ – parameter to calculate scour formed during the previoustime step ($x_0 = 1$);
$x_{equil} = 1 + h_{equil}/2h_f$ – relative depth of scour;
$t_{equil}$ – equilibrium time of scour, s;
$D_{equil} = 1/2\cdot(\pi\cdot m\cdot k^4)/(A_{equil}\cdot V_l^4)$ for water intakes with flow separation (= $3/5\cdot(\pi\cdot m)/(A_{equil}\cdot V_l^4)$ for water intakes without flow separation);
$h_f$ – water depth in the floodplain, m.

Equation (4.2) is transformed in to a more detailed form, expressing the parameters for parameter $D_{equil}$ for water intakes with flow separation at the structure:

$$N_{equil} = \frac{2t_{equil}A_{equil}\varphi^4 g^2}{\pi m k^4}\frac{\Delta h^2}{h_f^2} + N_0, \tag{4.3}$$

where $A_{equil}$ – parameter calculated with Eq. (2.14);
$\varphi$ – shear stress;
$\Delta h$ – maximum backwater value at bridge crossing determined by the Rotenburg (1969) [81] formula, m;
$k$ - coefficient of changes in discharge because of scour, which depends on the flow contraction (Gjunsburgs & Neilands, 2001) [24];
$g$ – acceleration of gravity, m/s$^2$;
$m$ – steepness of the scour hole.

For water intakes without flow separation at the structure, expressing the parameters for parameter $D_{equil}$ Eq. (4.2) can be expressed as:

$$N_{equil} = \frac{5t_{equil} A_{equil} \varphi^4 g^2}{3\pi m} \frac{\Delta h^2}{h_f^2} + N_0 , \tag{4.4}$$

where $A_{equil}$ – parameter calculated with Eq. (2.15).

In general form Eq. (4.3) for water intakes with flow separation at the structure can be written as:

$$N_{equil} = \frac{2A_{equil}\varphi^4 g^2}{\pi m k^4} \frac{h^2}{h_f^2} \frac{1}{\left(\frac{d}{h_f}\right)^{0.25}} \left\{ \frac{P_k}{2}\left[\left(\frac{Q}{Q_b}\right)^2 - 1\right] + \frac{1}{2}P_{kb}\sqrt{\frac{1}{\frac{Fr}{i}}}\left[\left(\frac{Q}{Q_b}\right)^2 + 1\right] + P_{kb} \right\}^2 \cdot t_{equil} + N_0 , \tag{4.5}$$

where $Q/Q_b$ – flow contraction rate;
$P_k = V_k^2/gh$ – kinetic parameter of flow in contraction in open-flow conditions;
$V_k$ – flow velocity in contraction, m/s;
$P_{kb} = V^2/gh_f$ – kinetic parameter of the open flow in natural conditions;
$V$ – approach flow velocity, m/s;
$Fr/i$ – ratio of the Froude number to the river slope;
$d/h_f$ – dimensionless sand grain size;
$h$ – average depth of the flow in the contracted section, m.

And Eq. (4.4) for water intakes without flow separation at the structure in general form can be written as:

$$N_{equil} = \frac{5A_{equil}\varphi^4 g^2}{3\pi m} \frac{h^2}{h_f^2} \frac{1}{\left(\frac{d}{h_f}\right)^{0.25}} \left\{ \frac{P_k}{2}\left[\left(\frac{Q}{Q_b}\right)^2 - 1\right] + \frac{1}{2}P_{kb}\sqrt{\frac{1}{\frac{Fr}{i}}}\left[\left(\frac{Q}{Q_b}\right)^2 + 1\right] + P_{kb} \right\}^2 \cdot t_{equil} + N_0 . \tag{4.6}$$

From Equation (4.5) for water intakes with flow separation at the structure equilibrium time of scour $t_{equil}$ is expressed and it reads as follows:

$$t_{equil} = \frac{(N_{equil} - N_0)(\pi m k^4 h_f^2)\left(\frac{d}{h_f}\right)^{0.25}}{\left\{\frac{P_k}{2}\left[\left(\frac{Q}{Q_b}\right)^2 - 1\right] + \frac{1}{2}P_{kb}\sqrt{\frac{1}{\frac{Fr}{i}}}\left[\left(\frac{Q}{Q_b}\right)^2 + 1\right] + P_{kb}\right\}^2 \cdot 2A_{equil}\varphi^4 g^2 h^2}. \tag{4.7}$$

Also from Eq. (4.6) for water intakes without flow separation at the structure equilibrium time of scour $t_{equil}$ is expressed and it reads as follows:

$$t_{equil} = \frac{(N_{equil} - N_0)(\pi m h_f^2)\left(\frac{d}{h_f}\right)^{0.25}}{\left\{\frac{P_k}{2}\left[\left(\frac{Q}{Q_b}\right)^2 - 1\right] + \frac{1}{2}P_{kb}\sqrt{\frac{1}{\frac{Fr}{i}}}\left[\left(\frac{Q}{Q_b}\right)^2 + 1\right] + P_{kb}\right\}^2 \cdot 2A_{equil}\varphi^4 g^2 h^2}. \tag{4.8}$$

In general form, equilibrium time of scour is a function of the following parameters:

$$t_{equil} = f\left(\frac{Q}{Q_b}; P_k; P_{kb}; \frac{Fr}{i}; \frac{d}{h_f}; \frac{\beta V_0}{V_l}; \frac{h}{h_f}; \frac{h_{equil}}{h_f}\right), \tag{4.9}$$

where $\beta V_0/V_l$ – relative velocity of the flow;
$h/h_f$ – relative depth of the flow;
$\beta$ – reduction coefficient of the critical flow velocity at the bended flow determined by using the Rozovskyi (1956) [82] approach.

Theoretical analysis of the developed equilibrium time of scour calculation method (Eq. 2.25) was made and it showed that equilibrium time of scour depends on flow contraction rate, kinetic parameter of flow in contraction in open-flow conditions, kinetic parameter of the open flow, ratio of the Froude number to the river slope, dimensionless sand grain size, ratio of the recalculated critical flow velocity to the local flow velocity, relative flow depth, and relative scour depth (Eq. 4.9).

## 4.6 Graphical analysis of hydraulic and riverbed parameter impact on equilibrium time of scour

Laboratory experiment data (Gjunsburgs & Neilands, 2001) [24] and calculation results of the suggested equilibrium time of scour estimation method (Eq. 2.25) for water intakes with and without flow separation at the structure were used to show the impact of hydraulic and riverbed parameters on equilibrium time of scour.

The influence of hydraulic and riverbed parameters on equilibrium time of scour at water intakes with flow separation at the structure are shown graphically in Figures (4.9 – 4.13). The influence of hydraulic and riverbed parameters on equilibrium time of scour at water intakes without flow separation at the structure can be seen graphically in Figures (4.14 – 4.18).

The points in the graphs (Figs. 4.9 – 4.18) indicate the calculated and computer modeled equilibrium points, where equilibrium time and scour have been achieved. Test types and numbers can be found in the notes under each figure.

When the contraction rate of the flow $Q/Q_b$ increases, it creates more critical conditions for the sediments around the water intake, thus resulting in an increase of the relative depth of scour $h_{equil}/h_f$. The relative depth of scour $h_{equil}/h_f$ will always be greater with fine sand ($d_1$ = 0.24 mm), than with coarse sand ($d_2$ = 0.67 mm), since finer particles are more easily picked up and scoured away. Therefore, the more contracted the flow becomes, the greater the scour hole will get, thereby increasing also the relative depth of scour (see Fig. 4.9 and Fig. 4.14).

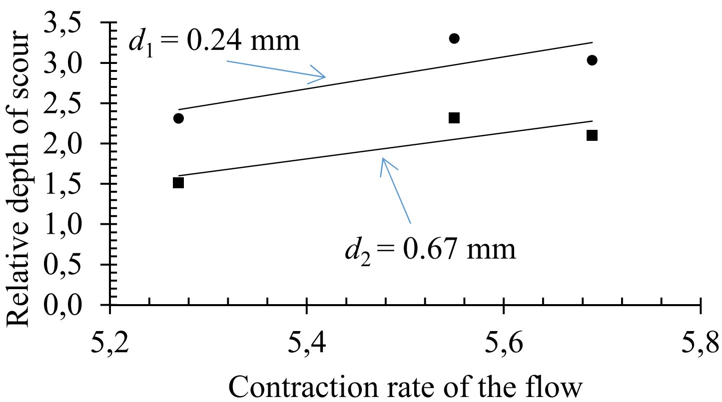



Figure 4.9. Relative depth of scour dependence from contraction rate of the flow
*Note:* Tests AL1, AL2 & AL3 with two sand grain sizes $d_1$ = 0.24 mm and $d_2$ = 0.67 mm.

Flow contraction creates a series of events that also has an impact on equilibrium time of scour, thereby if the flow contraction rate $Q/Q_b$ increases, equilibrium time of scour increases as well, consequently the greater the contraction rate of the flow $Q/Q_b$ value is, the greater the equilibrium time of scour value becomes. Since finer ($d_1$ = 0.24 mm) sand particles are more easily scoured away, it takes longer time to achieve equilibrium stage, than it is with coarse ($d_2$ = 0.67 mm) sand (see Fig. 4.10 and Fig. 4.15).

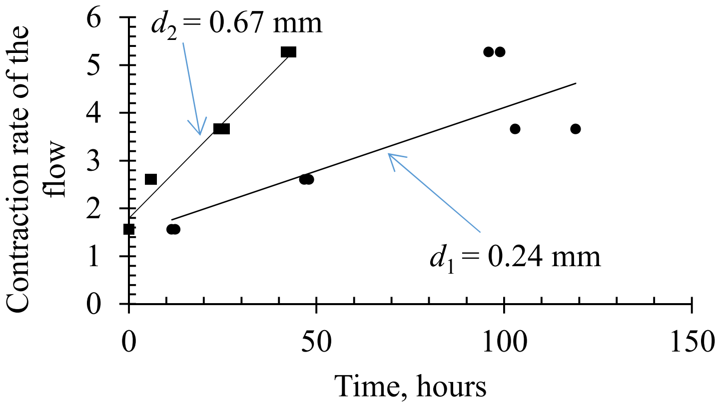



Figure 4.10. Contraction rate of the flow impact on equilibrium time of scour
*Note:* Tests AL1, AL4, AL7 & AL10 with two sand grain sizes $d_1$ = 0.24 mm and $d_2$ = 0.67 mm.

When the scouring process continues in the scour hole, it takes longer time to achieve time to equilibrium scour. In the case of coarse sand ($d_2$ = 0.67 mm) the depth of scour is achieved faster, thus resulting in a lesser relative depth of scour and at the same time lesser time to equilibrium scour, however with fine sand ($d_1$ = 0.24 mm) on the contrary, it takes more time to achieve equilibrium scour depth, since the scour depth continues to increase, consequently increasing the relative depth of scour $h_{equil}/h_f$. Relative depth of scour $h_{equil}/h_f$ is connected with equilibrium time of scour in a direct way – if the relative depth of scour $h_{equil}/h_f$ becomes greater, equilibrium time of scour increases as well (see Fig. 4.11 and Fig. 4.16).

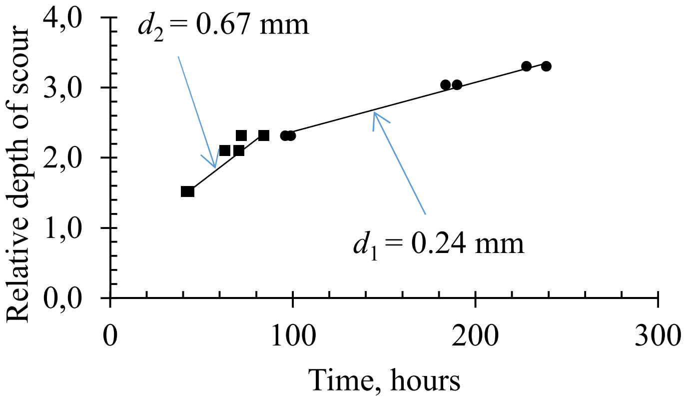



Figure 4.11. Relative depth of scour impact on equilibrium time of scour
*Note:* Tests AL1, AL2 & AL3 with two sand grain sizes $d_1$ = 0.24 mm and $d_2$ = 0.67 mm.

With an increase in Froude number *Fr*, there is also an increase in equilibrium time of scour; the greater the Froude number *Fr* value becomes, the further the scouring process continues in the scour hole, resulting also in an increased equilibrium time of scour. With fine sand ($d_1$ = 0.24 mm) the equilibrium time is greater, the scouring process takes longer to achieve the equilibrium stage; with coarse sand ($d_2$ = 0.67 mm) on the other hand, the scouring process ends more quickly, resulting in lesser equilibrium time of scour value (see Fig. 4.12 and Fig. 4.17).

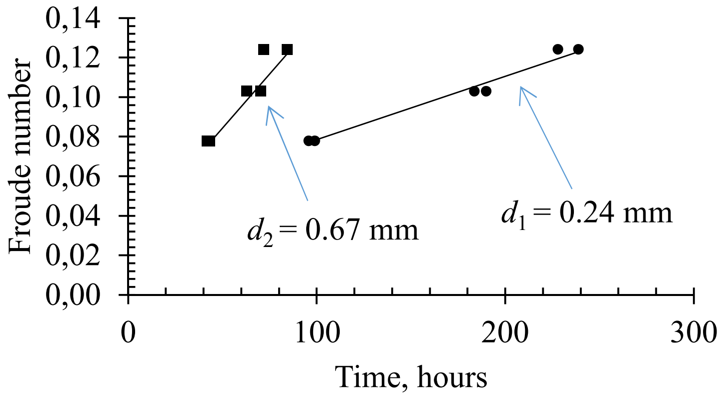



Figure 4.12. Froude number influence on equilibrium time of scour

*Note:* Tests AL1, AL2 & AL3 with two sand grain sizes $d_1 = 0.24$ mm and $d_2 = 0.67$ mm.

Approach flow velocity $V$ is one of the fundamental scouring agents. When local flow velocity $V_l$ at the water intake exceeds the critical value of sediment movement, the scouring process begins. Since coarse sand particles ($d_2 = 0.67$ mm) are heavier, it takes more energy to scour them away, so with an increase of relative velocity of the flow $V_l/\beta V_0$ the increase in equilibrium time of scour is medium, however for fine sand ($d_1 = 0.24$ mm) the increase in equilibrium time is more accelerating with an increase of relative velocity of the flow $V_l/\beta V_0$. So the greater the local flow velocity $V_l$ is and, at the same time, the smaller the recalculated critical flow velocity $\beta V_0$ for the fine sand is, the greater equilibrium time of scour will become (see Fig. 4.13 and Fig. 4.18).

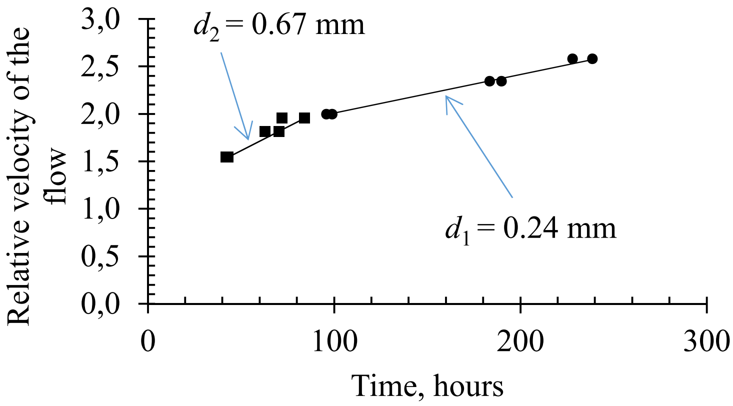



Figure 4.13. Relative velocity of the flow influence on equilibrium time of scour

*Note:* Tests AL1, AL2 & AL3 with two sand grain sizes $d_1 = 0.24$ mm and $d_2 = 0.67$ mm.

Graphical hydraulic and riverbed parameter dependence analysis of the developed equilibrium time of scour calculation method results was made and it showed that equilibrium time of scour depends on: flow contraction rate; relative depth of scour; Froude number; and relative velocity of the flow (Figs. 4.9 – 4.13 for water intake structures with flow separation at the structure, and Figs. 4.14 – 4.18 for water intake structures without flow separation at the structure).

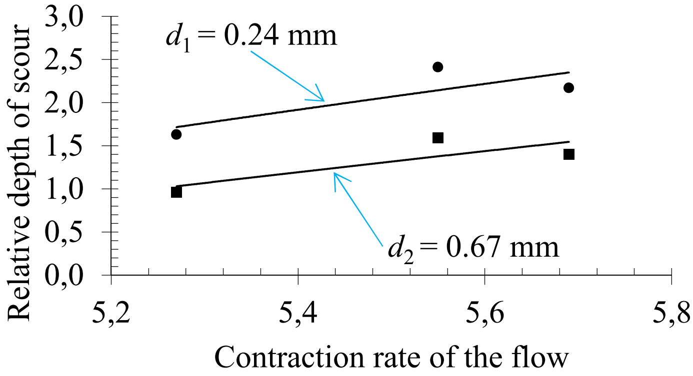



Figure 4.14. Relative depth of scour dependence from contraction rate of the flow
*Note*: Tests EL1, EL2, EL3 with two sand grain sizes $d_1$ = 0.24 mm and $d_2$ = 0.67 mm.

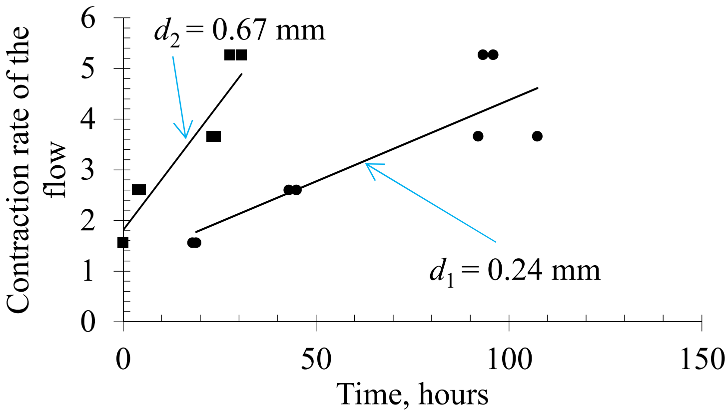



Figure 4.15. Contraction rate of the flow impact on equilibrium time of scour
*Note*: Tests EL1, EL4, EL7, EL10 with two sand grain sizes $d_1$ = 0.24 mm and $d_2$ = 0.67 mm.

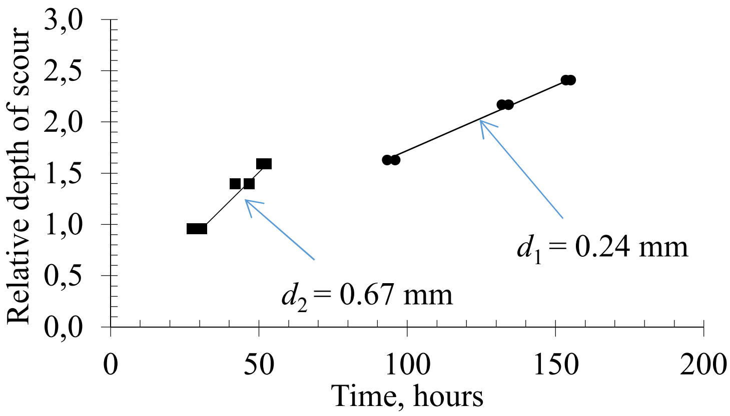



Figure 4.16. Relative depth of scour impact on equilibrium time of scour
*Note*: Tests EL1, EL2, EL3 with two sand grain sizes $d_1$ = 0.24 mm and $d_2$ = 0.67 mm.

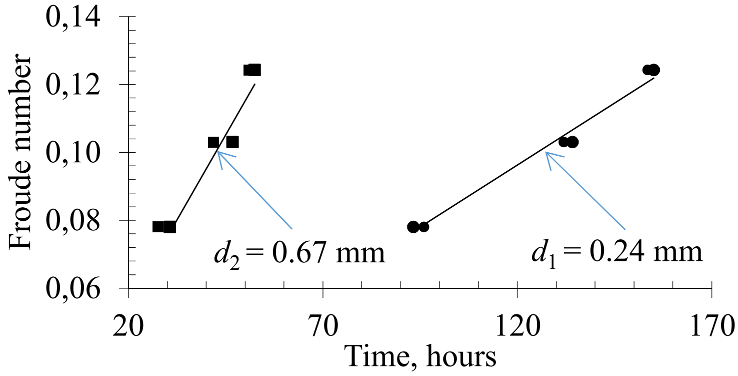



Figure 4.17. Froude number influence on equilibrium time of scour

*Note:* Tests EL1, EL2, EL3 with two sand grain sizes $d_1$ = 0.24 mm and $d_2$ = 0.67 mm.

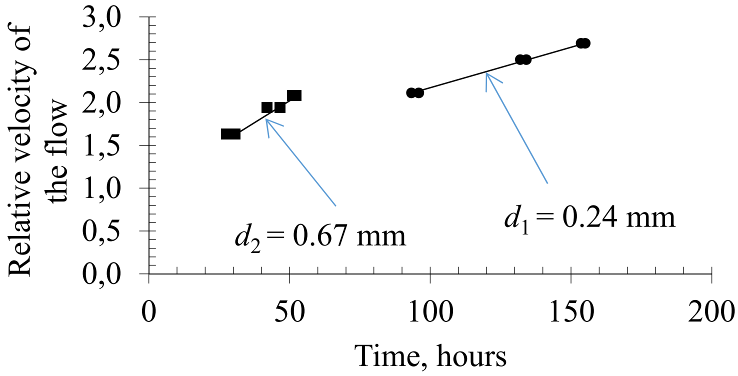



Figure 4.18. Relative velocity of the flow influence on equilibrium time of scour

*Note*: Tests EL1, EL2, EL3 with two sand grain sizes $d_1$ = 0.24 mm and $d_2$ = 0.67 mm.

# 5 LOCAL SCOUR RECOGNITION IN LATVIAN LEGISLATION

Legislation related to local scour at geotechnical structures available in Latvia is general and unspecific. Guidelines and recommendations that are found in Latvian Building Normatives (LBN) and Eurocodes are more informative and uncertain in respect of measures to be followed in cases of local scour, their evaluation and calculation. The law states that the use of Eurocodes is mandatory in all EU countries as of 2011. As Latvia is an EU member state since 2004, it is subject to follow Eurocodes.

Information related to flooding and scouring found in LBN 224-15 "Drainage systems and hydraulic structures" [52] is very modest. LBN 224-15 [52] states that when hydraulic structure calculations are made, the estimated flow rate with probability of excess (depending on the flood magnitude) should be taken into account, thus the highest water level is taken along with the average flow rate, on which any further calculations are based. In the same LBN 224-15 [52] maximum allowed approach flow velocities of the flow are defined, depending on the approach flow depth, riverbed material type and particle size. Based on these data a risk analysis is carried out for the riverbed and the appropriate measures for coast strengthening are applied. It is also important to recognize the fact that changing the natural situation at one point or section of a river, where a geotechnical structure is built, affects not only the downstream part of the river, but also the upstream part, therefore the risk analysis should include a much broader area, particularly focusing on flow changes in the downstream region, after the structure construction site.

Eurocode EN 1990 [54] establishes principles and requirements for the safety, serviceability and durability of structures, describes the basis for their design and verification and gives guidelines for related aspects of structural reliability. Eurocode EN 1990 [54] is intended to be used in conjunction with Eurocodes EN 1991 to EN 1999 for the structural design of buildings and civil engineering works, including geotechnical aspects, structural fire design, situations involving earthquakes, execution and temporary structures. Eurocode EN 1990 [54] states that engineering structures and bridge structures must last a lifetime of 100 years, in which time there should be no severe repairs, breakdowns, structural collapses, only annual maintenance works and upkeep.

Eurocode EN 1990 [54] stipulates that a construction is in a critical condition, if it is unable to perform its normal functions provided or it may lose its stability and become unfit for further use. This means that the formation of one of the critical design conditions can no longer meet the user's requirements. In structural calculations critical conditions are defined as the limit state. Structural collapse or excessive deformation limit state - construction or structural element, including substrate, pile, main wall, etc. internal damage or excessive deformation, where construction material strength is a key factor. Soil excessive deformation limit state - riverbed damage or excessive deformation, where the soil or rock strength is an important factor in ensuring resistance.

Effects on engineering structures from snow and wind force are also accounted for and their load calculation is given in Eurocodes EN 1991-1-3 [55] and EN 1991-1-4 [56], respectively.

Eurocode EN 1992-2 [57] states that engineering structures, which are affected and damaged by floating ice or other solid materials, that are found in water, must have a special defensive layer, which depending from the floating solid material force is at least 10 mm thick.

Eurocode EN 1997-1 [58], which applies to geotechnical structure designing, states that before beginning of any hydraulic engineering structure project, a geotechnical analysis is needed for the region of application. This service can be ordered both at local Latvian scientists, as well as foreign services. After a full geotechnical analysis, which includes soil parameters, layer thicknesses, stream forces, etc., the construction and strength of foundations can be selected. Geotechnical analysis takes into consideration wave induced loads, ice loads and a lot of other loads, including soil erosion, however referring to erosion, no actual erosion or scour calculation method is given, not for scour depth, nor time of scour. Furthermore, it is stated that rain and flood induced increased water discharges must also be accounted for in the project stage of geotechnical structures. The main task of geotechnical structure projects is to make sure that the structures collapse and excess deformation equilibrium state will not be reached and exceeded, considering all of the before mentioned loads and forces. A more simplified version of Eurocode EN 1997-1 [58] can be found in LBN 207-15 “Geotechnical structure designing” [51].

Typically, piles are used as the foundation construction. Pile installation depth depends from the riverbed structure, thickness and bottom layer load-bearing capabilities. Depending on the structure load the total number of piles is determined, while their length depends on the soil type and thickness of the layers; pile length may be from 4 meters up to several tens of meters in soils that are sandy or silty (unstable).

Since not all of the necessary information can be found in LBN and Eurocodes about geotechnical structure calculations, engineers complement their knowledge from literature which is available from other countries (England, Germany, USA). These may be special geotechnical structure design handbooks, guidelines or instructions with the necessary calculation examples [7 – 8], [35 – 37], [77 – 78].

Latvian geotechnical structure designer experience shows that in relation to local scour, there are no understandable and high quality materials available in Latvia, which are written in the national language. Since local scour phenomenon should not be ignored, it is necessary to take into account the maximum depth of scour in the design phase of geotechnical structures, therefore there is a need to develop a material that could be used by geotechnical structure designers.

# CONCLUSION

1. The proposed threshold criteria for equilibrium time of scour known from the literature are only depending on the size of the hydraulic structure, and not on hydraulic parameters of the flow.
2. The most common parameters used in equilibrium time of scour calculation methods are: approach flow depth; approach flow velocity; critical flow velocity; structure size parameters (abutment length and width, or pier diameter); median size of the sand; and sand density;
3. No method for equilibrium time of scour calculation at water intakes can be found, where the following parameters are being taken into consideration: contraction rate of the flow, local flow velocity near the structure, flood duration, flood sequence, flood probability, flood frequency, and bed stratification.
4. The differential equation of the bed sediment movement in clear-water conditions was used and a new equilibrium time of scour evaluation method for water intakes with and without flow separation at the structure in river floodplains was worked out (Eq. 2.25).
5. Ratio of the recalculated critical flow velocity to the local one at the head of the water intake was proposed as the hydraulic threshold criterion in equilibrium time of scour calculation, equal to $\beta V_{0t}/V_{lt} = 0.985$ (Eq. 2.28).
6. Using the new threshold criterion and following this sequence, values $h_{equil}$, $A_{equil}$, $D_{equil}$, $x_{equil}$, $N_{equil}$ and finally time to equilibrium scour $t_{equil}$ can be calculated (Chapter 2.2). An electronic time to equilibrium scour calculation model was created (see Appendix 1).
7. Using flow-altering method against scour at abutments, results in equilibrium depth of scour and time of scour reduction. (Figure 4.1 and Table 4.1).
8. As the ratio of approach flow velocity to critical flow velocity increases at live-bed scour conditions, equilibrium time of scour is reached faster with decreasing scour depth value (Figure 4.4 and Table 4.3).
9. Calculated and computer modeled water intake test data revealed that with an increase in flow contraction rate and with an increase in approach flow Froude number, equilibrium time of scour increases as well (Tables 4.4 – 4.7).
10. Calculated and computer modeled equilibrium time of scour value comparison showed good agreement, the calculated average relative errors are within 10 %.

11. Different author time of scour calculation method results revealed, that mostly the estimations are reliable, when the approach flow velocity is close to the critical flow velocity (Tables 4.12 – 4.13).
12. Theoretical analysis of the developed calculation method (Eq. 2.25) showed that equilibrium time of scour depends on: flow contraction rate; kinetic parameter of flow in contraction in open-flow conditions; kinetic parameter of the open flow; ratio of the Froude number to the river slope; dimensionless sand grain size; ratio of the recalculated critical flow velocity to the local flow velocity; relative flow depth; and relative scour depth (Eq. 4.9).
13. Graphical hydraulic and riverbed parameter dependence analysis of the developed calculation method results showed that equilibrium time of scour depends on: flow contraction rate; relative depth of scour; Froude number; and relative velocity of the flow (Figs. 4.9 – 4.18).
14. Legislation analysis related to local scour at geotechnical structures available in Latvia showed, that it is general and unspecific. There is a need to develop a material that could be used by geotechnical structure designers in Latvia.

## REFERENCES


1. Abou-Seida M.M., Elsaeed G.H., Mostafa T.M., and Elzahry E.F. Local scour at bridge abutments in cohesive soil// Journal of Hydraulic Research. – 2012. – Vol. 50(2). – pp. 171-180.
2. Alfieri L., Burek P., Feyen L., and Forzieri G. Global warming increases the frequency of river floods in Europe// Hydrology and Earth System Sciences. – Copernicus Publications. – 2015. – Vol. 19(5). – pp. 1119-1152.
3. Ballio F., and Orsi E. Time evolution of scour around bridge abutments// Journal of Water Engineering and Research. – 2000. – Vol. 2. – pp. 243-259.
4. Ballio F., and Radice A. A non-touch sensor for local scour measurements// Journal of Hydraulic Research. – 2003. – Vol. 41(1). – pp. 105-108.
5. Barredo, J.I. Major flood disasters in Europe: 1950-2005// Natural Hazards. – 2007. – Vol. 42(1). – pp. 125-148.
6. Bertoldi D.A., and Jones S.J. Time to scour experiments as an indirect measure of stream power around bridge piers// Proceedings of the International Water Resources Engineering Conference. Part 2, Memphis, TN, USA. – 1998. – Vol. 1. – pp. 264-269.
7. Bridge design handbook (Road bridge design guidelines). – Latvian Road Administration (in Latvian). – 2003. – 181 p.
8. Bridge Engineering Handbook, Second Edition: Construction and Maintenance. Wai-Fah Chen and Lian Duan (eds). – CRC Press. – 661 p.
9. Cardoso A.H., and Bettess R. Effects of time and channel geometry on scour at bridge abutments// Journal of Hydraulic Engineering. – 1999. – Vol. 125(4). – pp. 388-398.
10. Cardoso A.H., and Fael C.M.S. Time to equilibrium scour at vertical wall bridge abutments// Proceedings of the Institution of Civil Engineers: Water Management. – 2010. – Vol. 163(10). – pp. 509-513.
11. Coleman S.E., and Eling B. Sand wavelets in laminar open-channel flows// Journal of Hydraulic Research. – 2000. – Vol. 38(5). – pp. 331-338.
12. Coleman S.E., and Melville B.W. Initiation of bed forms on a flat sand bed// Journal of Hydraulic Engineering. – 1996. – Vol. 122(6). – pp. 301-310.
13. Coleman S.E., Lauchlan C.S., and Melville B.W. Clear water scour development at bridge abutments// Journal of Hydraulic Research. – 2003. – Vol. 43(4). – pp. 445-448.
14. Dey S., and Barbhuiya A.K. Time variation of scour at abutments// Journal of Hydraulic Engineering. – 2005. – Vol. 131(1). – pp. 11-23.
15. Dongol D.M.S. Local scour at bridge abutments. Research Report No. 544. – Auckland, New Zealand: School of Engineering, University of Auckland, 1994. – 409 p.
16. Ettema R. Scour at bridge piers. Research Report No. 216. – School of Engineering, University of Auckland, Auckland, New Zealand, 1980. – 527 p.
17. Ettema R., Yorozuya A., Muste M., and Nakato, T. Design estimation of abutment scour depths// Proceedings of 3rd International Conference on Scour and Erosion. - Gouda, Netherlands: Curnet. – 2006. – pp. 203-204.

18. Fael C., and Cardoso A. Evolution of scour at comparatively long vertical-wall abutments under clear-water flow conditions// River Flow 2008. – Altinakar, Kokpinar, Gogus, Tayfur, Kumcu & Yildirim (eds): Kubaba Congress Department and Travel Services. – 2008. – pp. 1593-1599.

19. Fael C.M.S., Simarro-Grande G., Martin-Vide J.P., and Cardoso A.H. Local scour at vertical-wall abutments under clear water flow conditions// Water Resources Research. – 2006. – Vol. 42(10). – 12 p.

20. Ghani A.A., Azamathullah H.M., and Mohammadpour R. Estimating time to equilibrium scour at long abutment by using genetic programming// 3rd International Conference on Managing Rivers in the 21st Century: Sustainable Solutions for Global Crisis of Flooding, Pollution and Water Scarcity: Rivers 2011. – Penang, Malaysia. – 2011. – pp. 369-374.

21. Gjunsburgs B., and Lauva O. Time of scour at elliptical guide banks// 2nd International Workshop on Hydraulic Structures: Data Validation. – Coimbra, Portugal. – 2015. – pp. 97-104.

22. Gjunsburgs B., and Lauva O. Time of Scour at Steady Flow Conditions at Structures in Plain Rivers// 7th Eastern European Young Water Professionals Conference. – Belgrade, Serbia. – 2015. – 8 p.

23. Gjunsburgs B., and Neilands R. Local velocity at bridge abutments on plain rivers// Proceedings of the International Conference on Fluvial Hydraulics: River Flow 2006. – Lisbon, Portugal. – 2006. – Vol. 2. – pp. 1649-1655.

24. Gjunsburgs B., and Neilands R. Scour development on time at the abutment of the bridge on plain rivers// Proceedings of Conference on Environmental Research, Engineering and Management. – Vilnius: Kaunas University of Technology. – 2001. – Vol. 1(15). – pp. 8-11.

25. Gjunsburgs B., Govsha J., and Lauva O. Scour at layered river bed: reason of the structures failure// The 9th International Conference Environmental Engineering. – Vilnius, Lithuania. – 2014. – 7 p.

26. Gjunsburgs B., Jaudzems, G., and Govsha E. Assessment of flood damage risk for abutments in river floodplains// River Flow 2010. – Dittrich, Koll, Aberle & Geisenhainer (eds): Bundesanstalt für Wasserbau. – 2010. – pp. 1185-1192.

27. Gjunsburgs B., Klive G., and Neilands R.j. Local scour at straight guide banks in plain rivers// Proc. of 7th International Conference on Environmental Engineering. – Vilnius: Vilnius Gediminas Technical University: Press “Technika”. – 2008. – pp. 522-529.

28. Gjunsburgs B., Lauva O., and Neilands R. Equilibrium time of scour near structures in plain rivers// The 9th International Conference Environmental Engineering. – Vilnius, Lithuania. – 2014. – 7 p.

29. Gjunsburgs B., Neilands R., and Kreslins A. Abutment scour development during floods// Proceedings of 17th Canadian Hydrotechnical Conference – Hydrotechnical Engineering: Cornerstone of a Sustainable Environment. – Edmonton, Alberta. – 2005. – pp. 593-601.

30. Gjunsburgs B., Neilands R., and Neilands R.j. Scour development at bridge abutments on plain rivers during the flood: Analysis of the method// Proceedings of 2nd International conference on Scour and Erosion. – Singapore: Stallion Press. – 2004. – pp. 199-206.

31. Gjunsburgs B., Neilands R.j., and Govsha E. Local scour at elliptical guide banks// Proceedings of 3rd International Conference on Scour and Erosion. – Gouda, Netherlands: Curnet. – 2006. – pp. 235-239.

32. Gjunsburgs B., Neilands R.j., and Govsha E. Local scour at elliptical guide banks in plain rivers// Proceedings of Conference on Fluvial Hydraulics: River Flow 2006. – London: Taylor & Francis, A.A. Balkema. – 2006. – pp. 1649-1655.

33. Gjunsburgs B., Neilands R.j., and Govsha E. Scour development and bridge abutment safety during the floods// Proceedings of Flow Simulation in Hydraulic Engineering. – Dresden: Dresden Technical University. – 2006. – pp. 157-167.

34. Grimaldi C., Gaudio R., Cardoso A.H., and Calomino F. Local scouring at bridge piers and abutments: time evolution and equilibrium// Proceedings of the International Conference on Fluvial Hydraulics: River Flow 2006. – Lisbon, Portugal. – 2006. – Vol. 2. – pp. 1657-1664.

35. Guidelines for the design of armoured slopes under open piled quay walls. Report of working group No. 22 of the permanent Technical Committee II. - PIANC – 1997. – 38 p.

36. ICE manual of bridge engineering. 2nd edition. Gerard Parke and Nigel Hewson (eds). – Thomas Telford. – 2008. – 704 p.

37. J. Rozite. Bridge Operation Handbook (in Latvian). – Riga. – 2009. 108 p.

38. Kandasamy J.K., and Melville B.W. Maximum local scour depth at bridge piers and abutments// Journal of Hydraulic Research. – 1998. – Vol. 36(2). – pp. 183-198.

39. Kothyari U.C., Hager W.H., and Oliveto G. Generalized approach for clear water scour at bridge foundation elements// Journal of hydraulic Engineering. – 2007. – Vol. 133(11). – pp. 1229-1240.

40. Lança R.M., Fael C., and Cardoso A. Assessing equilibrium clear water scour around single cylindrical piers// River Flow 2010. – Dittrich, Koll, Aberle & Geisenhainer (eds): Bundesanstalt für Wasserbau. – 2010. – pp. 1207-1213.

41. Lança R.M., Fael C.S., Maia R.J., Pêgo J.P., and Cardoso A.H. Clear-water scour at comparatively large cylindrical piers// Journal of Hydraulic Engineering. – 2013. – Vol. 139(11). – pp. 1117-1125.

42. Latishenkov A.M. Questions of artificially contracted flow. – Moscow: Gostroizdat (in Russian). – 1960. – 214 p.

43. Lauchlan C.S., Coleman S.E., and Melville B.W. Temporal scour development at bridge abutments// Proceedings of the XXIX Congress of the International Association of Hydraulics Research. – Beijing. – 2001. – pp. 738-745.

44. Laursen E.M. An analysis of Relief Bridge scour// Journal of the Hydraulics Division. – 1963. – Vol. 92(HY3). – pp. 93-118.

45. Laursen E.M. Predicting scour at bridge piers and abutments: a study to advance the methodology of assessing the vulnerability of bridges to floods. General Report No. 3. – Arizona Department of Transportation. – 1980.

46. Laursen E.M. Scour at bridge crossings// Journal of the Hydraulics Division. – 1960. – Vol. 86(HY2). – pp. 39-54.

47. Laursen E.M., and Toch A. Scour around bridge piers and abutments. Bulletin 4. – Iowa, U.S.A.: Iowa Institute of Hydraulic Research. – 1956. – 60 p.

48. Lauva O. Flow altering method against scour at water intake structures: Master thesis. – Riga, Latvia: Riga Technical University. – 2012. – 119 p.

49. Lauva O., and Gjunsburgs B. Scour duration at structures at steady flow conditions// 7th International Scientific Conference Rural development 2015: Towards the Transfer of Knowledge, Innovations and Social Progress. – Kaunas, Lithuania. – 2015. – 6 p.

50. Lauva O., Klive G., and Gjunsburgs B. Evaluation of Time of Scour at Guide Banks in Plain Rivers// 2nd International Conference Innovative Materials, Structures and Technologies. – Riga, Latvia. – 2015. – 9 p.

51. LBN 207-15 "Geotechnical design" (in Latvian) [online] / Available from: http://likumi.lv/ta/id/274440-noteikumi-par-latvijas-buvnormativu-lbn-207-15-geotehniska-projektesana/. – 2015. – 23 p.

52. LBN 224-15 "Drainage systems and hydraulic structures" (in Latvian) [online] / Available from: http://likumi.lv/ta/id/274993-noteikumi-par-latvijas-buvnormativu-lbn-224-15-melioracijas-sistemas-un-hidrotehniskas-buves/. – 2015. – 74 p.

53. Levi I.I. Dynamics of river flow. – Moscow: Gidrometeoizdat (in Russian). – 1969.

54. LVS EN 1990:2006 L. Eurocode: Basis of structural design (in Latvian). – TC Construction. – 2006. – 90 p.

55. LVS EN 1991-1-3+AC:2014 L. Eurocode 1 – Actions on structures – Part 1-3: General actions – Snow loads (in Latvian). – TC Construction. – 2014. – 45 p.

56. LVS EN 1991-1-4+A1+AC:2014 L. Eurocode 1: Actions on structures – Part 1-4: General actions – Wind actions (in Latvian). – TC Construction. – 2014. – 145 p.

57. LVS EN 1992-2:2008 L. Eurocode 2 - Design of concrete structures - Concrete bridges - Design and detailing rules (in Latvian). – TC Construction. – 2008. – 100 p.

58. LVS EN 1997-1+A1+AC:2015. 7. Eurocode 7 - Geotechnical design - Part 1: General rules (in Latvian). – TC Construction. – 2015. – 155 p.

59. Melville B.W. Bridge abutment scour in compound channels// Journal of Hydraulic Engineering. – 1995. – Vol. 121(12). – pp. 863-868.

60. Melville B.W. Local scour at bridge abutments// Journal of Hydraulic Engineering. – 1992. Vol. 118(4). – pp. 615-631.

61. Melville B.W. Pier and abutment scour: integrated approach// Journal of Hydraulic Engineering. – 1997. – Vol. 123(2). – pp. 125-136.

62. Melville B.W., and Chiew Y.M. Time scale for local scour at bridge piers// Journal of hydraulic Engineering. – 1999. – Vol. 125(1). – pp. 59-65.

63. Melville B.W., and Coleman S.E. Bridge scour. – CO, USA: Water Resources Publications, Highlands Ranch. – 2000. – 550 p.

64. Melville B.W., and Sutherland A.J. Design method for local scour at bridge piers// Journal of Hydraulic Engineering. – 1988. – Vol. 114(8). – pp. 1210-1226.

65. Mia M.F., and Nago H. Design method of time dependent local scour at circular bridge pier// Journal of hydraulic Engineering. – 2003. Vol. 129(6). – pp. 420-427.

66. Mohammadpour R., Ghani A.A., and Azamathullah H.M. Estimating time to equilibrium scour at long abutment by using genetic programming// 3rd International Conference on

Managing Rivers in the 21st Century: Sustainable Solutions for Global Crisis of Flooding, Pollution and Water Scarcity: Rivers 2011. – Penang, Malaysia. – 2011. – pp. 369-374.

67. Neilands R. The theoretical analysis of the method of scour development in time for engineering structures: promotional work. – Riga, Latvia: Riga Technical University. – 2010. – 99 p.

68. Neilands R., and Gjunsburgs B. Equilibrium scour depth at water intake structures// Construction science. – 2008. – pp. 84-93.

69. Neilands R., and Gjunsburgs B. Scour Development on Time at the Abutments of the Bridge on Plain Rivers// Environmental research, engineering and management. – 2001. – Vol. 1(15). – pp. 8-11.

70. Neilands R., Gjunsburgs B., and Jaudzems G. Wastewater Treatment Plant Hydraulic Performance Improvement// The 8th International Conference Environmental Engineering. Water Engineering, Energy for Buildings. – Vilnius, Lithuania: Vilnius Gediminas Technical University. – 2011. – Vol. 2. – pp. 555-561.

71. Neilands R.R. Equilibrium scour depth at water intakes: promotional work. – Riga, Latvia: Riga Technical University. – 2008. – 80 p.

72. Oliveto G., and Hager W.H. Further results to time-dependent local scour at bridge abutments// Journal of Hydraulic Engineering. – 2005. – Vol. 131(2). – pp. 97-105.

73. Oliveto G., and Hager W.H. Temporal evolution of clear water pier and abutment scour// Journal of Hydraulic Engineering. – 2002. – Vol. 128(9). – pp. 811-820.

74. Radice A., and Lauva O. On Flow-Altering Countermeasures for Scour at Vertical-Wall Abutment// Archives of Hydro-Engineering and Environmental Mechanics. – 2013. – Vol. 59. – pp. 137-153.

75. Radice A., Franzetti S., and Ballio F. Local scour at bridge abutments// River Flow 2002. – Bousmar & Zech (eds.): Sweets & Zeitlinger, Lisse. – 2002. – pp. 1059-1068.

76. Radice A., Teruzzi A., and Ballio F. Misure di erosione localizzata in live bed tramite un sensore ottico (Measurements of live-bed scour through an optical sensor)// Proc. 29th Convegno di Idraulica e Construzioni Idrauliche: Editoriale BIOS (in Italian). – Trento, Italy. – 2004. – Vol. 1. – pp. 1139-1145.

77. Recommendations of the Committee for Waterfront Structures, Harbours and Waterways (EAU 2004) 8th Edition; Translation of the 10th German Edition. – Ernst & Sohn. – 2004. – 660 p.

78. Recommendations of the Committee for Waterfront Structures, Harbours and Waterways (EAU 2012) 9th Edition; Translation of the 11th German Edition. – Ernst & Sohn. – 2012. – 676 p.

79. Richardson E.V., and Davis S.R. Evaluating scour at bridges. – 3rd Edition Hydraulic Engineering Circular No. 18. – Washington, DC, USA: FHWA, USDOT. – 1995. – 240 p.

80. Richardson E.V., and Davis S.R. Evaluating scour at bridges. – 4th Edition Hydraulic Engineering Circular No. 18. – Washington, DC, USA: FHWA, USDOT. – 2001. – 378 p.

81. Rotenburg I.S., and Volnov V.S. Project Examples of Bridge Crossing. – Moscow: Vishaja Shkola (in Russian). – 1969. – 284 p.

82. Rozovskyi I.L. Calculation of critical velocity at the bend of wide canals// Proceedings of the Institute of Hydrology and Hydrotechnics: Academy of Science of Ukraine (in Russian). – 1956.
83. Santos J.S., and Cardoso A.H. Time evolution of local scour at obstacles protruding from channel side walls// International Journal of Sediment Research. – 2001. – Vol. 16(4). – pp. 460-472.
84. Setia B. Equilibrium scour depth time// 3rd International Conference on Water Resources, Hydraulics & Hydrology. – University of Cambridge, UK. – 2008. – pp. 114-117.
85. Sheppard D.M., and Miller W.Jr. Live-bed Local Pier Scour Experiments// Journal of Hydraulic Engineering. – 2006. – Vol. 132(7). – pp. 635-642.
86. Sheppard D.M., Demir H., and Melville B. Scour at wide piers and long skewed piers. – National Cooperative Highway Research Program Report 682: Transportation Research Board, Washington, DC. – 2011. – 65 p.
87. Sheppard D.M., Melville B, and Demir H. Evaluation of Existing Equations for Local Scour at Bridge Piers// Journal of Hydraulic Engineering. – 2014. – Vol. 140. – pp. 14-23.
88. Sheppard D.M., Odeh M., and Glasser T. Large-scale clear-water local pier scour experiments// Journal of Hydraulic Engineering. – 2004. – Vol. 130(10). – pp. 957-963.
89. Studenitcnikov B.I. Flow erosion capacity and riverbed evaluation methods. - Moscow: Gostroizdat, (In Russian). – 1964. – 184 p.
90. Territorial Dynamics in Europe: Natural Hazards and Climate Change in European Regions. Territorial Observation No. 7. – European Union. – 2013. – 28 p.
91. Yanmaz A.M., and Altinbilek H.D. Study of time dependent local scour around bridge piers// Journal of Hydraulic Engineering. – 1991. – Vol. 117(10). – pp. 1247-1268.

# APPENDIXES

## APPENDIX 1

# Time to equilibrium scour calculation model

| Input parameters | | | |
|---|---|---|---|
| Value | Symbol | Parameter | Units |
| 0.8 | $\beta$ | Reduction coefficient of the critical flow velocity at the bended flow | - |
| 0.985 | $\beta V_{0t}/V_{lt}$ | Threshold condition | - |
| 0.00024 | $d$ | Sand grain size | m |
| 1.999 | $\gamma$ | Specific weight of sediments | $t/m^3$ |
| 9.8 | $g$ | Acceleration due to gravity | $m/s^2$ |
| 0.07 | $h_f$ | Water depth in the floodplain | m |
| 1 | $k_\alpha$ | A coefficient depending on the flow crossing angle | - |
| 1 | $k_m$ | A coefficient depending on the side-wall slope of the structure | - |
| 1.65 | $m$ | Steepness of the scour hole | - |
| 0.391 | $V_l$ | Local flow velocity at the structure | m/s |

| Output parameters | | | |
|---|---|---|---|
| Value | Symbol | Parameter | Units |
| **0.230** | $V_0$ | Critical flow velocity at the structure at plain bed | m/s |
| **0.115** | $h_{equil}$ | Equilibrium depth of scour | m |
| **0.567** | $A_{equil}$ | A parameter in the Levi (1969) formula | - |
| **234.671** | $D_{equil}$ | Constant parameter in short time interval | - |
| **1.825** | $x_{equil}$ | Relative depth of scour | - |
| **2.105** | $N_{equil}$ | Parameter | - |
| **9.83** | $t_{equil}$ | Equilibrium time of scour | days |
| **236** | $t_{equil}$ | Equilibrium time of scour | h |

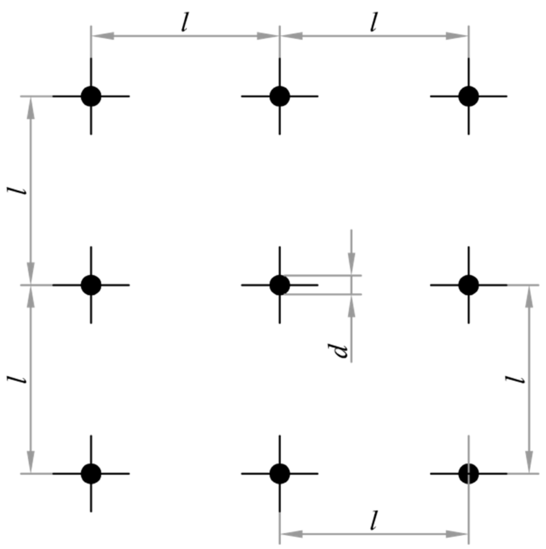
## APPENDIX 2

### Sediment discharge concept by Levi I.I. [53]

Sediment discharge is a number of particles crossing the cross section in unit time, to the weight of particles:

$$Q_S = b \cdot q_S = \gamma \frac{N \cdot V}{t}, \tag{A2.1}$$

where $q_S$ – sediment discharge in the unit width (in weight units), m$^3$/s;
$N$ – number of particles passing the cross section on width $b$ in time $t$;
$V$ – volume of the particles, m$^3$;
$b$ – section width, m.




It is proposed that the particles are moving with velocity $U_S$, with the distance $l$ between them. On the width $b$ there will be $n_1 = \frac{b}{l}$ particles.

The number of particles, which cross the cross section in $t$ (s), is determined as a ratio between the distance which they are passing in time $t$ and the distance between two next following particles:

$$n_2 = \frac{U_s \cdot t}{l}, \tag{A2.2}$$

where $U_S$ – sediment particle velocity, m/s;
$t$ – time, s;
$l$ – distance between sediment particles, m.

The common number of particles on width $b$ is equal:

$$N = n_1 \cdot n_2 = \frac{b \cdot U_s \cdot t}{l^2}. \qquad \text{(A2.3)}$$

Then the sediment discharge per unit width is:

$$\frac{1}{\gamma} q_s = \frac{N \cdot V}{b \cdot t} = \frac{U_s \cdot V}{l^2} = U_s \cdot d \frac{V}{l^2 \cdot d} = U_s dm, \qquad \text{(A2.4)}$$

where $\frac{V}{l^2 d}$ – the ratio of of the one particle volume to the entire layer of the particles with diameter $d$ on area $l^2$ - or dynamic coefficient of continuity $m$

$$m = \frac{V}{l^2 \cdot d} = \frac{\alpha \cdot d^2}{l^2} = f(U_s), \qquad \text{(A2.5)}$$

where α – coefficient depending on the shape of sediment particle;
$d$ – diameter of sediment particle, m.

Velocity of the sediment particles:

$$U_s = f(U - U_0), \qquad \text{(A2.6)}$$

where $U$ – approach flow velocity, m/s;
$U_0$ – critical flow velocity, m/s.

Coefficient of the continuity:

$$m = m_1 \left( \frac{U}{\sqrt{gd}} \right)^3, \qquad \text{(A2.7)}$$

where $m_1 = f\left(\frac{d}{h}\right)$,
$h$ – depth of the flow, m.

Then the sediment discharge:

$$q_s = \gamma \cdot U_s \cdot m \cdot d = f(U - U_0) \cdot \gamma \cdot m_1 \left( \frac{U}{\sqrt{gd}} \right)^3 \cdot d = f\left( \frac{U}{\sqrt{gd}} \right)^3 d(U - U_0) \cdot f\left(\frac{d}{h}\right) \cdot \gamma, \qquad \text{(A2.8)}$$

$$\mu_c = \frac{q_s}{q} = C_0 \left( \frac{U}{\sqrt{gd}} \right)^3 \frac{d}{h} \left( 1 - \frac{U_0}{U} \right) f\left(\frac{d}{h}\right), \qquad \text{(A2.9)}$$

where $\mu_c$ – relative contents of sediments in flow;
$q$ – relative flow discharge, m$^3$/s.

If we take into account that:

$$U_0 = a\sqrt{gd}\, f_1\left(\frac{h}{d}\right), \tag{A2.10}$$

where $a$ – coefficient 1.15;

and present $f_1\left(\frac{h}{d}\right)$ in exponential form - $\beta\left(\frac{d}{h}\right)^n$, then we have:

$$\mu_c = C\left(\frac{U}{\sqrt{gd}}\right)^3 \left(1 - \frac{U_0}{U}\right)\left(\frac{d}{h}\right)^K, \tag{A2.11}$$

where $K = 1 + n_1 = 1.25$ .

According to the test results the figure $\mu_C\% = f\left(\frac{U}{\sqrt{gd}}\right)$ with different $\frac{d}{h}$ is presented.

The results of the data processing allow us to form an equation for $Q_S$.

Sediment discharge in weight units is presented:

$$Q_s = 0.002\left(\frac{U}{\sqrt{gd}}\right)^3 d(U - U_0)\cdot\left(\frac{d}{h}\right)^{0.25}, \ [T/\mathrm{sec}]. \tag{A2.12}$$

<u>In volumetric units:</u>

$$Q_s = \frac{5.62}{\gamma}\left(1 - \frac{U}{U_0}\right)\frac{1}{d^{0.25}\cdot h^{0.25}}\cdot B\cdot U^4, \ [m^3/day]. \tag{A2.13}$$

The Eq.(13) is valid for ratio:

$$\frac{d}{h} > \frac{1}{500},$$

or can be valid till viscosity does not affect the flow motion:

$$\frac{d}{h} > \frac{1}{5000}.$$

## APPENDIX 3

## Critical velocity concept by Studenitcnikov B.I. [89]

The grain particles threshold stability in a river bed at clear water, steady uniform flow can be determined by using critical velocity $V_0$, which depends on the following parameters:

$\gamma$ – specific weight of water, t/m$^3$;

$\gamma_1$ – specific weight of particle, t/m$^3$;

$d$ – size of particle, m;

$h$ – depth of flow, m.

A stable bed is a rectangular channel, formed in a considerable width of flow zone ($B/h \geq 2.5$-3), with normal turbulence and velocities distribution in depth.

1. Lifting force acting on the particle:

$$F_y = K_1 \gamma \frac{\alpha V_0^2}{2g} \left(\frac{h}{d}\right)^{2n} \frac{\pi d^2}{4}, \tag{A3.1}$$

where $K_1$ – coefficient.

2. Weight of the particle in water:

$$G = (\gamma_1 - \gamma) \frac{\pi d^3}{6}. \tag{A3.2}$$

Stability of the particle will be when:

$$K_1 \cdot \gamma \cdot \frac{\alpha V_0^2}{2g} \cdot \left(\frac{d}{h}\right)^{2n} \frac{\pi d^2}{4} = (\gamma_1 - \gamma) \frac{\pi d^3}{6}. \tag{A3.3}$$

Then the critical velocity is equal to:

$$V_0 = \frac{A_1}{\alpha} \sqrt{\frac{\gamma_1 - \gamma}{\gamma}} \sqrt{g} \cdot h^n d^{0.5-n}, \tag{A3.4}$$

where $A_1$ – coefficient;
$\alpha$ – constant = 1.1;

or:

$$V_0 = A\sqrt{g} \sqrt{\frac{\gamma_1 - \gamma}{\gamma}} h^n d^{0.5-n}, \tag{A3.5}$$

where $A = f(\alpha)$.

Critical velocity depends on $d$ and $h$, and on the relative size of the particles $k = \frac{d}{h}$ or $d = k \cdot h$. Then:

$$V_0 = A\sqrt{g}\sqrt{\frac{\gamma_1 - \gamma}{\gamma}}k^{0.5-2n}\left(h^{0.5-n}d^{n}\right). \tag{A3.6}$$

It is necessary to find $n$ value, at which coefficient $A\sqrt{g}\sqrt{\frac{\gamma_1 - \gamma}{\gamma}}k^{0.5-2n}$ will have constant value at any relative grain size of the particles:

$$B = A\sqrt{g}\sqrt{\frac{\gamma_1 - \gamma}{\gamma}}k^{0.5-2n} = f(k) = const. \tag{A3.7}$$

Values $A$, $\gamma_1$, $\gamma$ do not depend on the relative grain size and at any $k$ (1/5, 1/10, 1/100 and so on) should reflect one condition $0.5 - 2n = 0$, then $k^{0.5-2n} = 0$ and $B = const$.

Solving equation $0.5\text{-}2n = 0$ we have $n = 0.25$ with: $\gamma_1 = const.$

$$B = A\sqrt{g}\sqrt{\frac{\gamma_1 - \gamma}{\gamma}}k^{0.5-2n} = \frac{V_0}{h^{0.25} \cdot d^{0.25}} = const. \tag{A3.8}$$

Processing of the test results and natural experimental data in a wide range of relative depth of flow $\frac{h}{d}$ (or relative size of the particles $\frac{d}{h}$) with value of the $B$ accepted as constant, the value $B$ is equal to $B = 3.6$ at $\gamma_1 = 2.65$ and $\gamma = 1$, and then $A$ is equal to 0.9

$$V_0 = 0.9\sqrt{\frac{\gamma_1 - \gamma}{\lambda}}\sqrt{g}(hd)^{0.25}. \tag{A3.9}$$

For constant specific weights of particles $\gamma_1 = 2.65$

$$V_0 = 1.15\sqrt{g}(hd)^{0.25}, \tag{A3.10}$$

or

$$V_0 = 3.6d^{0.25}h^{0.25}. \tag{A3.11}$$