

DOKUZ EYLÜL UNIVERSITY
GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

**EXPERIMENTAL AND NUMERICAL
INVESTIGATION OF THE LOCAL SCOUR
AROUND BRIDGE PIERS IN THE CASE OF THE
SEDIMENT FEEDING**

by
Fırat GÜMGÜM

February, 2019
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INVESTIGATION OF THE LOCAL SCOUR
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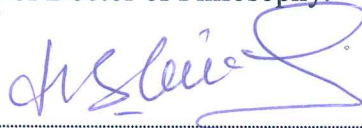
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Graduate School of Natural and Applied Sciences of Dokuz Eylül University
In Partial Fulfillment of the Requirements for the Degree of Doctor of
Philosophy in Civil Engineering, Hydraulics Hydrology Water Resources
Program**

**by
Fırat GÜMGÜM**

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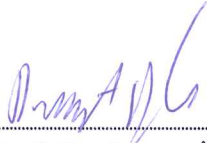
Ph.D. THESIS EXAMINATION RESULT FORM

We have read the thesis entitled “**EXPERIMENTAL AND NUMERICAL INVESTIGATION OF THE LOCAL SCOUR AROUND BRIGDE PIERS IN THE CASE OF THE SEDIMENT FEEDING**” completed by **FIRAT GÜMGÜM** under supervision of **PROF. DR. M. ŞÜKRÜ GÜNEY** and **PROF.DR. Z. FUAT TOPRAK** and we certify that in our opinion it is fully adequate, in scope and in quality, as a thesis for the degree of Doctor of Philosophy.



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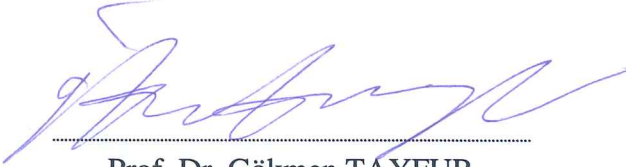
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EXPERIMENTAL AND NUMERICAL INVESTIGATION OF THE LOCAL SCOUR AROUND BRIDGE PIERS IN THE CASE OF THE SEDIMENT FEEDING

ABSTRACT

Although local scour around bridge piers were studied numerous times, researches mostly tended to investigate clear water scour. Live bed scour was investigated in few number of studies and unsteady flow conditions were not examined in detail. The aim of this study is to investigate live bed scour under flood hydrographs.

During the experiments, 16 different triangular hydrographs were generated. Circular piers with three different diameters were tested under these hydrographs. Uniform bed material of 1.63 mm diameter was used during the experiments. Experimental observations about mechanism of scour were presented by means of images obtained from the experiments. The effects of the peak flow rates and duration of hydrographs on scouring were discussed.

Numerical analysis was performed by using a computational fluid dynamics (CFD) software called Flow-3D. One of the hydrographs was selected to realize numerical study by using LES, RNG and two-equations $k-\varepsilon$ turbulence models. Numerical results were compared with experimental findings. Simulation results of LES turbulence model was more compatible with experimental results.

It was observed that the dominant element of the scour was flow rate during the rising limb and sediment transport rate during the falling limb. Three empirical equations were derived to predict time scale of maximum scour depth, temporal scour depth and maximum scour depth. Statistically significant determination coefficient values are obtained.

Keywords: Live bed scour, pier, unsteady flow, Flow3D

BESLEMELİ KATI MADDE HAREKETİ DURUMUNDA KÖPRÜ AYAKLARI ETRAFINDA OLUŞAN YEREL OYULMALARIN DENEYSEL VE SAYISAL ARAŞTIRILMASI

ÖZ

Köprü ayakları etrafında meydana gelen yerel oyulmalar bir çok çalışmaya konu olmuş olsa da araştırmacılar genellikle temiz su oyulmalarına yönelmişlerdir. Hareketli taban koşulları altında meydana gelen oyulmalara ait çalışmalar az sayıda olup, bu çalışmalarda taşkın hidrografi durumunda oluşan oyulmalar ayrıntılı olarak incelenmemiştir. Bu çalışmanın amacı, taşkın hidrografları durumunda dairesel köprü ayakları etrafında oluşan hareketli taban oyulmasını araştırmaktır.

Deneylerde 16 farklı üçgen hidrograf oluşturulmuştur. Üç farklı çapa sahip köprü ayakları, bu hidrograflar altında test edilmiştir. Deneylerde 1,63 mm çapına haiz üniform taban malzemesi kullanılmıştır. Oyulmanın mekaniğiyle ilgili gözlemler, deneyler sırasında elde edilen fotoğraflar ile anlatılmıştır. Hidrograf pik debilerinin ve hidrograf sürelerinin köprü ayakları etrafında oluşan oyulmalara etkisi, ayrıntılı olarak tartışılmıştır.

Nümerik araştırma Flow-3D adlı bir hesaplamalı akışkanlar dinamiği (CFD) yazılımı kullanılarak gerçekleştirilmiştir. Oluşturulan hidrograflardan bir tanesi seçilerek ve LES, RNG ve iki denklem $k-\epsilon$ türbülans modelleri kullanılarak nümerik analiz gerçekleştirilmiştir. Nümerik sonuçlar deneysel bulgular ile karşılaştırılmıştır. LES türbülans modelinin kullanıldığı simülasyonların deneysel sonuçlar ile daha uyumlu olduğu gözlenmiştir.

Hidrografın yükselme eğrisi boyunca akış hızının, alçalma eğrisi boyunca ise sediment taşınım miktarının oyulma üzerinde baskın unsur olduğu görülmüştür. Maksimum oyulma derinliğine ait zaman ölçeğini, zamana bağlı oyulma derinliğini ve maksimum oyulma derinliğini tahmin etmek amacıyla üç ayrı ampirik bağıntı türetilmiştir. İstatistiksel bakımdan anlamlı determinasyon katsayıları elde edilmiştir.

Anahtar Kelimeler: Hareketli taban oyulması, orta ayak, zamanla deęiřen akım,
Flow3D



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CHAPTER ONE

INTRODUCTION

1.1 Aim of the Study

Local scour around the bridge piers was registered as the most important reason for the bridge failures. Between 1989 and 2000 in the United States, 48% of more than 500 bridge failures were caused by flood and scour, and 62% of this value is directly related with floods. (Wardhana & Hadipriono, 2003).

Although there are many studies about local scour around the bridge piers, it has been studied under clear water conditions mostly. In a few studies conducted under live bed conditions, steady flow conditions or stepwise hydrographs were generally used. However, in the case of floods encountered in nature flow rate constantly changes. Furthermore, Kothyari et al. (1992a) stated that the peak flow may not be effective long enough to develop its full potential, so that the scour depth estimated by relations of steady flow conditions can be overestimated.

In this study, it was purposed to investigate scouring behavior around a circular bridge pier in live bed conditions under flood waves. For this purpose, 16 different triangular hydrographs having different maximum discharges and durations were formed and tested on uniform sediment. Effects of the flood duration by regarding both its rising and falling periods and effects of the flow velocity on scour depth, scour hole shape and sediment movement were investigated elaborately. With piers having 3 different diameters, effects of the pier diameter on scour depth and scour hole shape were discussed.

According to datasets obtained from experimental results, three different empirical equations were proposed in order to predict temporal scour depth, maximum scour depth and time scale of maximum scour depth. The experimental time dependent scour depth values were also used to test the empirical relation proposed by Oliveto & Hager (2002). The experimental maximum scour depth values were compared to the results

obtained from existing formulas of live bed scour. Besides, the related prototype values were determined by using the Froude similarity law assuming that the used piers correspond to the 1/10 and 1/20 scaled physical models.

One of the generated hydrographs was selected to perform numerical investigation by using a computational fluid dynamics (CFD) software called Flow-3D. LES, RNG and two equation k- ϵ turbulence models were used during numerical study and their results of temporal scour depths were compared with experimental results.

1.2 Scope of the Thesis

In the first chapter, brief information is given about aim of the study and scope of the thesis.

In the second chapter, previous studies available in literature about live bed scour are presented. Studies about clear water scour but under unsteady flow are also presented in this section. Several formulas present in literature are given to predict equilibrium scour depth.

In the third chapter, theoretical information about scour mechanism and scour parameters are explained. Simulation parameters and principles of Flow-3D software are also given in this chapter.

In the fourth chapter, the experimental setup and method are explained in detail. Hydrographs tested during the experiments are presented as both figures and table. Amount of collected and fed sediments are compared.

In chapter five, experimental results are analyzed elaborately. Experimental findings are presented and influence of the parameters affecting scour depth such as flood duration, flow velocity, pier diameter are discussed. Three empirical equations predicting time scale of maximum scour depth, temporal scour depth and time scale of maximum scour depth are given in this chapter.

The results of the numerical models simulated by Flow-3D are presented in the sixth chapter. Effects of various turbulence models on scour depth are also discussed and numerical results are compared with experimental findings.

In the seventh chapter, general conclusions and suggestion for further studies are presented.

Time dependent scour depths recorded by each transducer for each pier and different hydrographs are given in Appendix 1.

Appendix 2 involves the scour hole profiles determined by using laser meter.

CHAPTER TWO

PREVIOUS STUDIES

2.1 Previous Studies in Live Bed Scour

The studies on local scour in numerous scientific inquiries have been elaborated gradually, and the scouring have been further elucidated, by means of different scientific methods (regression analysis, artificial neural networks, fuzzy logic, so forth.) so as to obtain a prediction of scour hole geometry and scour depth. The most coddled part of the local scour, which has been the subject of many studies from past to present, is undoubtedly the clear water scour. The studies performed in live bed conditions are relatively scarce and flood conditions are not examined in detail. In order to determine the purpose and importance of the present study, it is essential to elaborate on the previous studies on live bed scour.

Laursen & Toch (1956), as one of the pioneers of the subject, took up field measurements and conducted various laboratory experiments under steady flow. They performed the first group of their experiences in two collateral flumes with 35 feet length and 5 feet width, and they used piers and abutments in different shapes. Flow depth was adjusted with a tailgate. Single uniform sediment was used with a 0.58 mm median grain size. They placed layers of colored sediment around the pier with a thickness of 0.01 foot in every 0.1 foot to observe the development of scour hole.

With the goal of studying flow and sediment characteristics, they built a similar flume with same size but with a sediment trap at the end of the flume and a sediment feeding system at the inlet (elevator hopper and conveyor). The flow depth was adjusted with a tailgate from 0.2 to 0.9 foot and flow velocity from 1.00 to 2.50 feet per second. They used four different uniform sediments with $d_{50} = 0.46$ to 2.3 mm media grain size. Since they could weigh the trapped bed load, they fed the flume with the same amount of sediment. It was also stated that although sediment trap successfully caught the sediment, small amount of sediment could be missed at very

high rates of transport. Equilibrium of transport was defined as a constant line between cumulative weight of trapped sand and time.

After running 8 to 50 hours duration of experiments, they found that scour depth reached the maximum rapidly and fluctuated with incoming dune. Equilibrium scour depth was defined as a constant line that scour depth fluctuated around.

Besides, they used various abutments and a modelled pier of a bridge prototype in Iowa with two different scales in their experiments and compared the relationship with field measurements. Generally, they studied both clear water and live bed conditions but in subcritical range ($F_r < 1$), different angle of attacks, single and multiple piers with different shapes, different flow depth, velocities and uniform sediment sizes.

Various scour hole patterns were provided to unravel the development of scour hole. The spiral roller was defined as active agent of scour that occurs at the base of the pier and decreases with the development of scour hole. It's stated that the angular velocity of the roller was mainly affected by mean velocity and little affected by flow depth. They found that the maximum scour depth occurred in front of the pier. Bed form was ripple first but it turned to dunes over time.

Various pier alignment factors were given for different angle of attacks, depending on pier width and length ratio. Pier shape factors were also given for different shapes of piers; circular, rectangular, elliptic, lenticular. Several protection methods were discussed such as rip-rap and collar.

Breusers et al. (1977) reviewed the literature about local scour around bridge piers both for laboratory model and field data, compared empirical results with theoretical considerations and discussed protection against scour. It was stated that horseshoe vortices were arisen from a blunt-nosed pier presence which causes high pressure on its nose so the roller was formed. All other piers were referred as sharp-nosed pier and no high pressure occurred on the nose so no vorticity. It was also stated that stream already had its own vorticity and a blunt-nosed pier only concentrated it.

Jain & Fischer (1979) investigated both clear water and live bed scour around bridge piers experimentally with the aim of observing the effects of flows with high Froude numbers on scour depth and comparing with the equations on literature generated at low Froude numbers. They also developed an equation to predict the scour depth at high Froude numbers.

They conducted their experiments in a sediment recirculating flume with a length of 27 m, width of 91 cm and depth of 41 cm. Three different uniform bed materials were used with median grain diameters of 0.25 mm, 1.50 mm and 2.50 mm, with a 20 cm thickness in flume. Two piers with diameters of 50.8 mm and 101.6 mm were used simultaneously in each run and placed at 11th and 20th meters of the flume respectively. By referring Rouse (1959), they suggested that second pier at downstream would not affect from wake turbulences (less than 1%) of first pier as long as it is at least 170 b far away from first pier, where b is width of first pier. With 160 L/s maximum discharge, flow depth was 10.16 cm except for depth tests. For depth tests, flume width was reduced to 61 cm and another pump was mounted to system, discharge and flow depth were increased to 185 L/s and 23 cm respectively and only small pier was used. Different bed slopes and Froude numbers were used ranging between 0.001-0.016 and 0.5-1.5 respectively.

As a result of their experiments, first the scour depth slightly decreased then increased with the increasing Froude number (so with the velocity). F_{rc} being the Froude number corresponding to the critical velocity, the slight decrease occurred in $F_r - F_{rc} < 0.15$ interval which was explained as the bed material removed from scour hole was less than transported into the scour hole. They observed no decrease on scour depth if $F_r - F_{rc} > 0.15$. Moreover, they found that higher Froude numbers caused larger live bed scour depths than the maximum clear water scour. It was also stated that bed-form scour mustn't be neglected at higher velocities.

Chiew (1984) aimed to investigate live-bed scour around bridge piers and effect of sediment properties. Most of his experiments were conducted in a sediment

recirculating flume with a length of 11.8 m, width of 44 cm and depth of 38 cm, under live bed and steady flow conditions. He also conducted some clear water experiments in a different flume.

In live bed experiments, 7 different uniform sediments were used with d_{50} ranging between 0.24 and 8.2 mm and 4 different non-uniform sediments with d_{50} and d_{84} ranging between 0.6-1.45 mm and 1.2-6.2 mm respectively. Sediment was collected by means of a settling tank at the end of the flume into a steep-sided hopper over the sediment recirculation pump intake. Water depth was controlled by an adjustable weir and this also worked to prevent sediment from entering the main pump. A maximum pier width was chosen as 45 mm in order to prevent wall effects, by referring Shen et al. (1966) where it was stated that ratio of flume and pier width being less than 8 causes wall effect. Bed was not leveled before the live bed experiments and it was stated as unnecessary.

Experiments were continued until an equilibrium scour depth was obtained, ranging from 4 to 20 hours. Scour depths were recorded by 1-minute intervals of total duration of 100 minutes and sediment transport rates were measured at this point. Although he conducted his first group experiments with a duration of 16 hours, he found no measurable difference on scour depth after 8-10 hours.

As a result of his live bed experiments, he found that scour depth decreased after threshold peak to a local minimum and then increased again but at a decreasing rate and reached a second peak, being consistent with Jain and Fischer (1979)'s results. He added that scour depth slightly decreased after second peak for higher velocities. He attributed this situation to the relationship between the magnitude of velocity and bed forms which directly affect the fluctuations on scour depth. The velocity that provided the local minimum also caused largest dunes and maximum fluctuations on scour depth. He explained the effect of large dunes as "When the dune height is large and the wavelength is short, the downflow and the horseshoe vortex have relatively little time to remove sediment from the scour hole before the next dune arrives and dumps sediment into the scour hole (p.94)." The velocities higher than providing local

minimum caused reduction on dune height and increment on dune wavelength. He added “As a consequence, there is more time to excavate the sediment dumped before the next dune arrives at the scour hole. Hence, the scour depth increases again (p.95).” In addition, he related the second peak (live bed maximum) with the advancement of transition flat bed. The second peak was found lower than threshold peak for dune sediments and higher than threshold peak for ripple sediments. It was also stated that antidunes began to form at a higher velocity that causes second peak and maximum scour depth began to decrease slightly.

At this point, he brought a clarification to the properties and the mechanics of ripple and dune forming sediments. It was stated that ripples occur if the median grain size of bed material is lower than 0.5-0.7 mm and only the bed material with 0.24 mm median grain size formed ripple in this study. He explained the scouring mechanism of a ripple bed as slowly developing scour hole compared to flat bed, once reaches equilibrium state no significant size and shape change occurs. He also emphasized a special case “When the experiment is started from an initially flat bed at relatively low velocity excess (which is associated with the formation of ripples) the maximum scour depth is reached rapidly (usually in less than 20 minutes) before ripples become fully developed (p.105).”

He explained scouring mechanism on dune bed as “On a dune-covered bed at relatively higher velocity excess, the scour hole in an initially flat bed develops quickly but bed features rapidly appear and the sediment transport rate is large enough to affect the initial peak of the scour depth. The scour depth fluctuations are apparent before the equilibrium scour depth is established (p.106).” Transition flatbed dunes and antidunes likewise explained elaborately.

Melville (1984) conducted his experiments under live bed conditions in a tilting flume with a length of 14.8 m, width of 0.6 m and depth of 0.4 m. Sediment was fed into the flow by means of a hopper and a conveyor at the inlet. Four uniform sediments were used with median grain diameters ranging from 0.24 to 1.40 mm. Three circular

piers were used with widths of 50.8, 79.5 and 101.6 mm. Flow depth was fixed at 10 cm and flow velocities were varied each experiment.

He explained the development of bed formations according to sediment and velocity, their effects on scour hole and general development of scour hole. His findings were compatible with Chiew (1984)'s results.

Kothyari et al. (1992a) conducted their live bed experiments in a flume with a length of 30 m, width of 1.0 m and depth of 0.60 m. A circular pier with a diameter of 65 mm was mounted in a working section with a length of 2.0 m, width of 1.0 m and depth of 1.2 m. Uniform sediment used with a median grain diameter of 0.71 mm. Sediment was fed into the flow manually at 3rd meter of the flume. Most of their experiments were carried out under steady flow and two stepwise hydrographs were tested. Results were also compared with previous studies.

They proposed a scheme to predict equilibrium scour depth in steady flow with a reasonable accurate for both clear water and live bed scour. Their scheme was based on temporal development of the cross-sectional areas of the primary vortex and horseshoe vortex and determination of particle movement in a stochastic way. According to their results and comparison with previous results, their scheme slightly underpredicted the scour depth and was insensible to fluctuations.

A model was adopted to approach unsteady flows for live bed conditions that considering each segment of stepwise hydrograph as steady flow. They found a recession on scour depth during the falling limb of hydrograph.

Melville (1997) discussed the effects of velocity, flow depth, pier width, shape and alignment, sediment size, channel geometry and their relations with each other and proposed an equation to predict equilibrium scour depth for both clear water and live bed conditions.

Sheppard & Miller (2006) conducted their live bed and a few clear water experiments in a flume with a length of 45 m, width of 1.5 m and depth of 1.2 under steady flow conditions and observed the development of scour depth and bed forms. They used a sediment pump for trapped bed load and two other water pumps with a total $1.2 \text{ m}^3/\text{s}$ capacity for both water and suspended sediment. Two different uniform sediments were used with median grain diameters of 0.27 and 0.84 mm and a single pier with a diameter of 0.152 m. Their live bed experiments took from 20 minutes to 51 hours with velocities ranging from 0.55 to 2.16 m/s and flow depths from 0.2 to 0.43 m.

It was stated that one of the main purposes of this study was gathering data for evaluating Sheppard's equation for live bed conditions. They made minor adjustments on Sheppard's equation set and compared the experimental results with the adjusted equation.

Jia et al. (2017) performed their three dimensional simulations with a computational fluid dynamics (CFD) software called CCHE3D. They simulated clear water scour under steady flow for uniform sediment and live bed scour under unsteady flow conditions for non-uniform sediment. Both circular and rectangular shaped piers were used during the simulations. They compared the numerical results with their experimental findings. It was stated that results of the simulations were compatible with experimental results for scour depth, shape of scour hole and development of scour depth. But slightly larger size of scour hole and little less slope of scour hole was observed.

2.2 Previous Studies under Unsteady Flow

Melville (1975) investigated local scour around circular bridge piers experimentally, tested previous equations given in literature and compared with field measurements. His experiments were performed in a flume with length of 19 m, width of 45.6 cm and depth of 44 cm. Discharge was controlled with a butterfly valve and flow depth was controlled with a sluice gate at downstream. Experiments in steady

flow conditions have been performed with a discharge of 17.1 L/s and flow depth of 15 cm. For unsteady flow conditions, same discharge and flow depth values were reached from 8.5 L/s and 12 cm, at 120 minutes of experiment by 20 minutes time intervals. A cylindrical pier with a diameter of 5.08 cm and a bed material of 0.385 mm median grain size were used.

According to his observations, wake region was formed initially by separated flow filaments as a result of contact of flow to the pier flanks. This separation of flow filaments caused two separate flow holes around the flanks rapidly and joined in front of the pier. After two minutes, horseshoe vortices appeared. The material around the pier was swept by horseshoe vortices and transported by wake vortices to downstream and began to form a mound. While horseshoe region and wake region were still active after 10 minutes, the transport behind the cylinder was considerably reduced. The crest of the mound was almost 3 pier diameters away from the pier after 25 minutes and scour hole boundary reached 6 cm away from the front of the pier. It was suggested that strong downflow was mainly responsible for the scour activity at the pier base and formed the shape of erosion area with horseshoe vortices. He explained how the shape of scour hole formed as "...over the greater percentage of the scour hole area material was not eroded but was observed to slip down the slope as the scour hole increased. ...the establishment of a conical hole at a slope equal to the dynamic angle of repose (p.92)".

He pointed that total erosion rate significantly reduced after 30 minutes of run and this reduction carried on over time. Another highlight at this stage was moving of crest of mound further to downstream due to exposure of slow erosion and reducing of upstream slope of mound. Time was chosen as 2.5 hours to reach equilibrium scour depth. Previously, a condition was suggested to be carried out for equilibrium scour depth: "The spiral roller takes up an equilibrium position dictated by the geometry of the experiment and the properties of approach flow and/or the roller's scouring potential becomes insufficient to remove further particles (p.89)".

As a result of his experiments, equilibrium scour depth for both steady and unsteady flows reached the same value. However initial scour depths for unsteady flow were lower than steady flow and steady flow reached equilibrium state earlier, those because of initial flow was low for unsteady flow. Velocity and turbulence profile and shear stress around a pier were also investigated elaborately.

Kothyari et al. (1992b) conducted their clear water experiments in a flume with a length of 30 m, width of 1.0 m and depth of 0.60 m under both steady and unsteady flow. They used five different circular piers with diameters of 65 mm, 80 mm, 115 mm, 170 mm and 200 mm. Both uniform and non-uniform sediments were tested with various numbers of median grain sizes and geometric standard deviations. Their hydrographs were formed as stepwise by adjusting inlet valve and tail gate simultaneously.

They proposed a scheme to predict maximum scour depth under steady flows. Also, it is stated that if each step of stepwise hydrographs were assumed as steady, this scheme gave satisfactory results for clear water conditions.

Chang et al. (2004) carried out their clear water experiments in a flume with length of 36 m, width of 1 m and depth of 1.1 m under both steady and unsteady flows. A circular pier with a diameter of 100 mm and two different sizes of uniform and non-uniform sediments were used. They formed stepwise hydrographs by adjusting inlet valve and tail gate.

As a result of their experiments, scour depth increased quicker in steady flow than unsteady flow but they reached almost same scour depth at the peak of the hydrograph. Only a slight increment was observed in unsteady flow after the peak discharge. They compared the effects of peak velocity and time to peak flow by keeping each one constant in different hydrographs. They found that effect of time to peak flow had no significant role and peak velocity was more important on development of scour depth. A scheme giving consistent results with Kothyari, Raju and Garde (1992b) was

proposed to predict equilibrium scour depth under unsteady flows by considering each step of stepwise hydrograph as steady.

Lai et al. (2009) studied maximum clear water scour depth under unsteady flow for both uniform and non-uniform sediments by evaluating data available in literature. They suggested that recession limb of a hydrograph had no significant role on maximum scour depth and they assumed scour depth at peak flow as maximum scour depth. Therefore only rising limb of hydrographs (named as Γ shaped hydrograph) was taken into consideration in their study. They proposed an equation to predict maximum clear water scour depth under unsteady flow.

Hager & Unger (2010) conducted their clear water experiments in a flume with length of 11 m length and 1 m width under unsteady flow conditions for both uniform and non-uniform sediment. They formed their flood waves with times to peak flow ranging from 300 s to 3600 s, discharges up to 0.13 m³ and flow depths up to 0.30 m. Two different circular piers were used with diameters of 11 cm and 20 cm. It was observed that scour depth didn't changed after the peak discharge therefore only rising limb of the flood was regarded.

They found that time ratio of hydrographs had no importance on final scour depth and, it was essentially affected by the flow velocity. They adjusted their previous equation for steady flow and proposed a new equation to predict final scour depth under unsteady flow. According to the comparison between two equations, steady flow overestimated scour depth was 30 % more than unsteady flow.

2.3 Available Formulas Concerning Live Bed Scour in Literature

Although there are many equations concerning local scour in the literature, only the equations about live bed scour are summarized below.

2.3.1 Laursen (1963)

Laursen (1963) proposed an equation to predict the equilibrium scour depth as a result of the experiments performed under live bed conditions and steady flow.

$$\frac{d_e}{b} = 1.11 \left(\frac{y}{b} \right)^{0.5} \quad (2.1)$$

where d_e is equilibrium scour depth, b is pier width and y is flow depth.

2.3.2 Breusers, Nicollet & Shen (1977)

Breusers, et al. (1977) proposed an equation to predict the equilibrium scour depth as a result of the experiments performed under live bed conditions and steady flow.

$$\frac{d_e}{b} = K_I \left[2.0 \tanh \left(\frac{y}{b} \right) \right] K_s K_\theta \quad (2.2)$$

where K_I is flow intensity factor; 1 for live bed scour, K_s is shape factor; 1 for circular pier, K_θ is pier alignment factor; 1 for the flow in the perpendicular direction to the pier.

2.3.3 Jain & Fischer (1979)

Jain & Fischer (1979), based on data collected from experiments with high Froude numbers, aimed to check the accuracy of the equations formed from experiments with low Froude numbers and proposed an equation to predict the equilibrium scour depth for live bed scour under steady flow (for $Fr - Fr_c > 0.2$).

$$\frac{d_e}{b} = 2.0 (Fr - Fr_c)^{0.25} \left(\frac{y}{b} \right)^{0.5} \quad (2.3)$$

Herein, F_c is the Froude number corresponding to the critical velocity ($F_c = V_c / \sqrt{gy}$). The critical velocity V_c can be found from (2.9)

2.3.4 Froehlich (1988)

Froehlich (1988) proposed an equation to predict equilibrium scour depth under steady flow and live bed conditions as a result of field measurements.

$$\frac{d_e}{b} = 0.32 K_s \left(\frac{b'}{b} \right)^{0.62} \left(\frac{y}{b} \right)^{0.46} Fr^{0.20} \left(\frac{b}{d_{50}} \right)^{0.08} + 1 \quad (2.4)$$

where, K_s is pier shape factor; 1.3 for square nosed piers, 1.0 for circular and round nosed piers and 0.7 for sharp nosed piers. b' is the normalized pier width according to the approaching flow and is expressed by $b' = b \cos \theta + L \sin \theta$, with the angle of the approaching flow θ and the pier length L .

2.3.5 Gao, Posada & Nordin (1993)

Gao et al. (1993) proposed an equation to predict equilibrium scour depth under live bed conditions by collecting data from previous experiments and measurements.

$$d_e = 0.65 K_s y^{0.15} b^{0.60} d_{50}^{-0.07} \left(\frac{V - V_i}{V_c - V_i} \right)^n \quad (2.5)$$

where,

$$n = \left(\frac{V_c}{V} \right)^{9.35 + 2.23 \log d_{50}} \quad (2.6)$$

V_i is the incipient flow velocity, required to start scour around pier and V_i/V_c varies between 0.4-0.6. V_c can be found from Eq. (2.9).

2.3.6 Melville (1997)

Melville (1997), in accordance with the collected data, proposed an equation to predict the equilibrium scour depth for both clear water and live bed scour under steady flow.

$$d_e = K_{yW} K_I K_d K_s K_\theta K_G \quad (2.7)$$

The depth factor, K_{yW} is defined according to relative approach current depth (y/b). For piers, this coefficient is indicated by K_{yb} .

$$K_{yb} = 2,4b; \quad b/y < 0,7, \quad (2.8a)$$

$$K_{yb} = 2\sqrt{yb}; \quad 0,7 < b/y < 5, \quad (2.8b)$$

$$K_{yb} = 4,5y; \quad b/y > 5 \quad (2.8c)$$

The flow intensity factor, K_I is defined as $K_I = V/V_c$ for uniform bed material. Melville and Sutherland (1988) proposed the following equation for the calculation of critical velocity (V_c).

$$\frac{V_c}{u_{*c}} = 5,75 \log \left(5,53 \frac{y}{d_{50}} \right) \quad (2.9)$$

The value u_{*c} in this equation is the critical shear velocity and can be found from the Shields diagram based on the d_{50} median grain size. However, in order to be practical, Melville (1997) transformed the Shields diagram to following equation set to calculate u_{*c} , by taking u_{*c} in m/s and d_{50} in mm;

$$u_{*c} = 0,0115 + 0,0125d_{50}^{1,4}; \quad 0,1 \text{ mm} < d_{50} < 1 \text{ mm} \quad (2.10a)$$

$$u_{*c} = 0,0305d_{50}^{0,5} - 0,0065d_{50}^{-1}; \quad 1 \text{ mm} < d_{50} < 100 \text{ mm} \quad (2.10b)$$

The grain size factor, K_d is defined according to relative grain size (b/d_{50}) as following:

$$K_d = 1; \quad b/d_{50} > 25, \quad K_d = 0,57 \log(2,24b/d_{50}); \quad b/d_{50} < 25 \quad (2.11)$$

The pier shape factor, K_s is defined 1.0 for circular and round nosed piers, 1.1 for square nosed piers and 0.9 for sharp nosed piers.

The pier alignment factor, K_θ is defined 1.0 for the flow in the perpendicular direction to the pier. Table 2.1 gives various K_θ values for different pier alignments according to ratio between pier length (l) and pier width (b).

The channel geometry factor, $K_G = 1$ for piers.

2.3.7 Richardson & Davis (2001)

Richardson & Davis (2001) proposed an equation to predict equilibrium scour depth for both clear water and live bed scour under steady flow.

$$d_e = 2.0K_1K_2K_3K_4\left(\frac{y}{b}\right)^{0.35} Fr^{0.43} \quad (2.12)$$

where, the pier shape factor, $K_1 = K_s$ is 1.0 for circular and round nosed piers, 1.1 for square nosed piers and 0.9 for sharp nosed piers. The pier alignment factor, $K_2 = K_\theta$ is given on Table 2.1. It can also be found from (2.13);

$$K_2 = \cos\theta + L/b \sin\theta^{0.65} \quad (2.13)$$

Table 2.1 K_θ values for various l/b and θ

l/b	K_θ				
	$\theta = 0^\circ$	$\theta = 15^\circ$	$\theta = 30^\circ$	$\theta = 45^\circ$	$\theta = 90^\circ$
4	1,00	1,50	2,00	2,30	2,50
8	1,00	2,00	2,75	3,30	3,90
12	1,00	2,50	3,50	4,30	5,00

The bed roughness factor, $K_3 = K_b$ is given in Table 2.2 and the dune height is indicated by H.

Table 2.2 K_3 values according to bed roughness.

Bed Condition	Dune Height (m)	K_3
Clear Water Scour	Not Available	1.1
Plane bed and Antidune Flow	Not Available	1.1
Small Dunes	$3 > H \geq 0.6$	1.1
Medium Dunes	$9 > H \geq 3$	1.2 to 1.1
Large Dunes	$H \geq 9$	1.3

The grain shape factor, $K_4 = K_d$ is 1.0 for $d_{50} < 2 \text{ mm}$. If $d_{50} \geq 2 \text{ mm}$,

$$K_4 = 0.4(V_R)^{0.15} \quad (2.14)$$

where,

$$V_R = \frac{V - V_{i,d_{50}}}{V_{c,d_{50}} - V_{i,d_{95}}} \quad (2.15)$$

V_{i,d_x} , is the initial scour velocity around d_x grain size.

$$V_{i,d_x} = 0.645 \left(\frac{y}{b} \right)^{0.053} V_{c,d_x} \quad (2.16)$$

V_{c,d_x} , is the critical velocity for d_x grain size.

$$V_{c,d_x} = 6.19 y^{1/6} d_x^{1/3} \quad (2.17)$$

Minimum value for K_4 is 0.4 and only should be used for $V < V_{i,d_{50}}$.

2.3.8 Yanmaz (2001)

Yanmaz (2001) proposed an equation to predict the equilibrium scour depth as a result of the experiments performed under live bed conditions and steady flow.

$$\frac{d_e}{b} = 1.564 \left(\frac{y}{b} \right)^{0.405} Fr^{0.413} \quad (2.18)$$

2.3.9 Sheppard & Miller (2006)

Sheppard & Miller (2006) performed experiments under steady flow and indicated two distinct peaks for live bed equilibrium scour depth. The first is the point where the flow velocity exceeds the critical velocity and bed material movement starts. After the first peak, a deeper equilibrium scour depth is defined as the second peak. Two separate equations are proposed to predict the scour depth in these two regions.

$$1 < V/V_c \leq V_p/V_c;$$

$$\frac{d_e}{b} = f_1 \left(\frac{y}{b} \right) \left[2.2 \left(\frac{V/V_c - 1}{V_p/V_c - 1} \right) + 2.5 f_2 \left(\frac{b}{d_{50}} \right) \left(\frac{V_p/V_c - V/V_c}{V_p/V_c - 1} \right) \right] \quad (2.19)$$

$$V/V_c > V_p/V_c;$$

$$\frac{d_e}{b} = 2.2 f_1 \left(\frac{y}{b} \right) \quad (2.20)$$

where,

$$f_1 \left(\frac{y}{b} \right) \equiv \tanh \left[\left(\frac{y}{b} \right)^{0.4} \right] \quad (2.21a)$$

$$f_2 \left(\frac{b}{d_{50}} \right) \equiv \frac{d / d_{50}}{0.4 (d / d_{50})^{1/2} + 10.6 (d / d_{50})^{-0.13}} \quad (2.21b)$$

In order to simplify the equation, the local minimum area of live bed scour was bypassed and the threshold scour and live bed peak scour were connected with a straight line (Sheppard, 2003). Because of this, as stated, the equation set slightly overpredicted for live bed conditions.

2.4 Parameters Affecting Scour Depth

2.4.1 Influence of the Relative Depth on the Scour Depth

The effect of the approaching flow depth increase on the scour depth has long been known issue. As a result of pier obstructing the flow, a surface roller is formed on the upstream side of the pier. Relation of scour depth with approaching flow depth depends on the interaction of surface roller and horseshoe vortices acting in opposite direction (Ettema, 1980). As the approaching flow depth increases, the interaction of these two vortices will decrease, downflow will weaken and after a point, scour depth will be independent of flow depth. Many previous studies show that (Dietz, 1972, Breusers et al., 1977, Raudkivi, 1986) after $y/b > 3$, the scour depth is independent from the flow depth. Melville (1997) suggested this boundary value as $b/y < 0.7$ and found that scour depth was affected by both flow depth and pier width when $0.7 < b/y < 5$ and it was independent from the pier width at shallow water conditions ($b/y > 5$).

2.4.2 Influence of Approaching Flow Velocity

The flow velocity is one of the primary parameters in scour. Because in the case of $V < V_c$, there will be no material movement at the bed and clear water conditions will be present, whereas in the case of $V > V_c$, live bed scour will occur. This situation affects the whole behaviour of scour. The effect of approaching flow velocity has been taken into account by different researchers in different ways. For example, Hancu (1971), Jain & Fischer (1980) and Yanmaz (2001) used Fr , Chiew (1984) and Melville

(1997) used V/V_c and Oliveto & Hager (2002) and Kothyari et al. (2007) used densimetric particle froude number (F_d).

2.4.3 Influence of Pier and Grain Size

The properties of bed material can be generalized under subheadings such as the grain diameter, uniformity, cohesiveness etc. Previous studies have shown that (Jain and Fischer, 1979, Ettema, 1980, Chiew, 1984) as the grain size decreases, the scour depth increases. Under clear water conditions, Ettema (1980) and Melville (1997) suggested that the sediment grain diameter is not significant on scour depth after $b/d_{50} > 25$, and Chiew (1984) stated that scour depth is independent of grain diameter after $b/d_{50} > 50$ as a result of his experiments carried out under live bed conditions.

Another important factor about grain is the gradation of the grain. If $\sigma_g > 1.3-1.4$, sediment is considered as non-uniform. If the flow velocity is high enough to move the fine-grained material, but too low to move the coarse-grained material, the coarse-grained material forms an armor layer. Previous studies (Ettema, 1980, Chiew, 1984) have shown that as the geometric standard deviation (σ_g) increases, the scour depth decreases.

2.4.4 Influence of Time

Melville & Chiew (1999) reported that 50% -80% of the equilibrium scour depth is reached in 10% of the time required to reach the equilibrium scour depth as a result of the experiments they performed under clear water conditions. It was stated that experiments should last at least a few days to reach equilibrium scour depth and experiments lasting 10-12 hours could give less than 50% of the equilibrium scouring depth. Equilibrium scour depth was found to be achieved if the scouring in the last 24 was less than 5% of the pier width. Many equations proposed in the literature take the equilibrium scour depth into account with the intent of calculating the maximum scour depth such as Chabert and Engeldinger (1956), Shen et al. (1969), Jain and Fischer (1979), Ettema (1980), Chee (1982), Chiew (1984), Yanmaz & Altinbilek (1991),

Melville (1997), Melville & Chiew (1999), Sheppard et al. (2004), Sheppard & Miller (2006). However, as Kothyari et al. (1992a) pointed out, the peak flow duration could be much shorter than the duration required to reach the equilibrium scour depth. Mia & Nago (2003), Kothyari et al. (2007) also stated that duration of flood waves could be much shorter and the final scour depth could be significantly smaller than the equilibrium scour depth. Yanmaz (2001) stated that a 50-hours experiment in a Froude model built on a 1/50 scale in the laboratory would correspond to 354 hours in nature.

The experiments in this study were performed under short-term hydrographs. On one hand, the effects of hydrographs with the same peak flow but with different durations, on the other hand, the same durations but different peak flow on scour depth were investigated. These hydrographs are presented in Section 4.4 in detail.

CHAPTER THREE

THEORETICAL ASPECT

Total scour is expressed by the interaction of three distinct scour components. These are erosions occurring on river bed over a long period of time, contraction scour due to a natural or bridge contraction and local scour originating from a pier or abutment that obstructs the flow (Melville, 1975). As Wardhana & Hadipriono (2003) stated, 62% of bridge failures are directly arisen from floods. Chiew (1984) also stated that bridge failures caused by local scour around bridge piers usually occur during floods. Therefore, local scours have an utmost important place within the total scour mechanism.

3.1 Mechanism of Local Scour

Scour resulting from an object that directly obstructs the flow is called local scour. This object can be a bridge pier, an abutment or an open-channel junction. Mechanism of scour around the bridge pier is based on the vortices around the pier, as claimed by many researchers (Laursen & Toch, 1956, Melville, 1975, Ettema, 1980, Chiew, 1984, Raudkivi, 1986 etc.)

If approaching flow is obstructed by pier, water surges at the upstream of pier, flow velocity approaches to zero and high pressure occurs at this point; so a downflow forms from this high pressure point to the base. As a result of interaction between approaching flow with downflow, so-called horseshoe vortices form (Raudkivi, 1986). Melville (1975) described the horseshoe vortex as follows:

“Horseshoe vortex is initially small, roughly circular in cross-section, and comparatively weak, increases dramatically in size and strength as the scour hole forms. During the development of scour hole, the horseshoe vortex expands and moves down into the hole, increasing its circulation throughout the scour process, but a diminishing rate. The shape of vortex follows that of the scour hole-cylinder combination” (p.194).

As prescribed above, the horseshoe vortex develops together with the scour hole. Chiew (1984) stated that “The downflow in front of the pier is the main cause of erosion of local scour around bridge piers. It acts somewhat like vertical jet in eroding the bed material (p.13).” Raudkivi (1986) also stated that the main component of the scour is downflow and the horseshoe vortex is not the cause, but the result of scour.

Another factor that cause scouring is wake vortex (Melville ,1975). He explains the wake vortices as

“The wake-vortex system, however is generated by the pier itself. The vortices are formed by the rolling up of the unstable shear layers generated at the pier surface at the separation points.... The wake vortex system acts somewhat like a vacuum cleaner in removing the bed material, which is carried downstream by the eddies being shed from the pier” (p.4).

3.1.1 Clear Water Scour

One factor to be considered when examining local scour is whether sediment movement occurs on the bed or not during the flow. No sediment movement occurs on the bed unless flow velocity engenders a greater shear stress than the critical shear stress of the bed material, then no sediment supply into the scour hole takes place. Such local scours are called clear water scour.

3.1.2 Live Bed Scour

Sediment movement occurs on the bed if the flow creates a shear stress greater than the critical shear stress of the bed material. In this case, sediment cluster entrained from the upstream fills the scour hole and approaching flow continues to scour until the next sediment cluster reaches. The state in which the sediment cluster entrained into the scour hole with the sediment separated from the scour hole oscillates about a constant value is called live bed equilibrium scour depth.

3.2 Parameters of Scour Mechanism

The parameters affecting the scour mechanism around bridge pier are given below (Yanmaz, 2002).

ρ , density of water; ν , kinematic viscosity of water; g , gravitational acceleration; y , approaching flow depth; V , mean approaching flow velocity; θ , angle between flow and pier axis; u_* , shear velocity; S_0 , bed slope; B , river width; C_c , contraction coefficient; K_θ , factor indicating the effect of flow alignment with respect to pier axis; K_a , factor of river route effect; K_b , roughness of the river bed upstream; K_c , roughness of channel sidewalls; K_G , factor indicating the effect of cross-sectional geometry of the channel; ρ_s , density of grain; d_{50} , median diameter of grain; σ_g , geometric standard deviation of grain distribution; C , cohesion of bed material; K_d , factor indicating the effect of grain shape; b , pier width; K_s , factor indicating the effect of pier shape; K_g , factor indicating the group effect of piers; K_r , roughness of pier surface; K_v , factor indicating the effect vertical inclination of pier; t , duration of flow.

From these numerous dimensional variables, several dimensionless parameters were derived. The most important dimensionless parameters affecting the scour are flow velocity (taking into account as flow intensity (V/V_c) , Froude number, densimetric Froude number, ... etc.), relative flow depth (y/b) , geometric standard deviation σ_g , relative pier width (b/d_{50}) . The relative scour depth (d/b) is generally expressed in terms of these dimensionless parameters.

Some researchers (Oliveto & Hager, 2002; Kothyari, et al. 2007) investigated also the effect of this dimensionless densimetric particle Froude Number (F_d) .

3.3 Basic Equations of Local Scour

The equations of the water flow around bridge pier and sediment transport are three dimensional complex equations. Their solutions require advanced numerical solution techniques. In this study, Flow-3D software manufactured by Flow Science Inc. was

used to solve these equations. Flow-3D is a CFD (computational fluid dynamics) software that is based of VOF (Volume of Fluid) method. It simultaneously solves the three dimensional Navier-Stokes equations (3.1a, b and c) and continuity equation (3.4):

$$\frac{\partial u}{\partial t} + \frac{1}{V_F} \left(uA_x \frac{\partial u}{\partial x} + vA_y R \frac{\partial u}{\partial y} + wA_z \frac{\partial u}{\partial z} \right) - \xi \frac{A_y v^2}{xV_F} =$$

$$-\frac{1}{\rho} \frac{\partial p}{\partial x} + G_x + f_x - b_x - \frac{R_{SOR}}{\rho V_F} (u - u_w - \partial u_s)$$

$$\frac{\partial v}{\partial t} + \frac{1}{V_F} \left(uA_x \frac{\partial v}{\partial x} + vA_y R \frac{\partial v}{\partial y} + wA_z \frac{\partial v}{\partial z} \right) + \xi \frac{A_y uv}{xV_F} =$$

$$-\frac{1}{\rho} \left(R \frac{\partial p}{\partial y} \right) + G_y + f_y - b_y - \frac{R_{SOR}}{\rho V_F} (v - v_w - \partial v_s)$$

$$\frac{\partial w}{\partial t} + \frac{1}{V_F} \left(uA_x \frac{\partial w}{\partial x} + vA_y R \frac{\partial w}{\partial y} + wA_z \frac{\partial w}{\partial z} \right) =$$

$$-\frac{1}{\rho} \frac{\partial p}{\partial z} + G_z + f_z - b_z - \frac{R_{SOR}}{\rho V_F} (w - w_w - \partial w_s)$$

where u, v, w are the velocity components in x, y, z coordinate directions, A_x, A_y, A_z are the fractional area open to the volume in x, y, z coordinate directions, V_F is the fractional volume open to flow, R_{SOR} is a mass source, ρ is the fluid density, G_x, G_y, G_z are body accelerations, f_x, f_y, f_z are viscous accelerations, b_x, b_y, b_z are flow losses in porous media or across porous baffle plates, u_w, v_w, w_w are the velocity of the source component, u_s, v_s, w_s are the velocity of the fluid at the surface of the source relative to the source itself, R is a coefficient determined by $R = r_m/r$; r is cylindrical coordinate and r_m is a fixed reference radius. For a variable dynamic viscosity μ , the viscous accelerations are:

$$\rho V_F f_x = wsx - \left[\frac{\partial}{\partial x} (A_x \tau_{xx}) + R \frac{\partial}{\partial y} (A_y \tau_{xy}) + \frac{\partial}{\partial z} (A_z \tau_{xz}) + \frac{\xi}{x} (A_x \tau_{xx} - A_y \tau_{yy}) \right] \quad (3.2a)$$

$$\rho V_F f_y = wsy - \left[\frac{\partial}{\partial x} (A_x \tau_{xy}) + R \frac{\partial}{\partial y} (A_y \tau_{yy}) + \frac{\partial}{\partial z} (A_z \tau_{yz}) + \frac{\xi}{x} (A_x + A_y \tau_{xy}) \right] \quad (3.2b)$$

$$\rho V_F f_z = wsz - \left[\frac{\partial}{\partial x} (A_x \tau_{xz}) + R \frac{\partial}{\partial y} (A_y \tau_{yz}) + \frac{\partial}{\partial z} (A_z \tau_{zz}) + \frac{\xi}{x} (A_x \tau_{xz}) \right] \quad (3.2c)$$

herein,

$$\tau_{xx} = -2\mu \left[\frac{\partial}{\partial x} - \frac{1}{3} \left(\frac{\partial u}{\partial x} + R \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z} + \frac{\xi u}{x} \right) \right] \quad (3.3a)$$

$$\tau_{yy} = -2\mu \left[R \frac{\partial v}{\partial y} + \xi \frac{u}{x} - \frac{1}{3} \left(\frac{\partial u}{\partial x} + R \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z} + \frac{\xi u}{x} \right) \right] \quad (3.3b)$$

$$\tau_{zz} = -2\mu \left[\frac{\partial w}{\partial z} - \frac{1}{3} \left(\frac{\partial u}{\partial x} + R \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z} + \frac{\xi u}{x} \right) \right] \quad (3.3c)$$

$$\tau_{xy} = -\mu \left(\frac{\partial v}{\partial x} + R \frac{\partial u}{\partial y} - \frac{\xi v}{x} \right) \quad (3.3d)$$

$$\tau_{xz} = -\mu \left(\frac{\partial u}{\partial z} + \frac{\partial w}{\partial x} \right) \quad (3.3e)$$

$$\tau_{yz} = -\mu \left(\frac{\partial v}{\partial z} + R \frac{\partial w}{\partial y} \right) \quad (3.3f)$$

$$\frac{\partial}{\partial x} (uA_x) + R \frac{\partial}{\partial y} (vA_y) + \frac{\partial}{\partial z} (wA_z) + \xi \frac{uA_x}{x} = \frac{R_{SOR}}{\rho} \quad (3.4)$$

There are six different turbulence models available in Flow 3D: the Prandtl mixing length model, one-equation (k) model, two-equation ($k - \varepsilon$) model, RNG (renormalization group) model, two-equation ($k - \omega$) model and LES (large eddies simulation) model.

Flow 3D estimates the motion of sediment by predicting following the erosion, advection and deposition of sediment (Flow3D v11.2 Manual). It does so by:

- Computing the suspended sediment transport
- Computing the settling of sediment due to gravity
- Computing the entrainment of the sediment due to bed shearing and flow perturbations
- Computing the bed load transport, whereby sediment grains roll, hop or slide along the packed sediment bed

Flow 3D computes the entrainment lift velocity from the equation (3.5) of Mastbergen & Van den Berg (2003) and settling velocity from the equation (3.6) of Soulsby (1997):

$$u_{lift,i} = \alpha_i n_s d_*^{0.3} (\theta_i - \theta'_{cr,i})^{1.5} \sqrt{\frac{\|g\| d_i (\rho_{s,i} - \rho_f)}{\rho_f}} \quad (3.5)$$

$$u_{settling,i} = \frac{\nu_f}{d_i} \left[(10.36^2 + 1.049 d_{*,i}^3)^{0.5} - 10.36 \right] \quad (3.6)$$

where α_i is entrainment coefficient as an empirical parameter and suggested 0.018 for sands, n_s is the outward pointing normal to the packed bed interface, θ_i and $\theta'_{cr,i}$ are local and modified critical shear stress parameters respectively, ν_f is kinematic viscosity of fluid and d_* is dimensionless median grain parameter calculated from:

$$d_{*,i} = d_i \left[\frac{\rho_f (\rho_{s,i} - \rho_f) \|g\|}{\mu_f^2} \right]^{1/3} \quad (3.7)$$

where μ_f is dynamic viscosity of fluid. It calculates the Shields parameter from Eq. (3.8) and the critical Shields parameter from Eq. (3.9) of Soulsby – Whitehouse, respectively:

$$\theta_i = \frac{\tau}{\|g\| d_i (\rho_{s,i} - \rho_f)} \quad (3.8)$$

where τ is the quadratic law of bottom shear stress for 3D turbulent flow and shallow water turbulent flow and calculated using the law of the wall, $\|g\|$ is the magnitude of the gravitational vector, d_i is median grain diameter and $\rho_{s,i}$ and ρ_f are density of sediment and fluid respectively

$$\theta_{cr,i} = \frac{0.3}{1 + 1.2 d_{*,i}} + 0.055 \left[1 - \exp(-0.02 d_{*,i}) \right] \quad (3.9)$$

It modifies the critical Shields parameter for sloping surfaces to include angle of repose:

$$\theta'_{cr,i} = \theta_{cr,i} \frac{\cos \psi \sin \beta + \sqrt{\cos^2 \beta \tan^2 \varphi_i - \sin^2 \psi \sin^2 \beta}}{\tan \varphi_i} \quad (3.10)$$

where β is the angle of slope of bed, φ_i is user-defined angle of repose for the sediment i and ψ is the angle between the flow and the upslope direction.

For bed load transport, one of following three equations can be chosen to calculate volumetric transport rate of sediment per width of bed:

- Meyer, Peter & Müller (1948)'s formula:

$$\phi_i = \beta_{MPM,i} - (\theta_i - \theta'_{cr,i})^{1.5} c_{b,i} \quad (3.11)$$

- Nielsen (1992)'s formula:

$$\phi_i = \beta_{Nie,i} - \theta_i^{0.5} (\theta_i - \theta'_{cr,i}) c_{b,i} \quad (3.12)$$

- Van Rijn (1984)'s formula:

$$\phi_i = \beta_{VR,i} d_{*,i}^{-0.3} \left(\frac{\theta_i}{\theta'_{cr,i}} - 1.0 \right) c_{b,i} \quad (3.13)$$

where β_{MPM} , β_{Nie} and β_{VR} are bed-load transport coefficients and suggested as 8.0, 12.0 and 0.053 respectively, $c_{b,i}$ is the volume fraction of species i in the bed material (it doesn't exist in the original equations but software added it to account for the effect of multiple species) and ϕ_i is the dimensionless bed-load transport rate and related to the volumetric bed-load transport rate $q_{b,i}$:

$$q_{b,i} = \phi_i \left[\|g\| \left(\frac{\rho_{s,i} - \rho_f}{\rho_f} \right) d_i^3 \right]^{1/2} \quad (3.14)$$

It converts volumetric bed-load transport rate into the bed-load velocity to compute the motion of the sediment in each computational cell (Van Rijn, 1984):

$$u_{bedload,i} = \frac{q_{b,i}}{\delta_i c_{b,i} f_b} \quad (3.15)$$

where f_b is the critical packing fraction of the sediment and δ_i is the bed-load thickness and calculated from following equation:

$$\frac{\delta_i}{d_i} = 0.3 d_{*,i}^{0.7} \left(\frac{\theta_i}{\theta'_{cr,i}} - 1.0 \right)^{0.5} \quad (3.16)$$

Another factor that is required to be calculated is suspended sediment concentration and it is calculated by its own transport equation:

$$\frac{\partial C_{s,i}}{\partial t} + \nabla(u_{s,i} C_{s,i}) = \nabla \cdot \nabla(D C_{s,i}) \quad (3.17)$$

where $C_{s,i}$ is the suspended sediment concentration (sediment mass per volume of sediment-fluid mixture) of species i , D is diffusivity, $u_{s,i}$ is the suspended sediment velocity of species i . Suspended sediment volume concentration ($c_{s,i}$) is calculated from:

$$c_{s,i} = C_{s,i} / \rho_{s,i} \quad (3.18)$$

and the suspended sediment velocity:

$$u_{s,i} = \bar{u} + u_{\text{settling},i} c_{s,i} \quad (3.19)$$

where $\bar{u}$ is velocity of sediment-fluid mixture.

Flow-3D has an option to calculate maximum turbulent mixing length dynamically for the turbulence models k- ϵ , k- ω and RNG. When this option is selected, the model computes the bounded turbulent time and length scales from the following equations:

$$\tilde{T}_T = \max \left[T_T^{\text{low}}, \min \left(T_T^{\text{up}}, \frac{k_T}{\epsilon_T} \right) \right] \quad (3.20)$$

$$\tilde{L}_T = \max \left[L_T^{\text{low}}, \min \left(L_T^{\text{up}}, \text{CNU} \frac{k_T^{3/2}}{\epsilon_T} \right) \right] \quad (3.21)$$

where, k_T is the turbulent kinetical energy and calculated from Eq. (3.22), ϵ_T is the turbulent kinetic energy dissipation and calculated from Eq. (3.23), CNU is a parameter taken 0.09 by default.

$$k_T = \frac{1}{2} \left(\overline{u'^2} + \overline{v'^2} + \overline{w'^2} \right) \quad (3.22)$$

$$\begin{aligned} & \frac{\partial \epsilon_T}{\partial t} + \frac{1}{V_F} \left\{ u A_x \frac{\partial \epsilon_T}{\partial x} + v A_y \frac{\partial \epsilon_T}{\partial y} + w A_z \frac{\partial \epsilon_T}{\partial z} \right\} \\ &= \frac{\text{CDIS1} \cdot \epsilon_T}{k_T} (P_T + \text{CDIS3} \cdot G_T) + \text{Diff}_\epsilon - \text{CDIS2} \frac{\epsilon_T^2}{k_T} \end{aligned} \quad (3.23)$$

where u' , v' and w' are components of the fluid velocity associated with the chaotic fluid turbulent velocity fluctuations, CDIS1, CDIS2 and CDIS3 are coefficients taken 1.44, 1.92 and 1.02 by default, P_T is turbulent kinetic energy production and calculated from Eq. (3.24), G_T is buoyancy production term and taken 0 as default for the

problems those are not related with thermally buoyant flow and $Diff_\epsilon$ is the diffusion of dissipation and calculated from Eq. (3.25).

$$P_T = \left(\frac{\mu}{\rho V_F} \right) \left\{ \begin{aligned} & 2A_x \left(\frac{\partial u}{\partial x} \right)^2 + 2A_y \left(R \frac{\partial v}{\partial y} + \xi \frac{u}{x} \right)^2 + 2A_z \left(\frac{\partial w}{\partial z} \right)^2 \\ & + \left(\frac{\partial v}{\partial x} + R \frac{\partial u}{\partial y} - \xi \frac{v}{x} \right) \left[A_x \frac{\partial v}{\partial x} + A_y \left(R \frac{\partial u}{\partial y} - \xi \frac{v}{x} \right) \right] \\ & + \left(\frac{\partial v}{\partial z} + \frac{\partial u}{\partial x} \right) \left(A_z \frac{\partial u}{\partial z} + A_x \frac{\partial w}{\partial x} \right) \\ & + \left(\frac{\partial v}{\partial z} + R \frac{\partial w}{\partial y} \right) \left(A_z \frac{\partial v}{\partial z} + A_y R \frac{\partial w}{\partial y} \right) \end{aligned} \right\} \quad (3.24)$$

$$Diff_\epsilon = \frac{1}{V_F} \left\{ \frac{\partial}{\partial x} \left(v_\epsilon A_x \frac{\partial \epsilon_T}{\partial x} \right) + R \frac{\partial}{\partial y} \left(v_\epsilon A_y \frac{\partial \epsilon_T}{\partial y} \right) + \frac{\partial}{\partial z} \left(v_\epsilon A_z \frac{\partial \epsilon_T}{\partial z} \right) + \xi \frac{v_\epsilon A_x \epsilon_{Tx}}{x} \right\} \quad (3.25)$$

Flow-3D computes the maximum turbulence mixing length as 7% of the length scale.

3.4 Numerical Modelling in Flow-3D

In numerical modeling, the last 5.6 meters of flume was disregarded in order to reduce the number of the mesh cells, hence to save the simulation time. CGS unit system was preferred for the simulations. Mesh cell sizes were taken as 1x1 cm along the flow direction and the channel width and 0.5x0.5 in the vertical direction. General overview of the flume model is given in Figure 3.1. Red zones indicate the concrete blocks, blue zone indicates the packed sediment and cyan layer indicates the initial water surface.

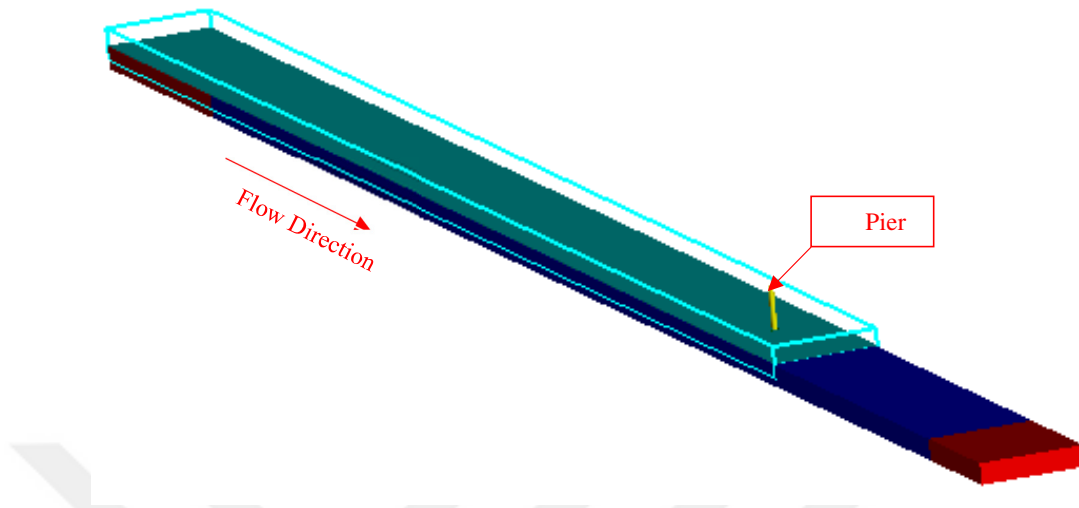


Figure 3.1 General overview of the flume model

The sides and the bottom of the flume were defined as wall boundary condition. The boundary condition at the downstream end was defined as outflow. The upstream boundary condition was defined as hydrograph together with water elevation. Figure 3.2 a, b, c and d represent the interface of boundary conditions and Figure 3.3 a and b represent the defined discharge and flow depth, respectively. 16 cm thickness of sediment layer was taken into account while determining the flow levels.

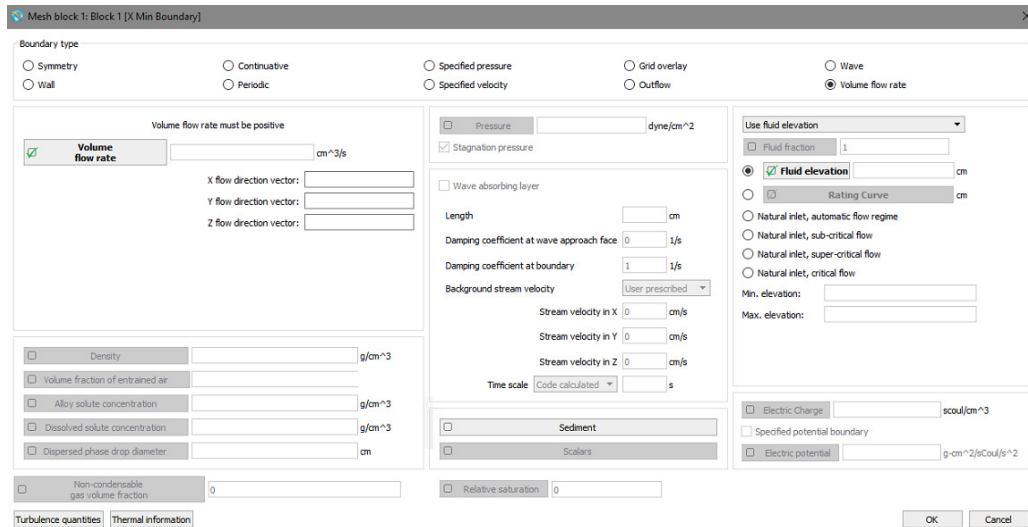


Figure 3.2 a) Interface of boundary condition of the flume inlet

Mesh block 1: Block 1 [Y Min Boundary]

Boundary type

☐ Symmetry
☒ Wall
☐ Continutive
☐ Periodic
☐ Specified pressure
☐ Specified velocity
☐ Grid overlay
☐ Outflow
☐ Wave
☐ Volume flow rate

☐ Velocity
 X Velocity: cm/s
 Y Velocity: cm/s
 Z Velocity: cm/s

☐ Pressure dyne/cm²
☒ Stagnation pressure

☐ Wave absorbing layer
 Length cm
 Damping coefficient at wave approach face 1/s
 Damping coefficient at boundary 1/s
 Background stream velocity User prescribed
 Stream velocity in X 0 cm/s
 Stream velocity in Y 0 cm/s
 Stream velocity in Z 0 cm/s
 Time scale Code calculated s

☐ Density g/cm³
☐ Volume fraction of entrained air
☐ Alloy solute concentration g/cm³
☐ Dissolved solute concentration g/cm³
☐ Dispersed phase drop diameter cm
☐ Non-condensable gas volume fraction 0

☐ Sediment
☐ Scalars
☐ Relative saturation 0

Use fluid fraction
☐ Fluid fraction 1
☐ Fluid elevation -1.0e+10 cm
☒ Rating Curve cm
☐ Natural inlet, automatic flow regime
☐ Natural inlet, sub-critical flow
☐ Natural inlet, super-critical flow
☐ Natural inlet, critical flow
 Min. elevation:
 Max. elevation:

☐ Electric Charge scoul/cm³
☐ Specified potential boundary
☐ Electric potential g-cm²/sCoul/s²

Turbulence quantities Thermal information

OK Cancel

Figure 3.2 b) Interface of boundary condition of the flume walls

Mesh block 1: Block 1 [X Max Boundary]

Boundary type

☐ Symmetry
☐ Wall
☐ Continutive
☐ Periodic
☐ Specified pressure
☐ Specified velocity
☒ Outflow
☐ Grid overlay
☐ Wave
☐ Volume flow rate

☐ Allow fluid to enter at outflow boundary

☐ Pressure dyne/cm²
☒ Stagnation pressure

☐ Wave absorbing layer
 Length cm
 Damping coefficient at wave approach face 1/s
 Damping coefficient at boundary 1/s
 Background stream velocity User prescribed
 Stream velocity in X 0 cm/s
 Stream velocity in Y 0 cm/s
 Stream velocity in Z 0 cm/s
 Time scale Code calculated s

☐ Density g/cm³
☐ Volume fraction of entrained air
☐ Alloy solute concentration g/cm³
☐ Dissolved solute concentration g/cm³
☐ Dispersed phase drop diameter cm
☐ Non-condensable gas volume fraction 0

☐ Sediment
☐ Scalars
☐ Relative saturation 0

Use fluid fraction
☐ Fluid fraction 1
☐ Fluid elevation -1.0e+10 cm
☒ Rating Curve cm
☐ Natural inlet, automatic flow regime
☐ Natural inlet, sub-critical flow
☐ Natural inlet, super-critical flow
☐ Natural inlet, critical flow
 Min. elevation:
 Max. elevation:

☐ Electric Charge scoul/cm³
☐ Specified potential boundary
☐ Electric potential g-cm²/sCoul/s²

Turbulence quantities Thermal information

OK Cancel

Figure 3.2 c) Interface of boundary condition of the flume outlet

Mesh block 1: Block 1 [Z Min Boundary]

Boundary type

☐ Symmetry
 ☐ Continuitive
 ☐ Specified pressure
 ☐ Grid overlay
 ☐ Wave

☒ Wall
 ☐ Periodic
 ☐ Specified velocity
 ☐ Outflow
 ☐ Volume flow rate

Velocity

☐ X Velocity: cm/s
☐ Y Velocity: cm/s
☐ Z Velocity: cm/s

Pressure

☐ Pressure: dyne/cm²
☒ Stagnation pressure

Wave absorbing layer

☐ Length: cm
☐ Damping coefficient at wave approach face: 1/s
☐ Damping coefficient at boundary: 1/s
☐ Background stream velocity: User prescribed
☐ Stream velocity in X: cm/s
☐ Stream velocity in Y: cm/s
☐ Stream velocity in Z: cm/s
☐ Time scale: Code calculated s

Use fluid fraction

☐ Fluid fraction: 1
☐ Fluid elevation: -1.0e+10 cm
☐ Rating Curve: cm
☐ Natural inlet, automatic flow regime
☐ Natural inlet, sub-critical flow
☐ Natural inlet, super-critical flow
☐ Natural inlet, critical flow
 Min. elevation:
 Max. elevation:

☐ Density: g/cm³
☐ Volume fraction of entrained air:
☐ Alloy solute concentration: g/cm³
☐ Dissolved solute concentration: g/cm³
☐ Dispersed phase drop diameter: cm
☐ Non-condensable gas volume fraction: 0

☐ Sediment
☐ Scalars
☐ Relative saturation: 0

☐ Electric Charge: scoul/cm³
☐ Specified potential boundary
☐ Electric potential: g-cm⁻²/scoul/s⁻²

Turbulence quantities Thermal information

OK Cancel

Figure 3.2 d) Interface of boundary condition of the flume bottom

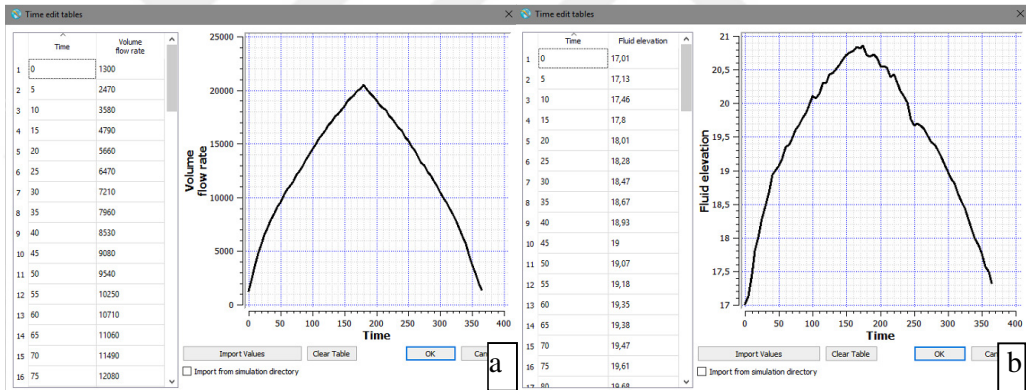


Figure 3.3 a and b Input parameters related to discharge and flow depth

In order to represent the initial conditions of experiments, initial flow depth was set to 17 cm (1 cm above sediment layer) and initial flow velocity was set to 16.25 cm/s. The interface that shows the input parameters of initial conditions is given in Figure 3.4.

Initial

Search for: Find

- Initial
 - Global
 - Velocities

X direction	16,25	cm/s
Y direction	0	cm/s
Z direction	0	cm/s
Turbulent k.e.		erg/g
Fluid rotation about z-axis	0	1/s
Temperature		K
Dispersed phase drop diameter		cm
Uniform non-condensable gas volume fraction	0	
Uniform relative saturation	0	
Electric charge density	0	scoul/cm ³
Uniform alloy solute concentration	0	g/cm ³
Uniform dissolved solute concentration	0	g/cm ³
Volume fraction of entrained air	0	
 - Pressure

Uniform pressure	
Use fluid elevation	
 - Fluid initialization

Initial fluid elevation	17	cm
Initial fluid volume	0	cm ³
 - Wave boundary
 - Void initial state
 - Suspended sediment concentration
 - Fluid Regions

Pointers	
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Figure 3.4 Input parameters of initial conditions

Flow-3D has a rich database of materials and their properties. Choosing “water at 20°C” option loaded default values of the specified material and those values are given at Figure 3.5. Some of those values can be changed if it is desired.

General Physics Fluids Meshing & Geometry Output Numerics

Search for: Find

- Properties
 - Fluid 1
 - Material Name: Water at 20 C
 - Density Properties

Density	1	g/cm ³
Volumetric Thermal Expansion	1	1/K
 - Viscosity

Constant	0,01	g/cm/s
----------	------	--------
 - Thermal Properties

Specific Heat	4,182e+07	erg/g/K
Thermal Conductivity	59700	erg/cm/s/K
Power source per unit mass	0	erg/s/g
Heat transfer to void type 1	0	erg/s/cm ² /K
Heat transfer to void type 2	0	erg/s/cm ² /K
Heat transfer to void type 3	0	erg/s/cm ² /K
e * SB constant - void type 1	0	erg/s/cm ² /K ⁴
e * SB constant - void type 2	0	erg/s/cm ² /K ⁴
e * SB constant - void type 3	0	erg/s/cm ² /K ⁴
 - Solidification Model

Liquidus Temperature	273,25	K
Solidus Temperature	273,15	K
Latent Heat of Fusion	3,35e+09	erg/g
Latent Heat Release Definition	Linearly with temperature	
 - Compressibility

		cm ² /dyne ²
--	--	------------------------------------

Figure 3.5 Input parameters of fluid

Hid33-20 hydrograph was simulated for the pier of 4 cm diameter under RNG, LES and k- ϵ turbulence models. This hydrograph was chosen to minimize the simulation duration. The required maximum turbulence mixing length for RNG and k- ϵ turbulence models can be entered manually or the option of “dynamically computed” can be selected. In this study, since the flow depth changes continuously, the second alternative was used. Input interfaces of turbulence and gravity are given in Figure 3.6 a and b respectively.

Figure 3.6 a) and b) Input parameters related to turbulence model and gravity required by Flow-3D

Sediment scour model requires various parameters to calculate the related equations used by Flow-3D. Critical Shields parameter was calculated from Shields diagram as 0.038. Shield diagram acquired from Liu (2001) is given in Figure 3.7. The frequently used Meyer-Peter & Müller equation was chosen to predict the bed load transport. As suggested in Flow-3D manual, the bed load and entrainment coefficients were taken as 8 and 0.018 respectively. Figure 3.8 shows the sediment module interface of Flow-3D. The following default values are used in numerical solution (Flow-3D v11.2 Manual):

- The Angle of Repose describes the maximum resting angle of the bed (typically between 30 and 40 degrees), and is used to modify the local critical

Shields parameter for slope effect. Angle of repose value of the sediment was measured as 32 degrees.

- The Richardson-Zaki coefficient multiplier controls the effect of drag on the settling sediment particles as they become concentrated. The default value is 1.0, which means that the enhancement to the drag due to particle-particle interaction is as defined by the Richardson-Zaki model.
- The Maximum packing fraction controls the maximum solid fraction when sediment is settled and packed within the domain. This value is assumed to be the same, regardless of the arrangement of sediment species. The default value is 0.64, which is the maximum packing fraction of spheres. For multidisperse sediments, this number can be higher. For monodisperse, irregular sediment particles, it can be lower.
- The packed sediment bed roughness is defined by the d_{50} of the sediment material and the bed roughness / d_{50} ratio. This values was left 1 as default.

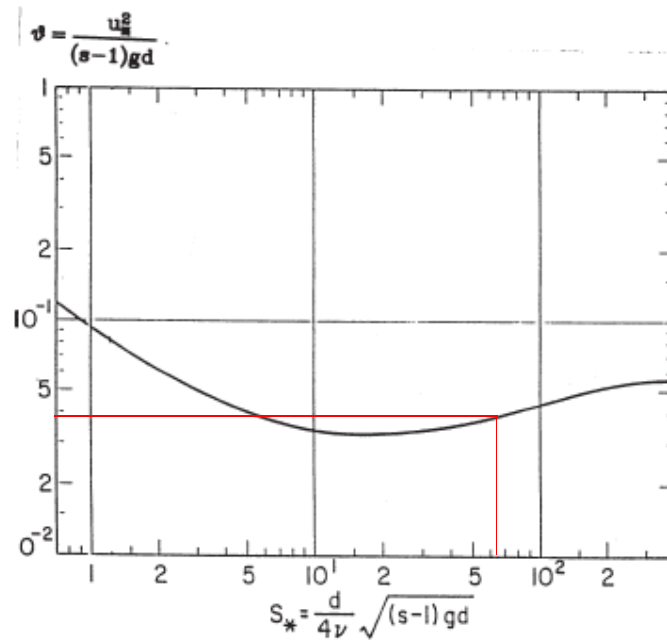


Figure 3.7 Shields diagram (Liu, 2001)

$$S_* \text{ was calculated as } S_* = \frac{0.00163 \sqrt{(2.65-1) \cdot 9.81 \cdot 0.00163}}{4 \cdot 10^{-6}} \cong 66 \text{ from the Shields}$$

diagram, the critical Shields parameter (θ_c) was found approximately as 0.038.

Activate sediment scour model

Critical Shields Number definition

☒ Prescribed value

☐ Calculate value using Soulsby-Whitehouse equation

Bed Load Transport Rate equation

☒ Meyer-Peter & Müller equation

☐ Nielsen equation

☐ Van Rijn equation

Richardson-Zaki coefficient multiplier: 1

Molecular diffusion coefficient: 0 g/cm/s

Turbulent diffusion multiplier: 0

Local adjustments of Critical Shields Number

☐ For slope effect

Number of sediment species: 1

Maximum packing fraction: 0.6400

Bed roughness / d50 ratio: 1

	Name	Diameter	Density	Critical Shields Number	Entrainment Coefficient	Bed Load Coefficient	Re
Sediment 1	Sand	0.163	2.65	0.038	0.018	8	32

Repose Angle

Figure 3.8 Interface of sediment module

CHAPTER FOUR

EXPERIMENTAL STUDIES

4.1 Experimental Set-up

Experiments were carried out in a tilting flume 18.6 m long, $B=0.8$ m wide and 0.75 m deep constructed in the Hydraulic Laboratory of Dokuz Eylul University Civil Engineering Department, within the scope of the project TÜBİTAK 106M274. The channel bottom slope was equal to 0.006. General and schematic views of the flume are given in Figure 4.1a and 4.1b.

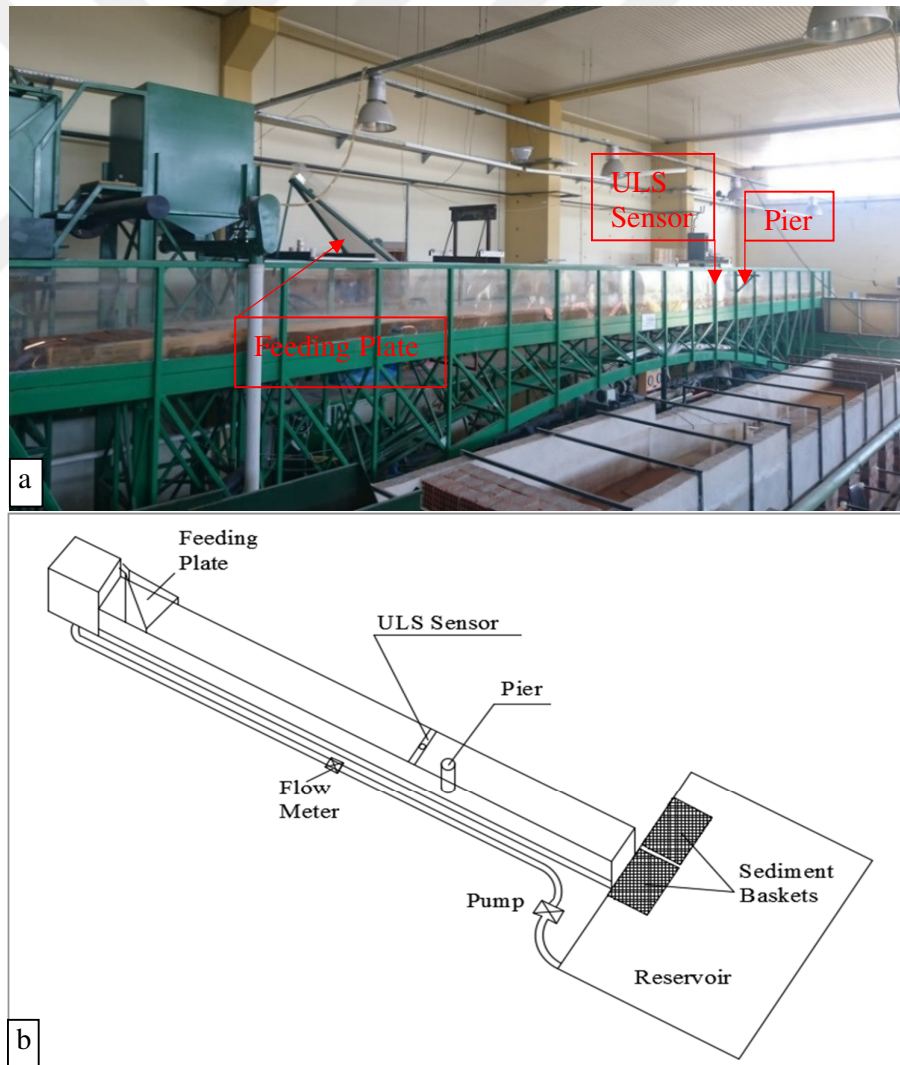


Figure 4.1 a) and b) General and schematic views of the flume (Personal archive, 2018)

Three different circular piers were used during the experiments with 4 cm (D4), 8 cm (D8) and 11 cm (D11) diameters, placed at 12th meter of the flume. The first 8 and the last 5 meters of the flume were filled with 20 cm thick gas concrete blocks. After the 3rd meter of the flume, the flume was covered with sediment having 1.63 mm median diameter (d_{50}) and 1.303 geometric standard deviation (σ_g) to form a bed of 25 cm. The granulometric curve of the bed material is plotted in Figure 4.2.

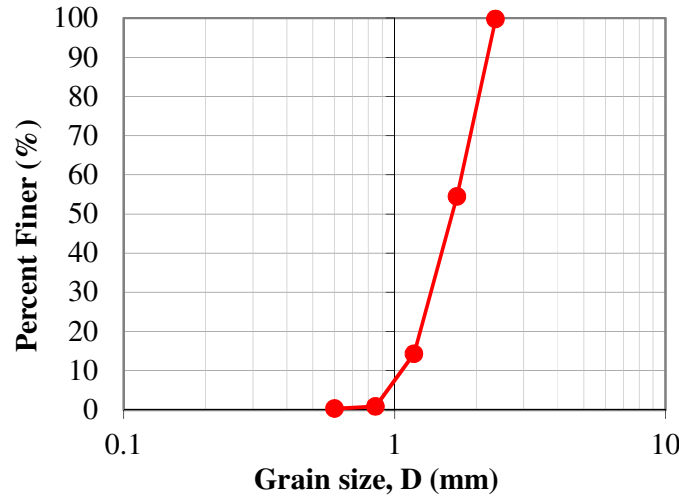


Figure 4.2 Granulometric curve

Water flows to the flume by means of a pump with 100 L/s discharge capacity, connected to the speed control unit. The speed control unit regulates the pump's revolution speed to desired value within desired time with the aid of the software called DriveLink-C (Figure 4.3a and Figure 4.3b).

Water returns to a reservoir of 27 m³ volume located below the downstream of the flume and pumped again to the flume simultaneously. Discharge was measured by an electromagnetic flow meter (OPTIFLUX 1000 manufactured by Krohne) mounted on the pipe before inlet of the channel and time dependent discharge values (L/s) which appear on the screen were recorded by a HD camera (Figure 4.3c).

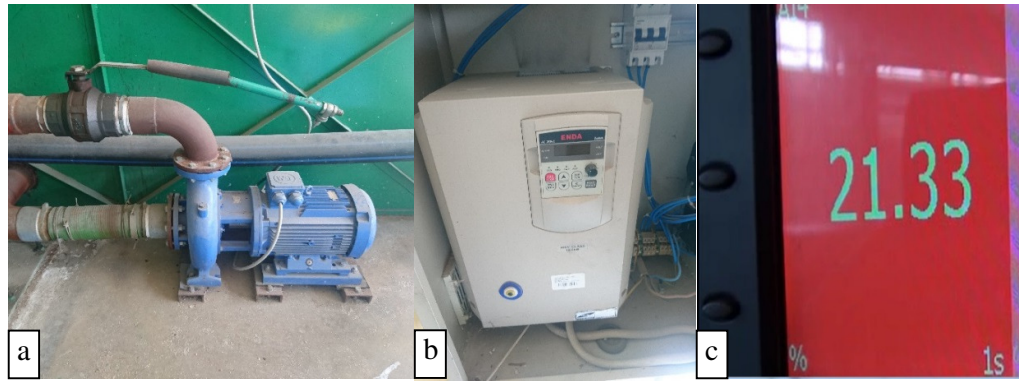


Figure 4.3 a) Pump b) Velocity control device c) Discharge value read from screen in L/s (Personal archive, 2018)

Scour depth around the bridge pier was measured by Ultrasonic Velocity Profiler (UVP) device. As also stated in Guney *et al.*, (2012), it's possible to determine the bed morphology by using UVP which is normally designed for velocity measurement. The scour depth was determined from echo amplitude values by placing UVP transducers vertically. Transducers with 4 MHz emitting frequency which have the highest emitting frequency available in laboratory were used in experiments. Guney *et al.* (2012) stated that high emitting frequency provides high spatial resolution.

There are three transducers used in this study. One at the upstream of the pier (T2) and two at the flanks of the pier (T1 and T3). Placement of UVP transducers is illustrated in Figure 4.4a. Figure 4.4b represents the UVP device and its transducers.

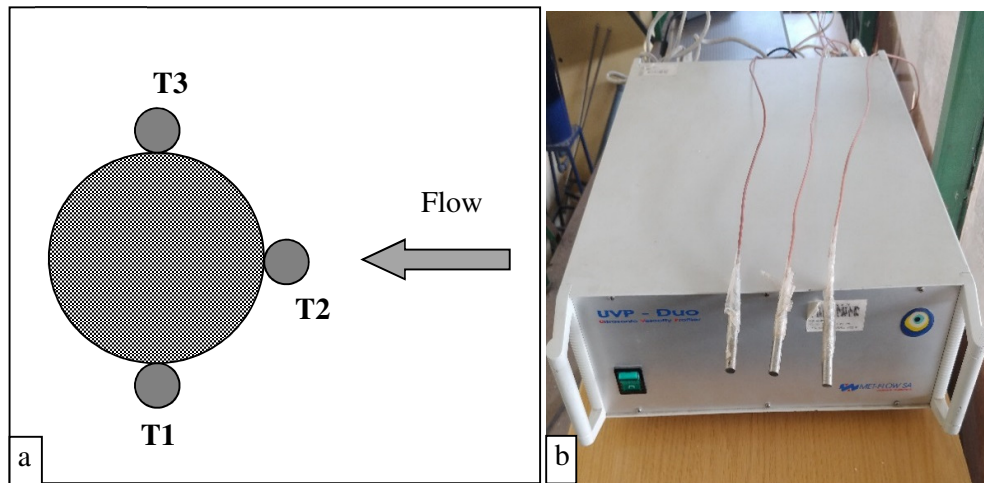


Figure 4.4 a) Placement of transducers b) UVP device and its transducers (Personal archive, 2018)

Flow depth was measured by UltraLab ULS (Ultrasonic Level Sensor) manufactured by General Acoustics. ULS sensors measure flow surface simultaneously with ± 1 mm precision. Difference between flow and bed surfaces gives the flow depth. The sensor was placed at 11th meter of flume. Approaching flow depth was measured at one meter from the pier, accordingly to the criterion $(B-b)/2$, suggested by Oliveto & Hager (2002).

The sediment was collected by means of the baskets located at the end of the flume (Figure 4.5a). Each basket was replaced with another at intervals of 1 min. Sediment was fed into the flume manually by means of the sediment feeding plate designed to provide uniform feeding. The sediment was introduced to the channel by two different size of cups containing 280 ± 10 g and 560 ± 10 g sediment (Figure 4.5b). In order to determine the amount of sediment to be used in feeding, a two-phased approach was adopted. First, all the experiments were conducted without sediment feeding for 7 hydrographs, total sediment load was collected minute by minute, dried and weighed. A regression analysis was performed to obtain a relationship between the discharge and the sediment load. Then, the experiments were carried out by feeding with amounts predetermined from this analysis.



Figure 4.5 a) Sediment baskets b) Sediment feeding plate and cups (Personal archive, 2018)

4.2 Experimental Method

Before beginning the experiments, the following preparations were realized:

- Before leveling the sediment in the flume, a base flow was conveyed to the flume to get sediment totally saturated. Keeping sediment in same condition before each experiment is necessary to make a sound comparison.
- After a 20 mins of water drainage, whole sediment in the flume was leveled carefully by means of a scrapper plate moving on the rails over the flume walls (Figure 4.6).
- Pier was mounted at the center line of the 12th meter of the flume. ULS sensor was placed at 11th meters of the flume.
- Since the discharge delivered by the pump is affected from the level of water in the supply reservoir, the level of water in the reservoir was aligned to obtain same the discharge to be delivered by the pump.

The view of flume before beginning of the experiment is given in Figure 4.7.



Figure 4.6 Scrapper Plate (Personal archive, 2018)



Figure 4.7 View of flume before beginning of experiment (Personal archive, 2018)

4.3 Generated Hydrographs

16 different triangular hydrographs were formed with different characteristics, as indicated in Table 4.1. The name of the hydrograph ($H_{i,j-k}$) was defined so that the subscripts i and j denote rising and falling durations in minutes respectively and the following subscripts k indicates the peak discharge of the hydrograph (L/s).

Table 4.1 Hydrographs with base flows, peak discharges and flood durations

Hydrograph	Base Flow (L/s)	Peak Discharge (L/s)	Flood Duration (min)	
			Rising	Falling
Hid33-20	1.5	20	3	3
Hid33-31	1.5	31	3	3
Hid36-31	1.5	31	3	6
Hid39-31	1.5	31	3	9
Hid33-38	1.5	38	3	3
Hid55-23	1.5	23	5	5
Hid55-28	1.5	28	5	5
Hid55-31	1.5	31	5	5
Hid55-33	1.5	33	5	5
Hid55-38	1.5	38	5	5
Hid55-44	1.5	44	5	5
Hid1010-28	1.5	28	10	10
Hid1010-33	1.5	33	10	10
Hid1010-44	1.5	44	10	10
Hid1515-44	1.5	44	15	15
Hid2020-44	1.5	44	20	20

The discharge-time hydrographs $Q(t)$ with fed sediment graphs $q_s(t)$ and corresponding water depth-time hydrographs $y(t)$ are given in Figure 4.8.

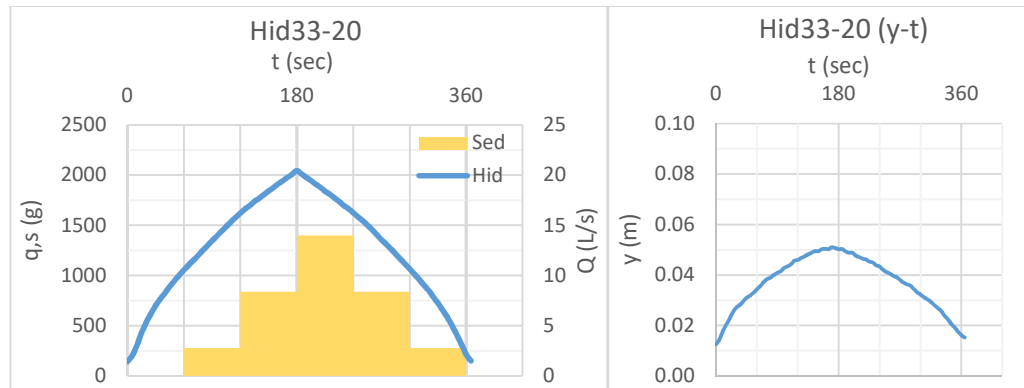


Figure 4.8 Water discharge-time, sediment amount-time and water depth-time hydrographs

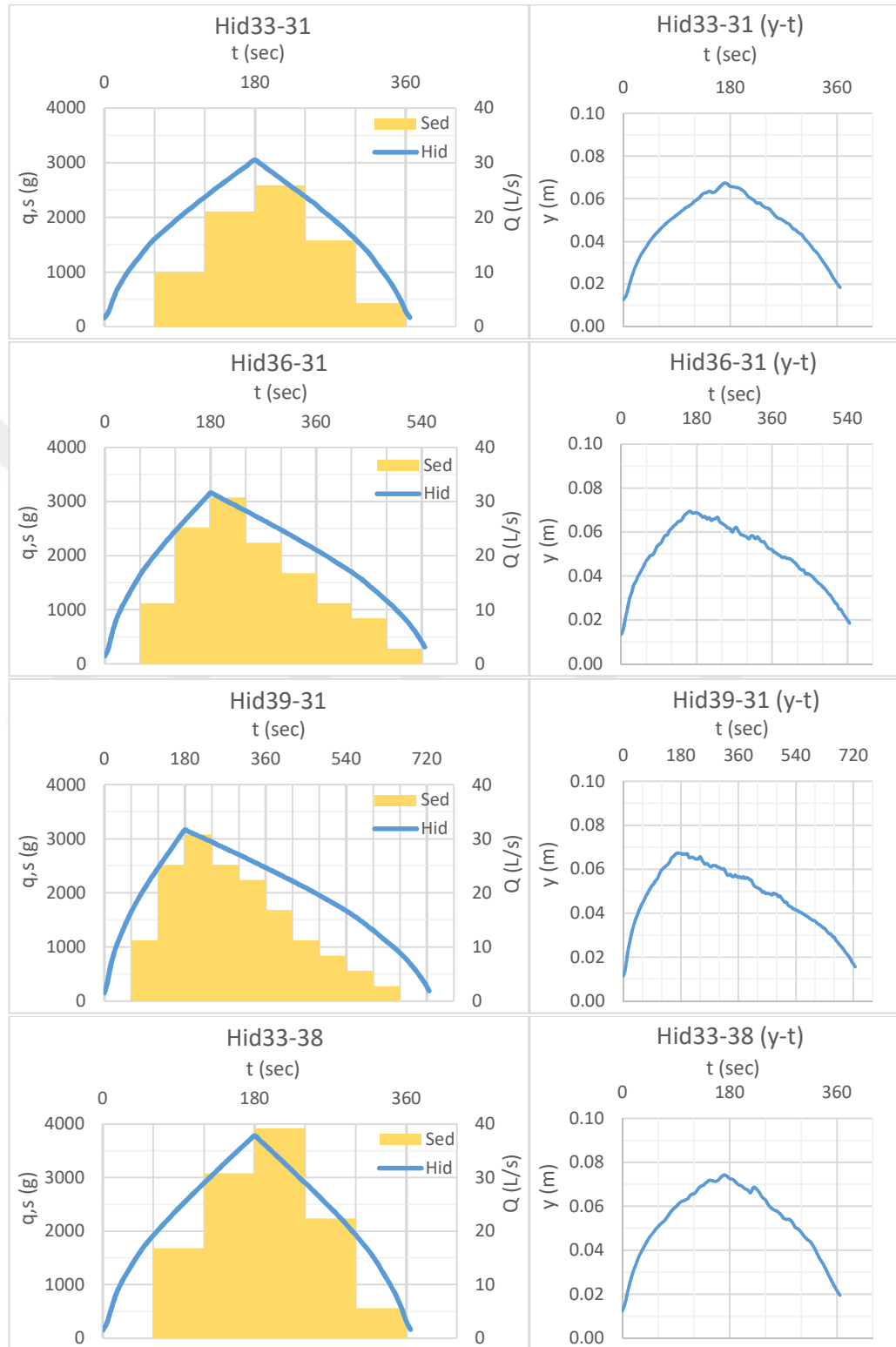


Figure 4.8 continues

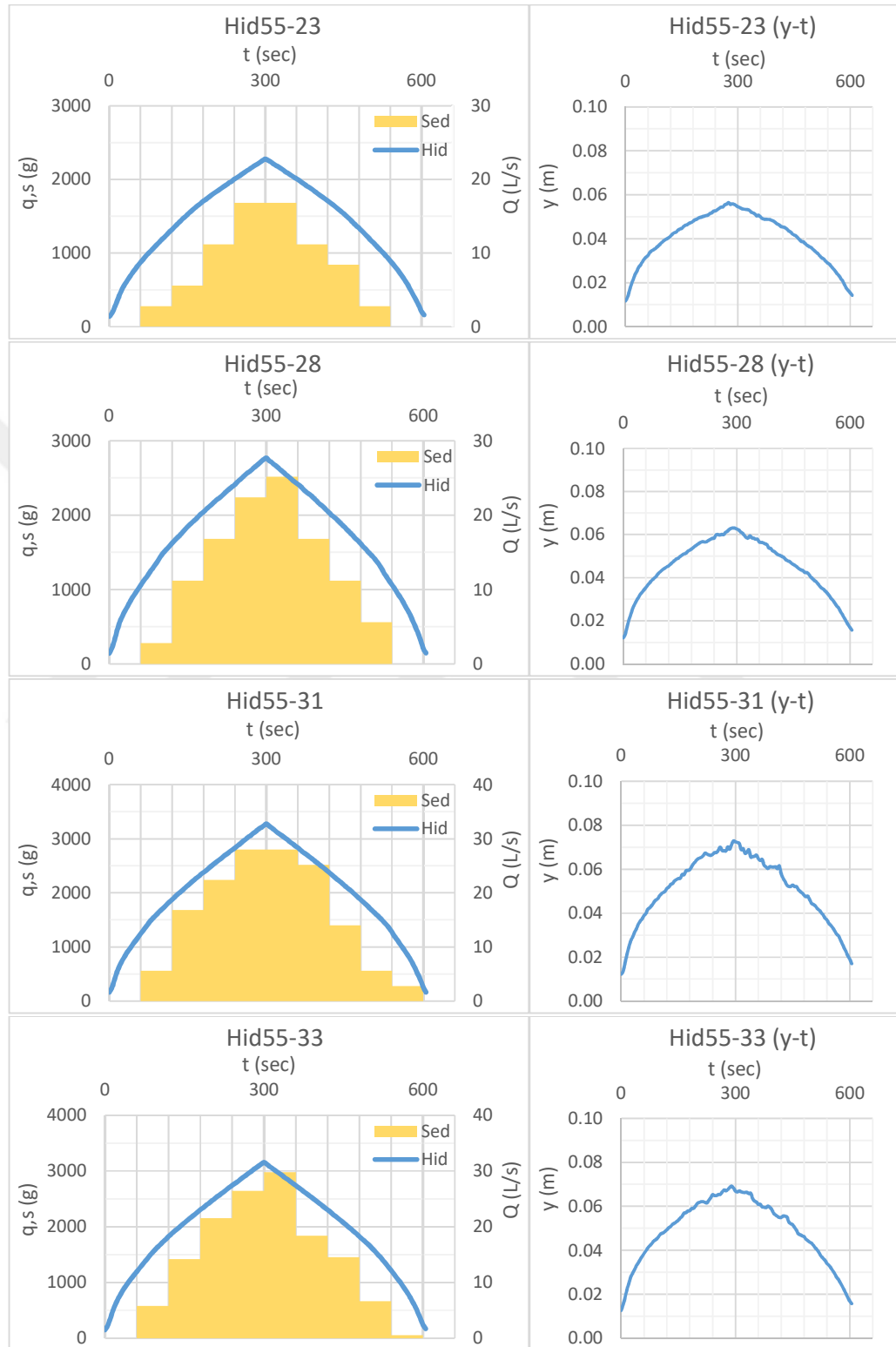


Figure 4.8 continues

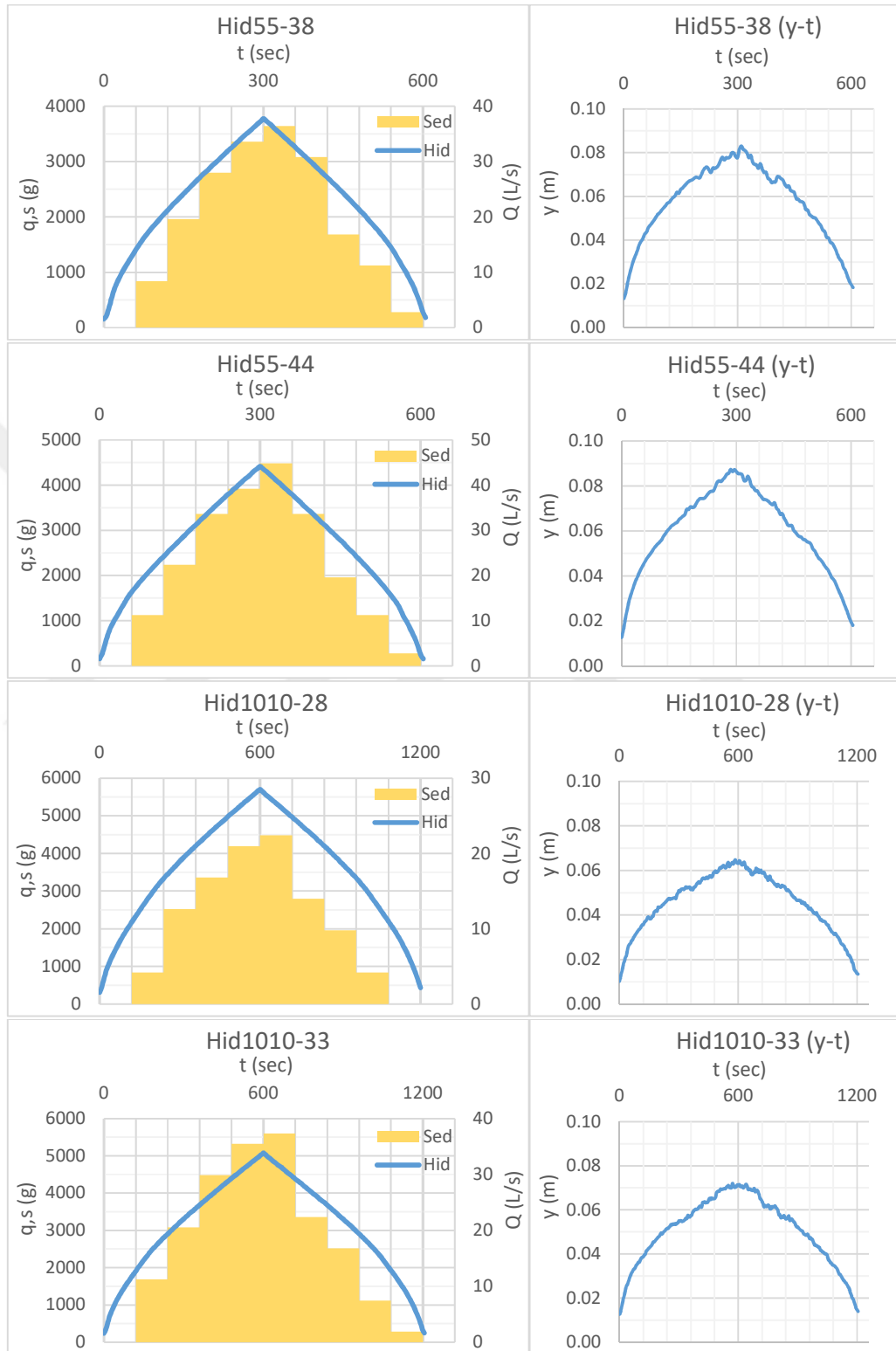


Figure 4.8 continues

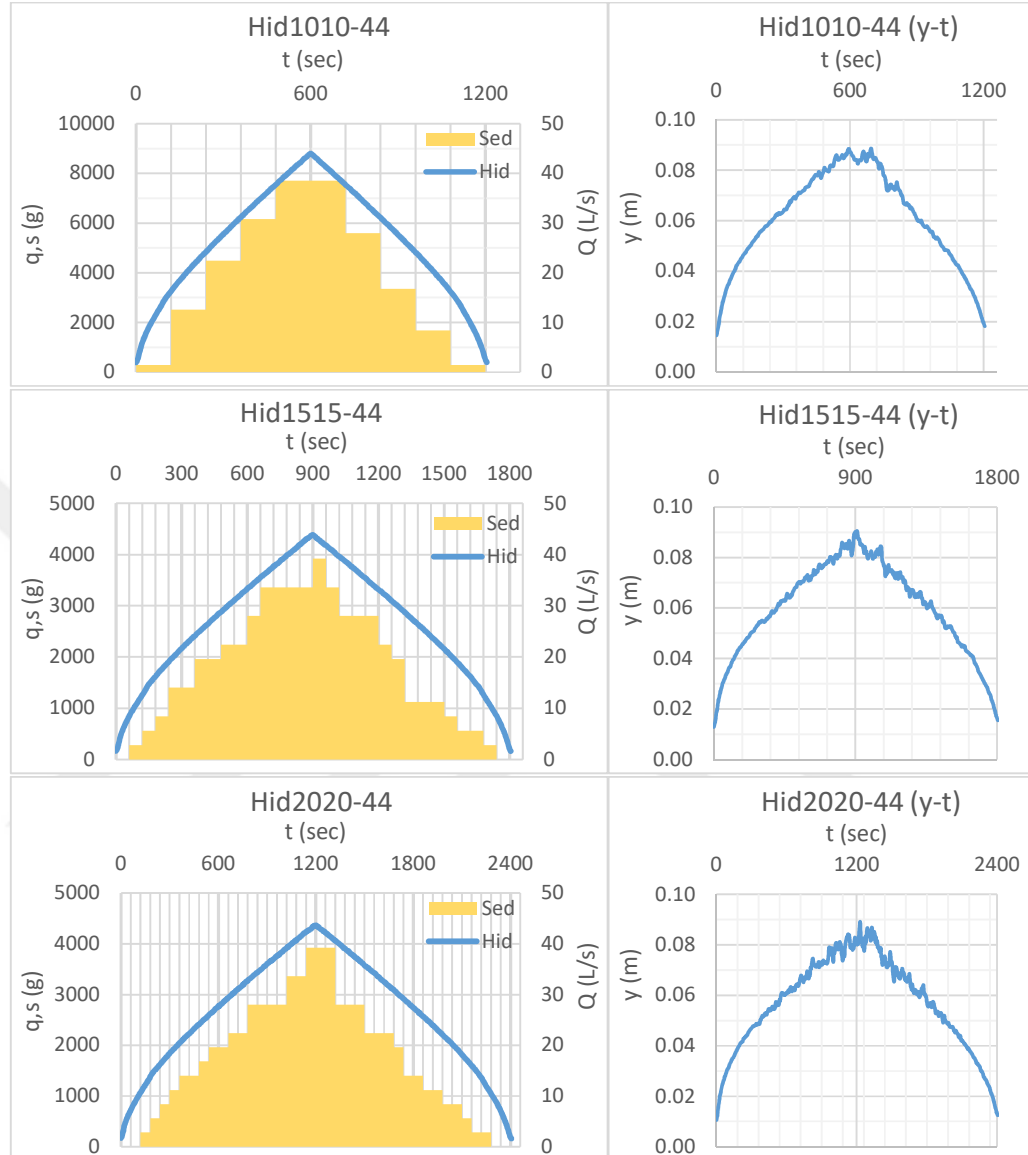


Figure 4.8 continues

During the experiments, the followed steps are given below:

- Desired hydrograph was generated by running the related software and chronometer was started simultaneously at time 00:00 (minute : second).
- At time 00:30, ULS software was commenced to measure bed level, since the measured bed level is necessary to determine the flow depth.
- Approximately at time 04:00, water entered the flume. Metal plates were used to prevent the scour caused by this initial flow. Any deformation around the pier was put right by means of a trowel.

- Approximately at time 07:00, water reached the upstream end of the flume and steady state conditions occurred. No scouring was observed during the base flow. A camera started to record the events around the pier during the experiment.
- At time 10:00 hydrograph started and UVP software was activated. UVP transducers were started to measure flatbed level, since no scour has started yet. Flood wave reached the pier with a delay of 25-30 secs, according to the hydrograph used.
- Sediment was supplied into the flow according to the prescribed sediment graph. Sediment was collected from baskets at the end of the flume minute by minute. Figure 4.9 represents collected and fed sediment amounts, in the case of various hydrographs. $W_{s,c}$ and $W_{s,f}$ indicate the weights of collected and fed sediment respectively.
- UVP and ULS software continued to collect data through 1 min after the flood wave ended.

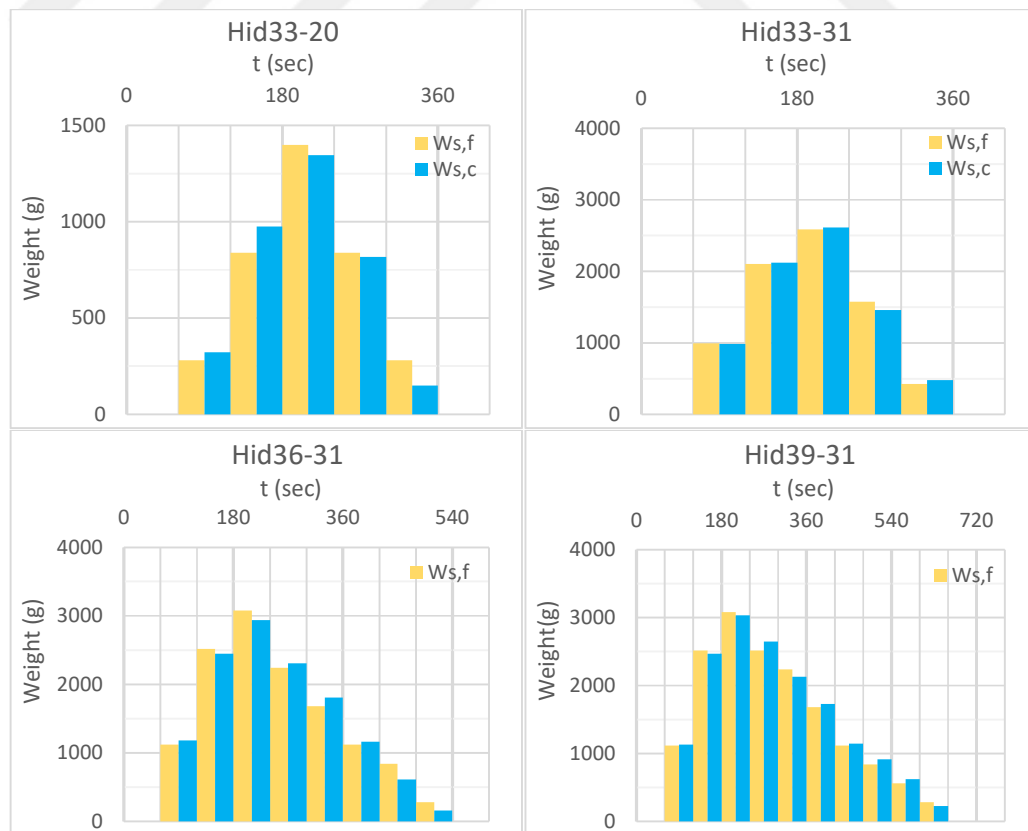


Figure 4.9 Weights of collected and fed sediment

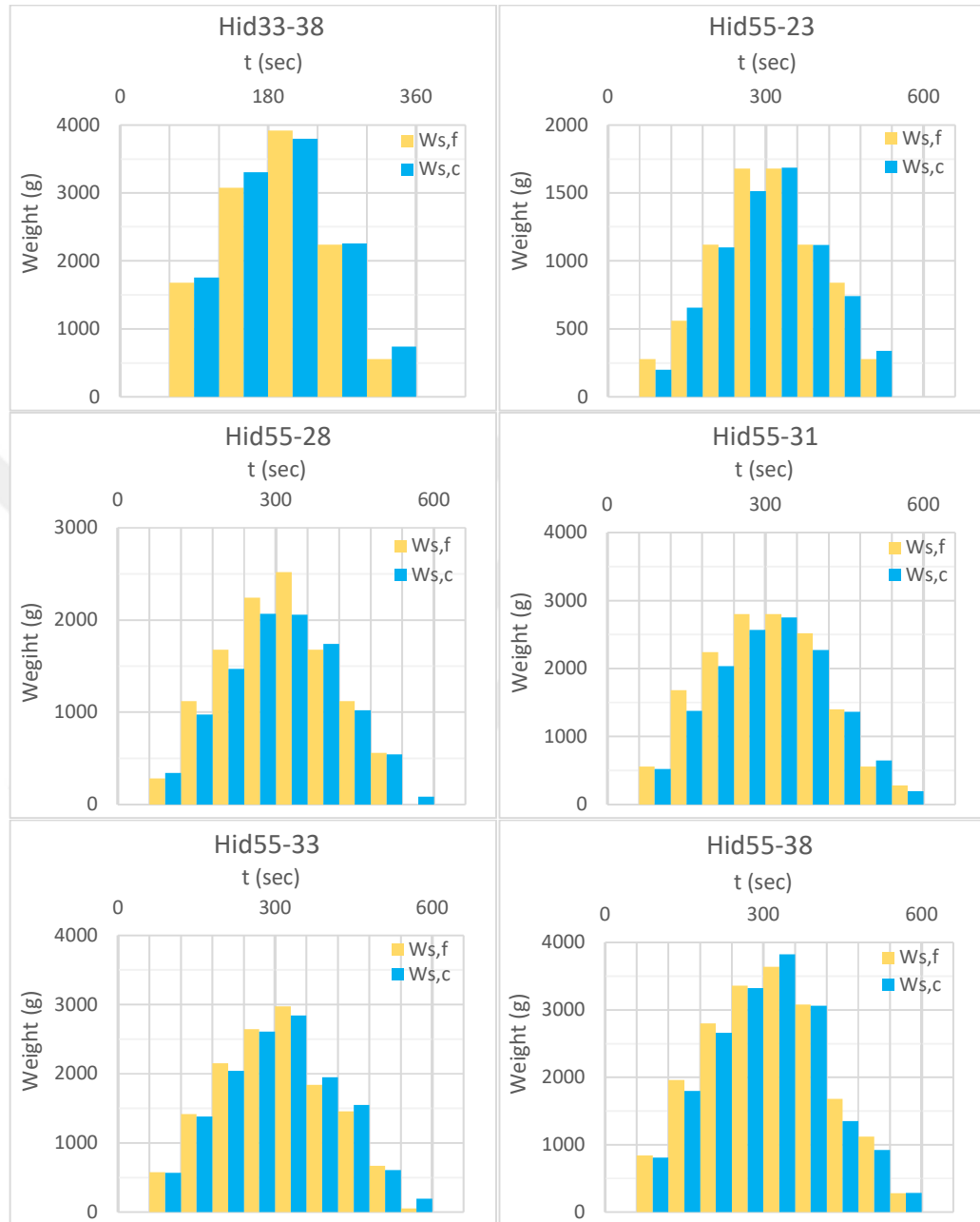


Figure 4.9 continues

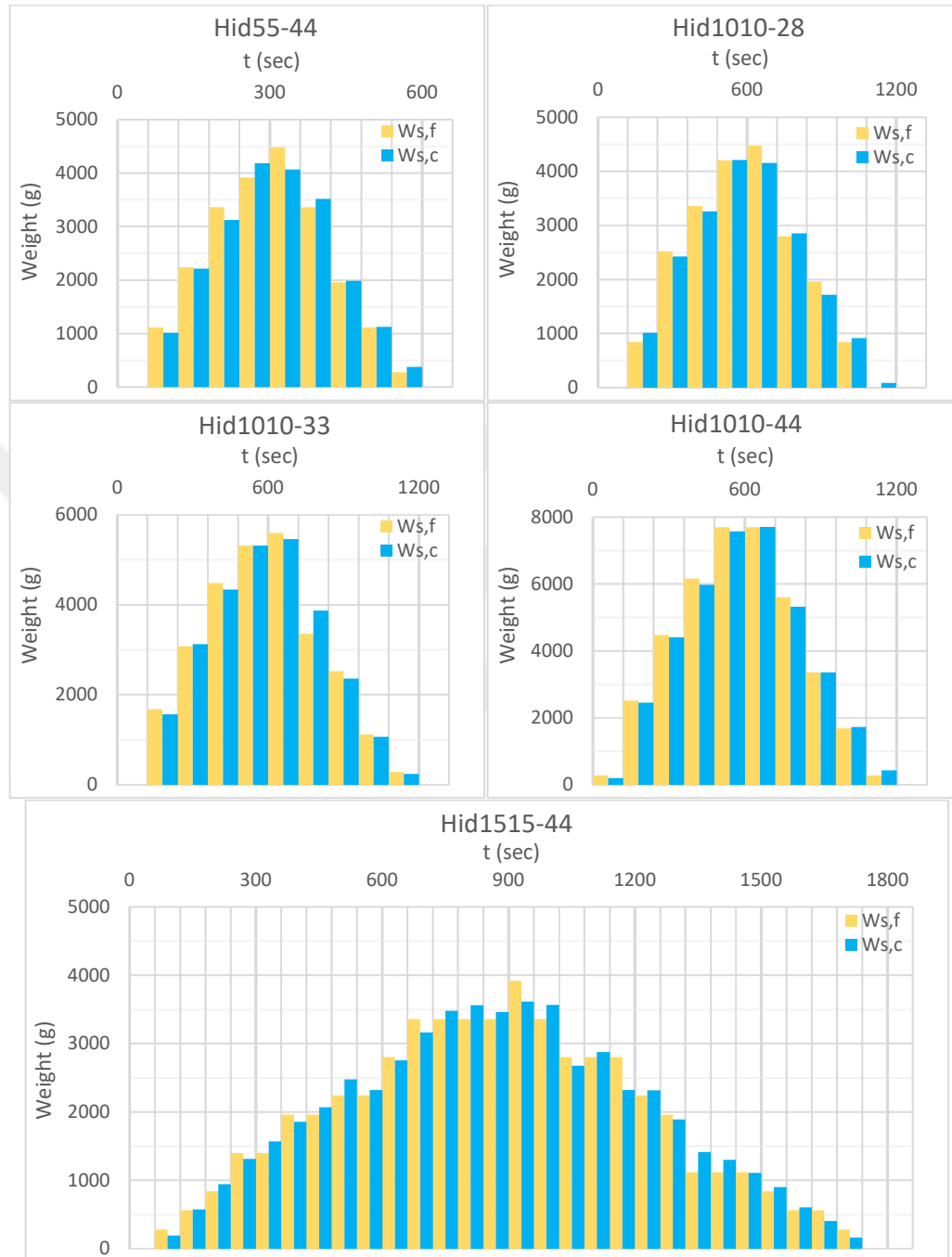


Figure 4.9 continues

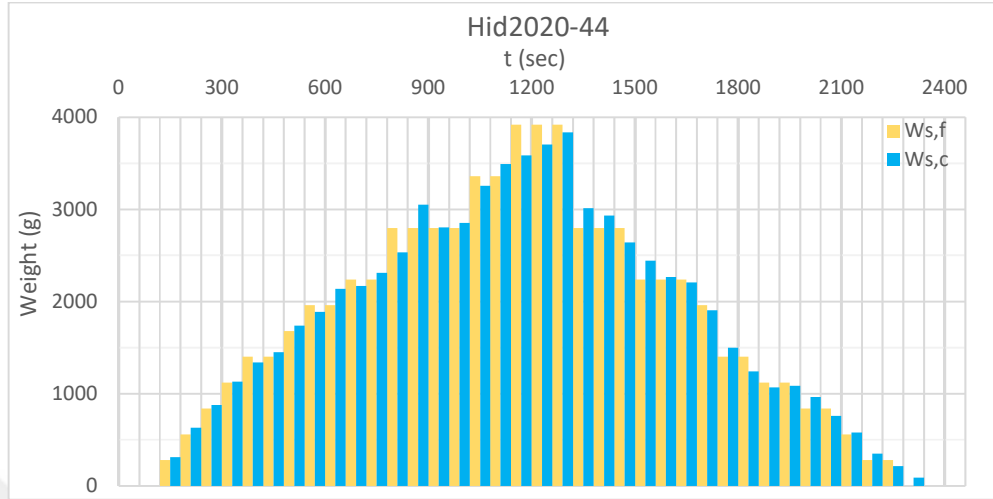


Figure 4.9 continues

Once the experiment was completed, the following tasks were performed:

- Photographs of the flume and the scour hole were taken.
- The scour hole was profiled with a laser meter through the center of the flume.
- The sediment collected from each basket was spread over sheets on the floor separately. The day after, dried sediment was weighed and used for feeding in the next experiment.

At the end of the experiment, raw data gathered from UVP device were analyzed. Ultrasonic beams emitted from transducer cause high echo amplitude value on the measurement axis when it contacts with bed surface (Guney et al., 2012). Location of the first peak of echo amplitude gives the distance between transducer and bed surface, regardless of its sign. Figure 4.10 represents an echo graph of T2 transducer mounted on the 8 cm pier, at 100th second of Hid1010-33 hydrograph, as an illustrative example. In this figure, the horizontal axis represents the echo distance (scour depth), and the velocity values are given on the vertical axis. This raw data also obtained as text file and processed on MATLAB software in order to transform it to a more evaluable aspect. The time dependent scour depth obtained from the MATLAB output that belongs to the same experiment, pier and transducer is given in Figure 4.11, as an example. In this figure, the point A corresponds to the records given in Figure 4.10.

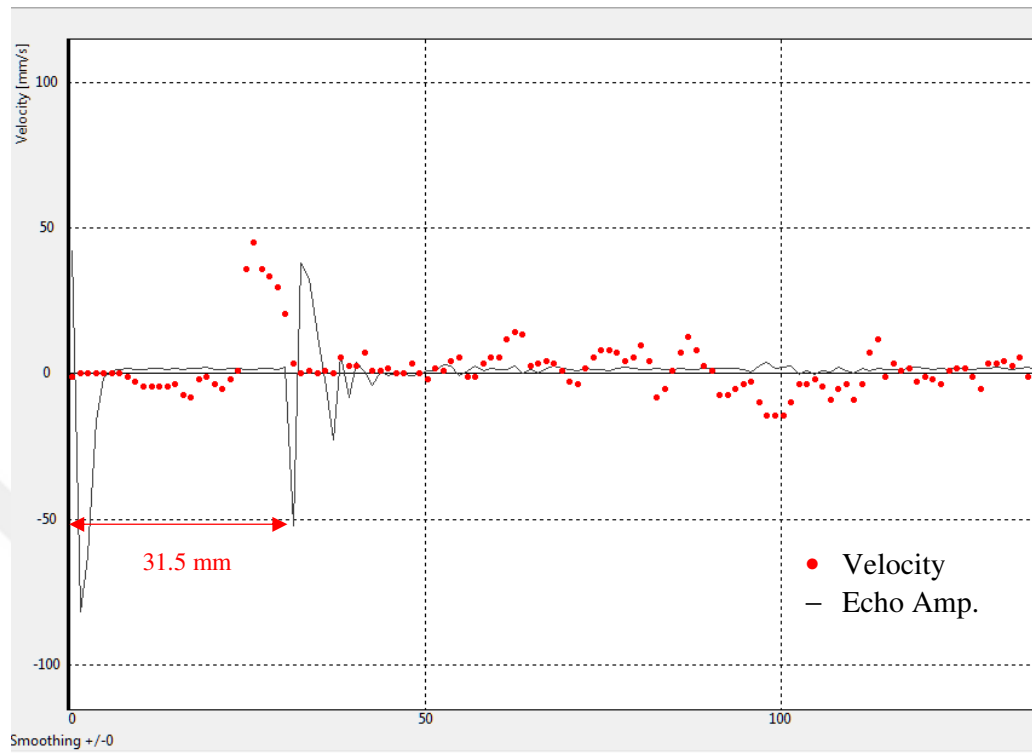


Figure 4.10 UVP records at 100th seconds obtained from T2 transducer

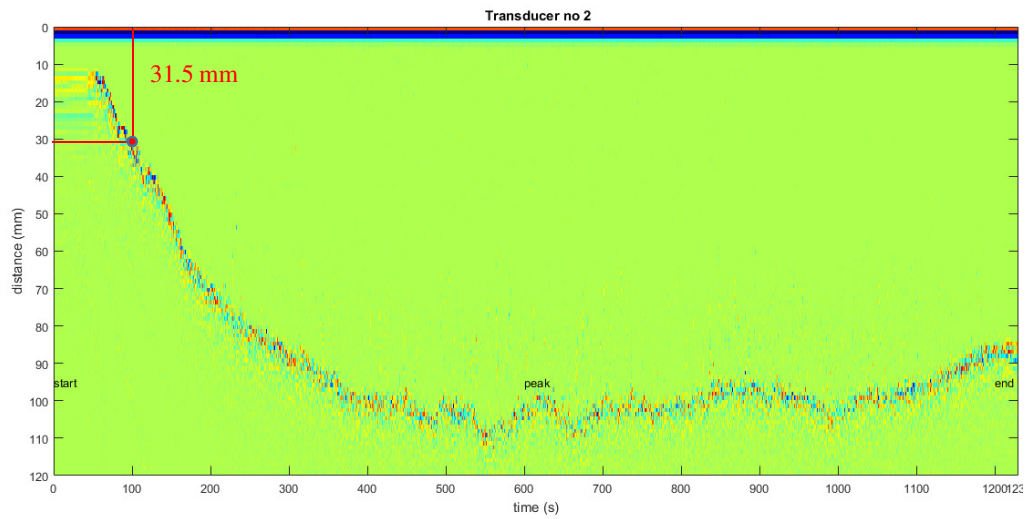


Figure 4.11 Time dependent scour depth obtained from MATLAB software

CHAPTER FIVE

EXPERIMENTAL FINDINGS AND DISCUSSIONS

16 different triangular hydrographs described in Chapter 3 were used during the experiments. All hydrographs were performed second time in order to verify the former one. A third repetition was realized if found necessary.

5.1 Experimental Observations

During the base flow, no scour had been observed as it was intended. After ten minutes of the base flow, water level in the flume began to rise. Flow filaments were evidently separated into two directions at the flanks of the pier and wake region began to develop. Two separate scour holes began to form at the flanks of the pier and met in front on the pier a few seconds later. The development of the scour process at different instants of the experiment for the 11 cm diameter pier in the case of the hydrograph Hid55-31 is depicted in Figures 5.1 a to 5.1 i, as illustrative example. Fig. 5.1 a corresponds to 10th sec of the flood and the subsequent figures are given with 2 sec intervals.

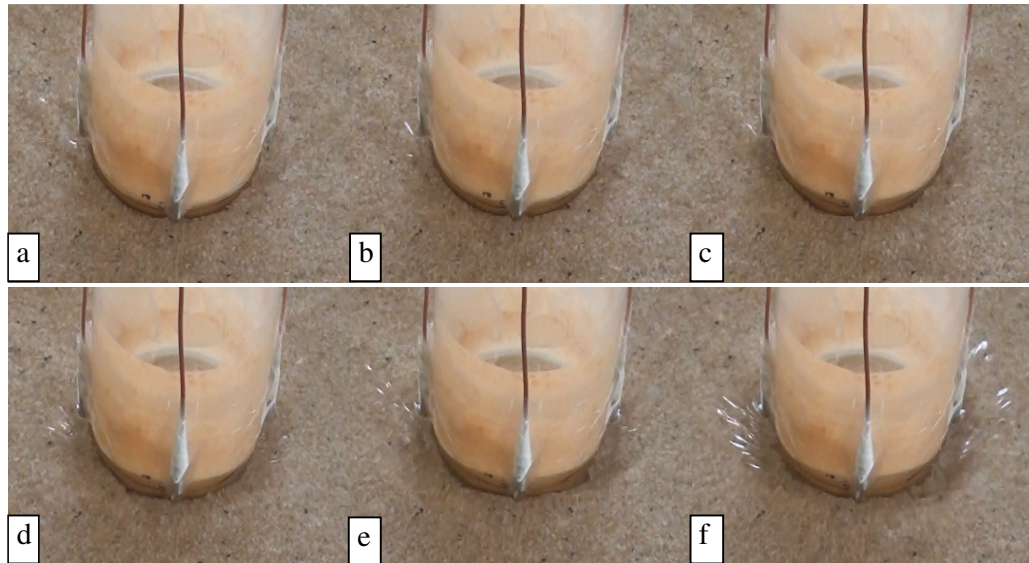


Figure 5.1 a) to 5.1 i) Development of initial scour (Personal archive, 2017)

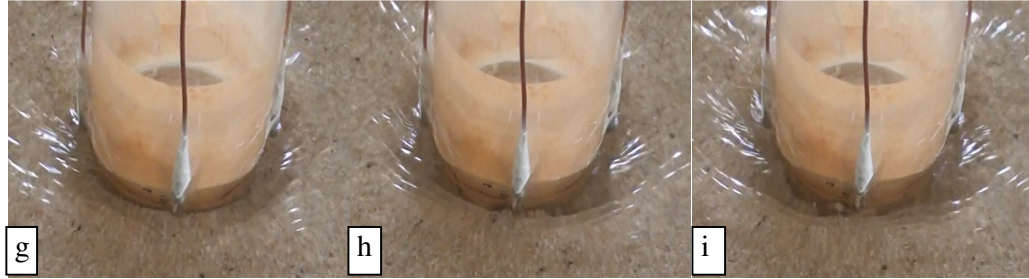


Figure 5.1 continues

With the increasing discharge, development of horseshoe vortices enhanced the size of scour hole rapidly and scour hole took shape of a frustum. After the flow velocity reached its critical value, bed material began to supply into the scour hole. Since the flow velocity increased continuously and flow rate was higher than sediment transport rate, scour depth increased continuously with a relatively low fluctuation until the peak discharge. The bed material removed from the scour hole was drifted to downstream 3-4 pier diameters away from pier, by wake vortices.

As the falling limb started, fluctuations became clearer, especially at high velocities. With the decreasing flow velocity, flow rate lost its advantage against sediment transport rate and sediment transport became dominant element on scour depth. Sediment clusters continued to supply into the scour hole and decreasing flow velocity caused lesser amount of sediment to move out of the scour hole. As the consequence of this process, scour depth began to decrease. Thus, a final scour depth lower than maximum scour depth occurred. This process is illustrated in Figure 5.2 a, b, c and d.

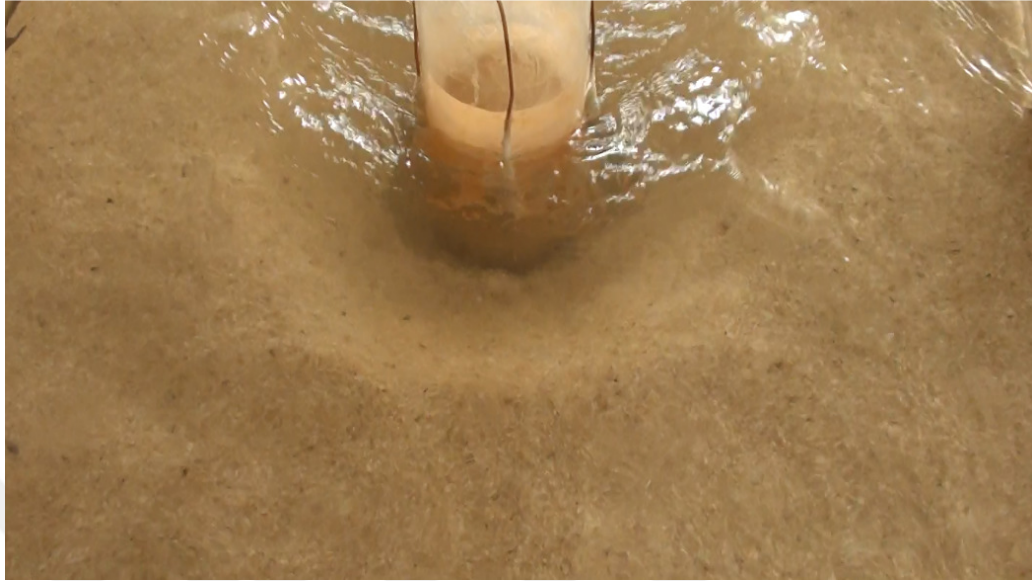


Figure 5.2 a) Scour hole of the D11 pier on Hid2020-44 at 10 mins of the flood (Personal archive, 2018)



Figure 5.2 b) Scour hole of the D11 pier on Hid2020-44 at 20 mins (peak) of the flood (Personal archive, 2018)

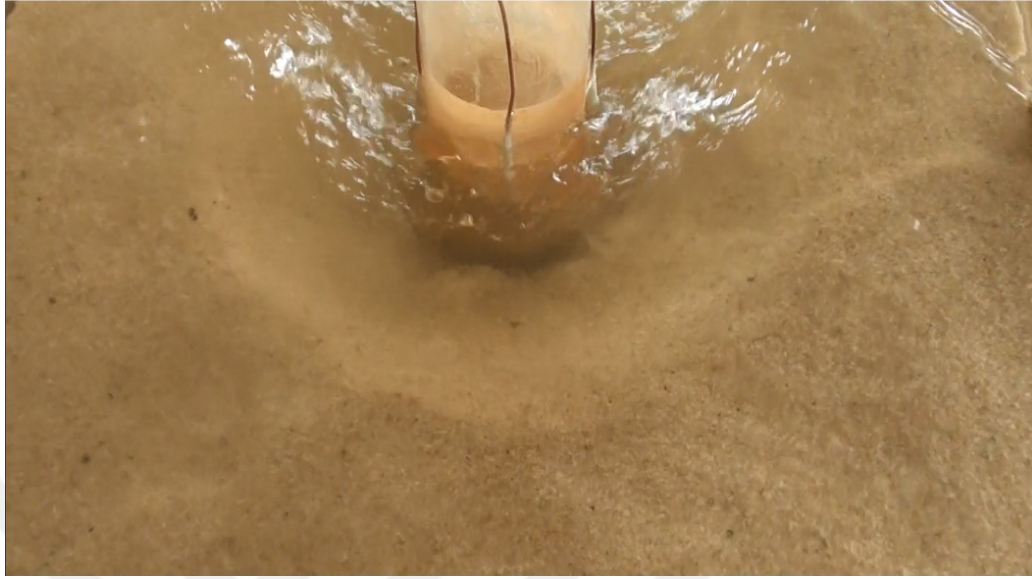


Figure 5.2 c) Scour hole of the D11 pier on Hid2020-44 at 30 mins of the flood (Personal archive, 2018)



Figure 5.2 d) Scour hole of the D11 pier on Hid2020-44 at 40 mins (end) of the flood (Personal archive, 2018)

5.2 Evaluating Experimental Results

Time dependent scour depths for various piers and hydrographs recorded from three transducers are given in Figure 5.3.

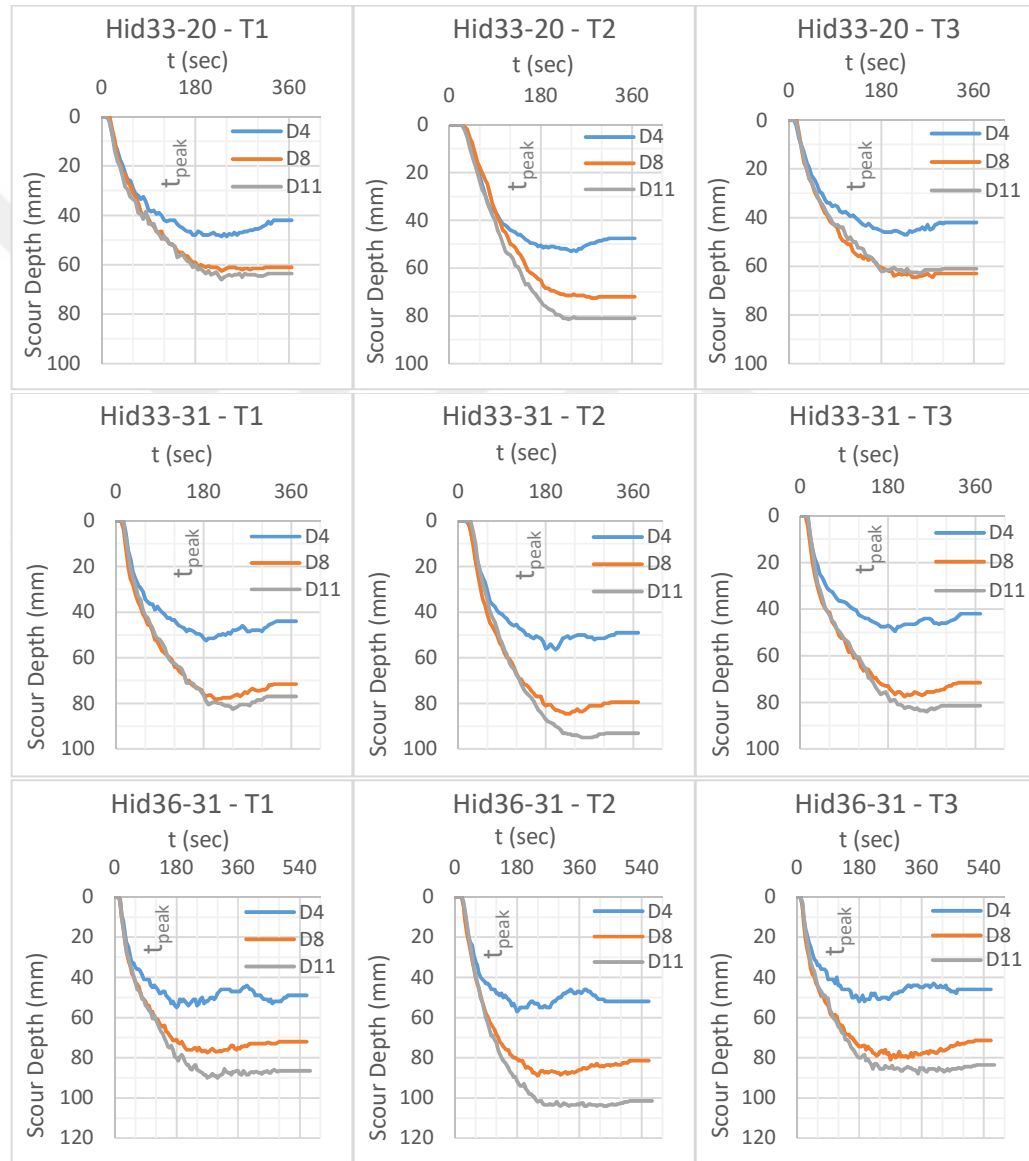


Figure 5.3 Time dependent scour depths for each transducer and hydrograph

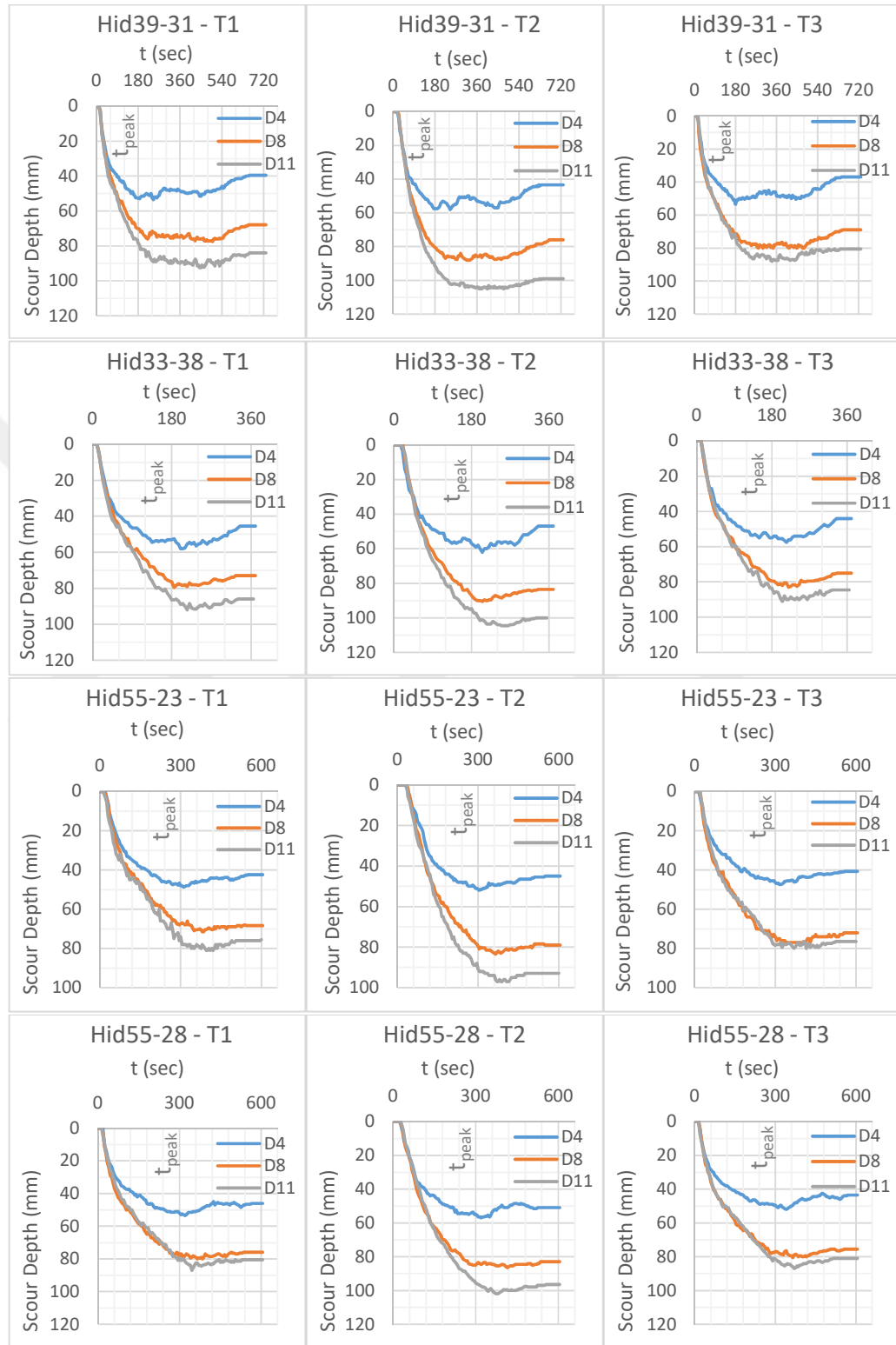


Figure 5.3 continues

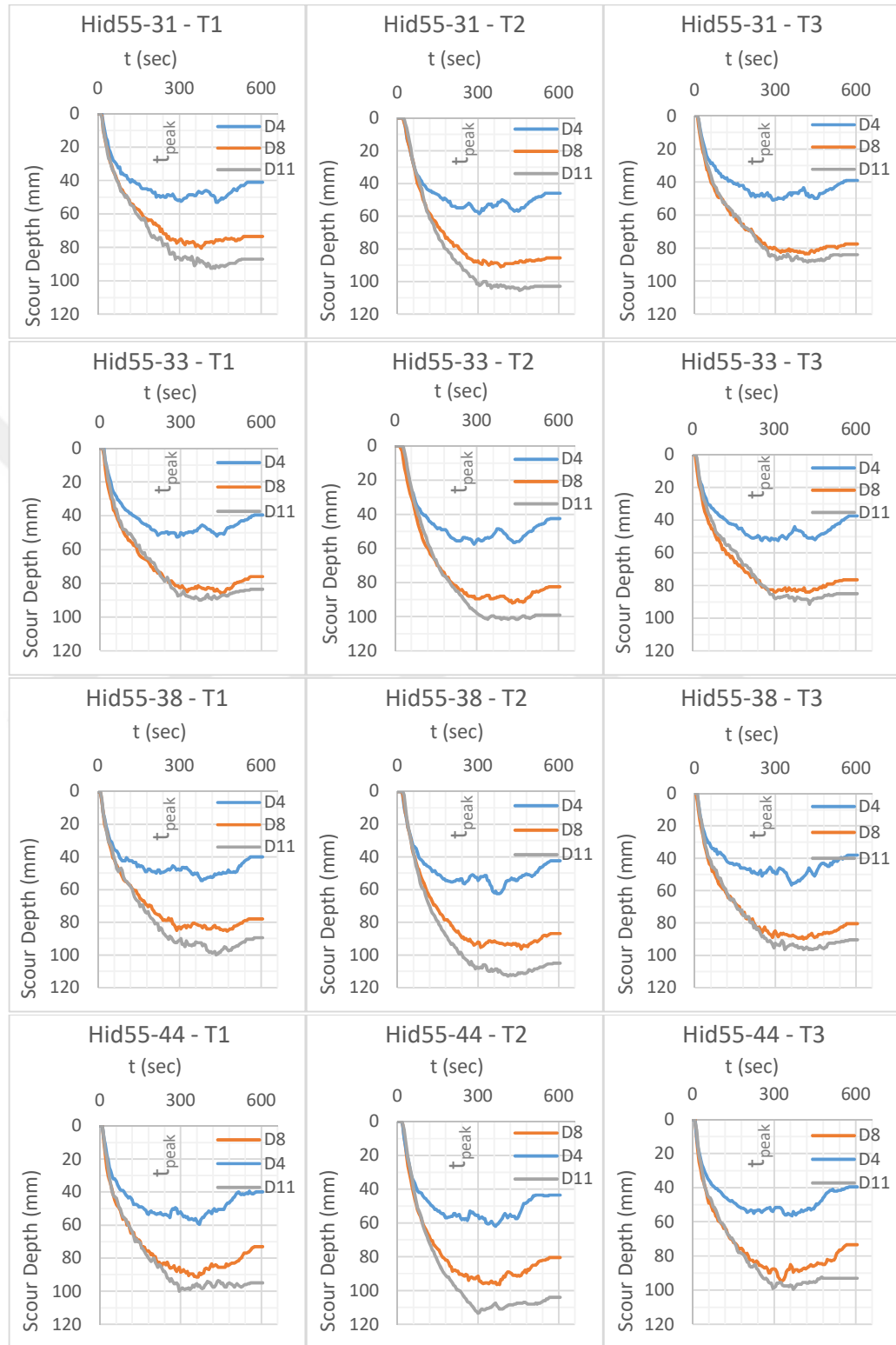


Figure 5.3 continues

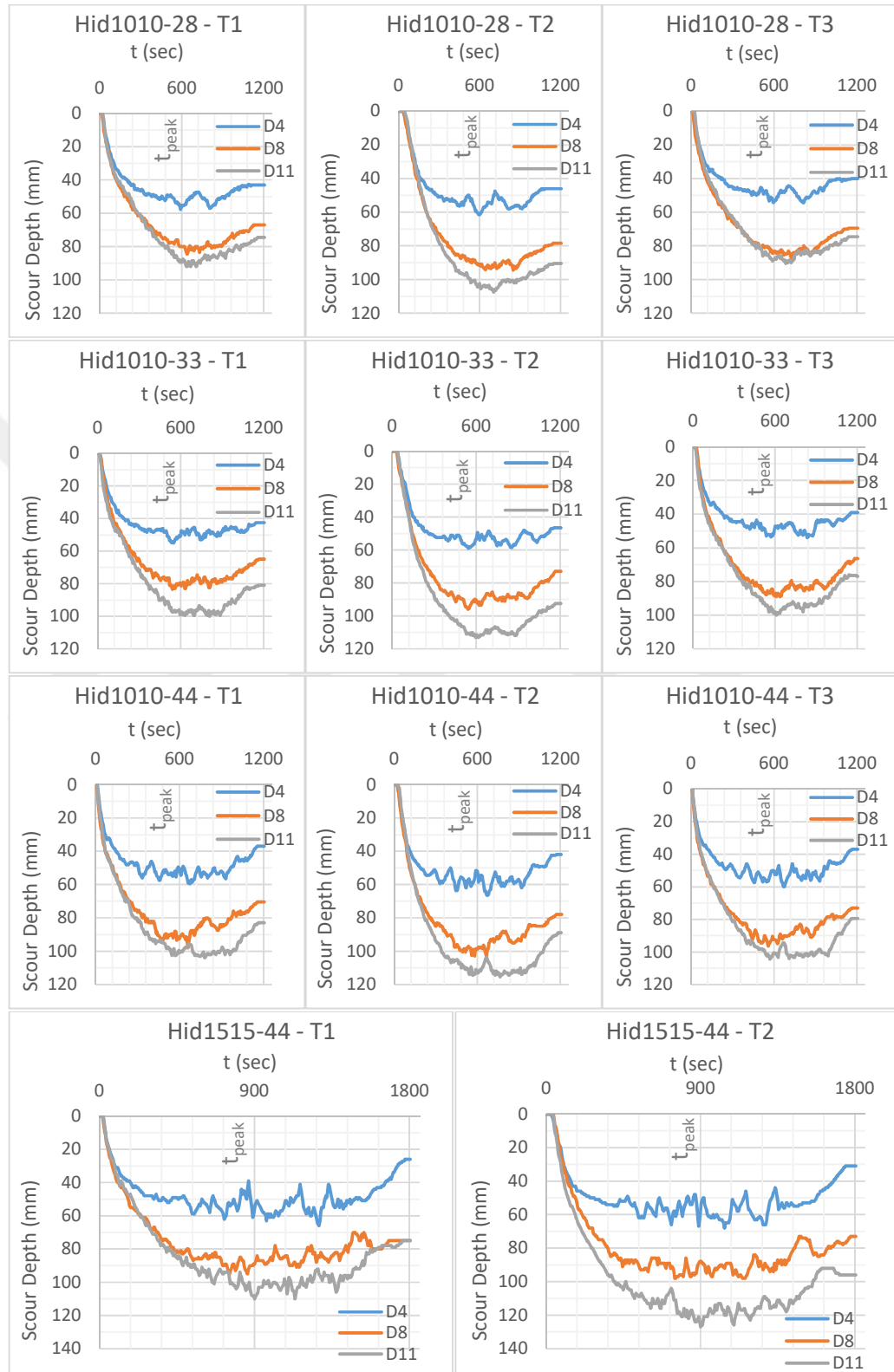


Figure 5.3 continues

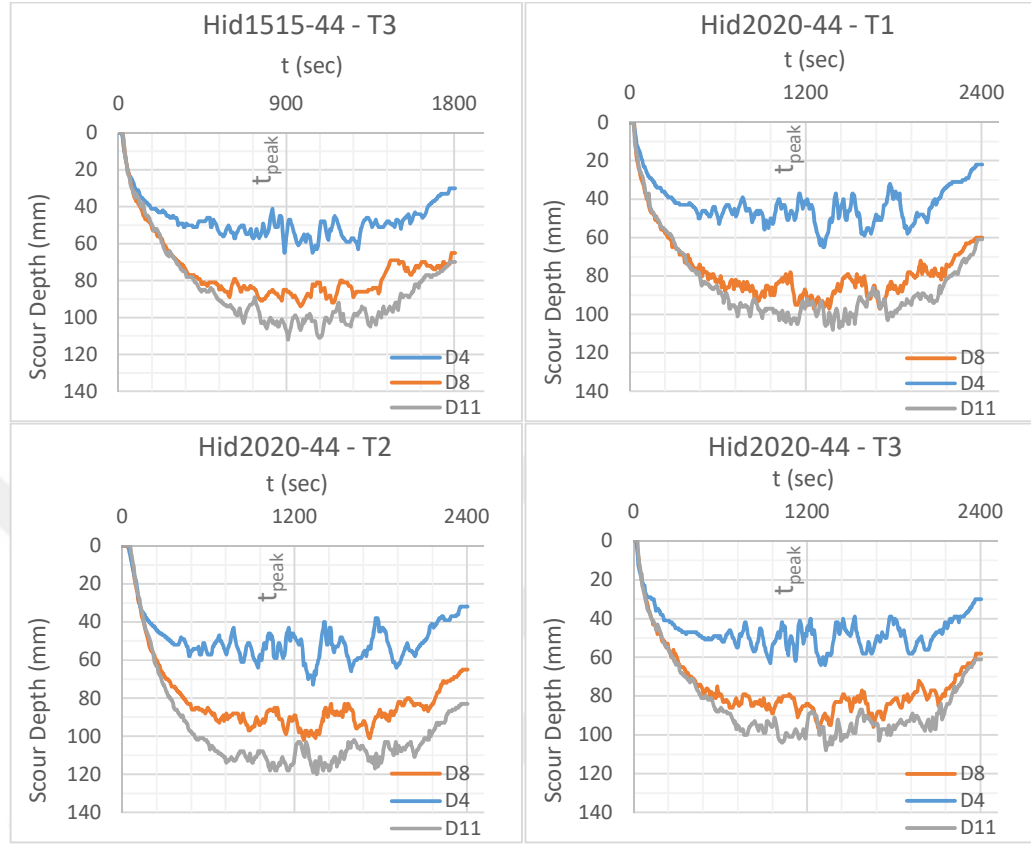


Figure 5.3 continues

It's not possible to mention equilibrium scour depth concept for these experiments. In clear water floods, a state of equilibrium occurs since scour depth increases during the rising limb and the change of scour depth during the falling limb is negligible (Chang et al. ,2004, Hager & Unger, 2010). This is not the case in live bed under unsteady flow conditions. In live bed - steady flow conditions, fluctuations become steady around a line which is called the live bed equilibrium scour depth (Laursen & Toch, 1956, Jain & Fischer, 1979, Chiew, 1984, Kothyari, Raju & Garde, 1992a, Melville, 1984, Sheppard, 2006). In our experiments, fluctuations never reached a constant value. Especially during the falling limb, sediment transport rate became more apparent with decreasing flow velocity. Therefore, maximum and final scour depths will be assessed in this study.

As seen from Figure 5.3, scour depth increased with pier diameter as expected. Initially, scour process started at the flanks of the pier and scour depth at flanks were

higher than that in front of the pier. After certain time, scour depth in front of the pier became greater and maximum scour depth occurred at this point. Since wake vortices form first and horseshoe vortices form with the development of scour hole, reason of this situation can be formation of downflow and horseshoe vortices. This is consistent with Melville (1975)'s "Horseshoe vortex is initially small, roughly circular in cross-section, and comparatively weak, increases dramatically in size and strength as the scour hole forms." explanation. Table 5.1 shows the exact time values when scour depth in front of the pier exceeded average scour depths at flanks. Time dependent scour depths for each pier and hydrograph are also given on Appendix 1.

Table 5.1 Time and scour depths values where scour depth in front of pier exceeded average scour depths of flanks

	D4			D8			D11		
	t (sec)	T _{1,3average}	T ₂	t (sec)	T _{1,3average}	T ₂	t (sec)	T _{1,3average}	T ₂
		d (mm)	d (mm)		d (mm)	d (mm)		d (mm)	d (mm)
Hid33-20	95	38.0	40.0	115.0	47.5	50.0	90.0	41.0	45.0
Hid33-31	70	37.5	38.0	85.0	53.5	54.0	90.0	53.0	54.0
Hid36-31	65	36.0	38.0	70.0	47.5	51.0	70.0	47.0	51.0
Hid39-31	60	35.5	38.0	55.0	41.5	44.0	65.0	43.5	45.0
Hid33-38	55	39.0	42.0	65.0	49.5	50.0	50.0	44.0	46.0
Hid55-23	105	32.5	34.0	85.0	39.0	40.0	120.0	47.5	50.0
Hid55-28	70	31.5	32.0	105.0	51.5	52.0	115.0	51.5	52.0
Hid55-31	60	29.5	30.0	90.0	49.0	50.0	90.0	49.0	50.0
Hid55-33	70	32.5	34.0	100.0	55.5	56.0	90.0	46.5	49.0
Hid55-38	55	34.0	36.0	85.0	52.5	53.0	70.0	47.0	52.0
Hid55-44	50	34.0	35.0	90.0	58.5	60.0	65.0	47.0	48.0
Hid1010-28	145	35.5	38.0	145.0	45.0	48.0	165.0	47.0	48.0
Hid1010-33	135	34.5	36.0	110.0	40.5	41.0	115.0	44.5	46.0
Hid1010-44	85	33.5	34.0	100.0	46.5	47.0	115.0	50.5	52.0
Hid1515-44	120	35.5	36.0	145.0	46.5	47.0	85.0	33.5	34.0
Hid2020-44	115	28.0	30.0	175.0	48.0	50.0	200.0	52.5	53.0

As it was mentioned above, moving sediment clusters caused fluctuations on scour depth curve. After one cluster fell into the scour hole and decreased scour depth, flow continued to scour and increased scour depth till the following sediment cluster arrived. From Figure 5.3, pier diameter affected the height of these fluctuations. Since the amount of transported sediment was the same for a given hydrograph, this amount of sediment filled more volume in small scour holes than larger scour holes. This is why the piers with relatively small width caused relatively high fluctuations. Table 5.2 shows greatest fluctuation heights (h_f) of the pier D11 and ratios of the greatest fluctuation heights of the piers D4 to D11 and D8 to D11, for each hydrograph. The incompatibilities are generally observed in the case of the hydrographs with low peak discharge and short duration which cause less reliable measurements.

Table 5.2 Greatest fluctuation heights for each transducer, according to relevant pier and hydrograph

	T1			T2			T3		
	$h_{f,D4} / h_{f,D11}$	$h_{f,D8} / h_{f,D11}$	$h_{f,D11}$ (mm)	$h_{f,D4} / h_{f,D11}$	$h_{f,D8} / h_{f,D11}$	$h_{f,D11}$ (mm)	$h_{f,D4} / h_{f,D11}$	$h_{f,D8} / h_{f,D11}$	$h_{f,D11}$ (mm)
Hid33-20	1.50	1.00	2	2.00	1.00	1	2.00	1.00	1
Hid33-31	2.50	2.00	2	6.00	2.00	1	2.00	1.00	3
Hid36-31	2.00	1.25	4	2.50	1.00	4	1.00	1.00	5
Hid39-31	1.17	0.83	6	1.75	1.25	4	1.75	1.25	4
Hid33-38	1.33	1.00	3	1.50	1.00	2	1.33	1.00	3
Hid55-23	1.67	1.33	3	4.00	2.00	1	1.00	1.00	3
Hid55-28	1.00	1.00	6	1.33	1.00	3	0.75	0.75	4
Hid55-31	1.14	0.57	7	3.00	1.33	3	1.80	1.00	5
Hid55-33	1.14	0.71	7	3.33	1.67	3	2.50	1.50	4
Hid55-38	1.25	1.00	8	3.00	1.75	4	3.00	1.75	4
Hid55-44	2.00	1.00	5	1.67	1.17	6	2.00	1.60	5
Hid1010-28	1.29	1.00	7	1.25	0.75	8	2.00	0.83	6
Hid1010-33	1.25	1.00	8	1.75	1.50	8	1.67	1.50	6
Hid1010-44	2.00	1.38	8	1.80	1.30	10	2.00	1.36	11
Hid1515-44	1.85	1.31	13	1.47	1.27	15	1.33	1.20	15
Hid2020-44	2.00	1.29	14	1.94	1.12	17	1.20	0.95	20

5.3 Effect of the Duration of Rising and Falling Limbs

Hydrographs with the same peak discharges but different durations were classified separately in order to analyze effect of time.

5.3.1 Scour Depths for the Pier D4

Figure 5.4, 5.5 and 5.6 illustrate the effect of rising and falling durations for the pier of 4 cm diameter at T1, T2 and T3 transducers respectively.

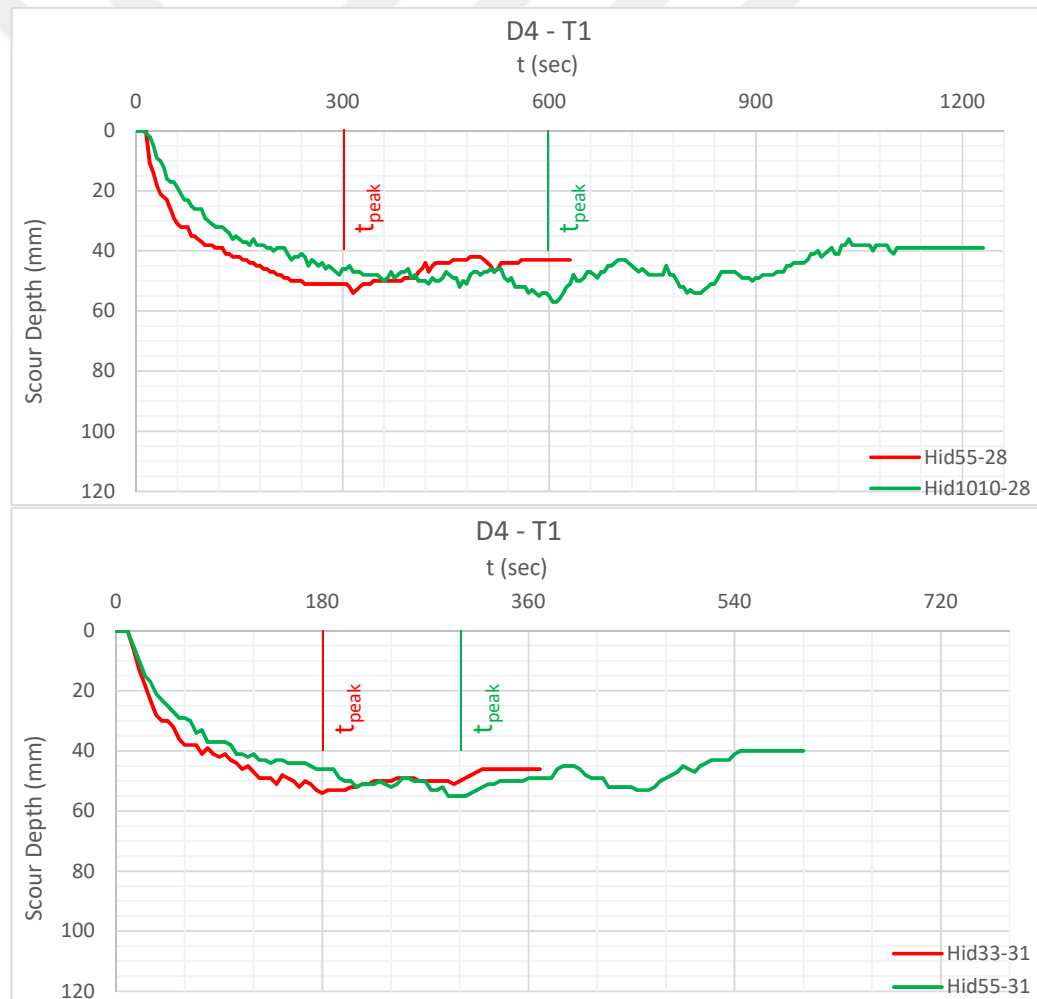


Figure 5.4 Effect of rising and falling durations for the pier D4/transducer T1

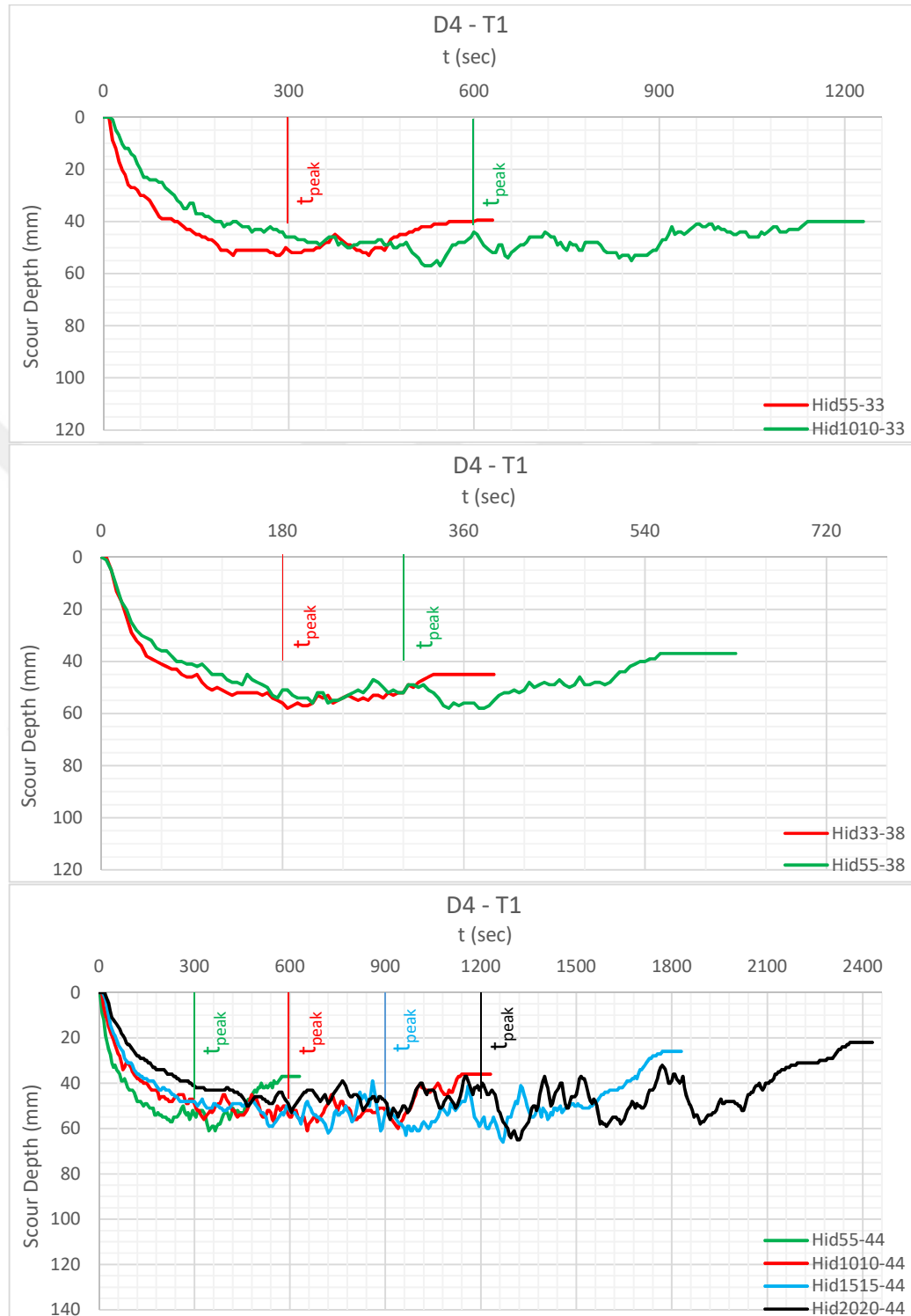


Figure 5.4 continues

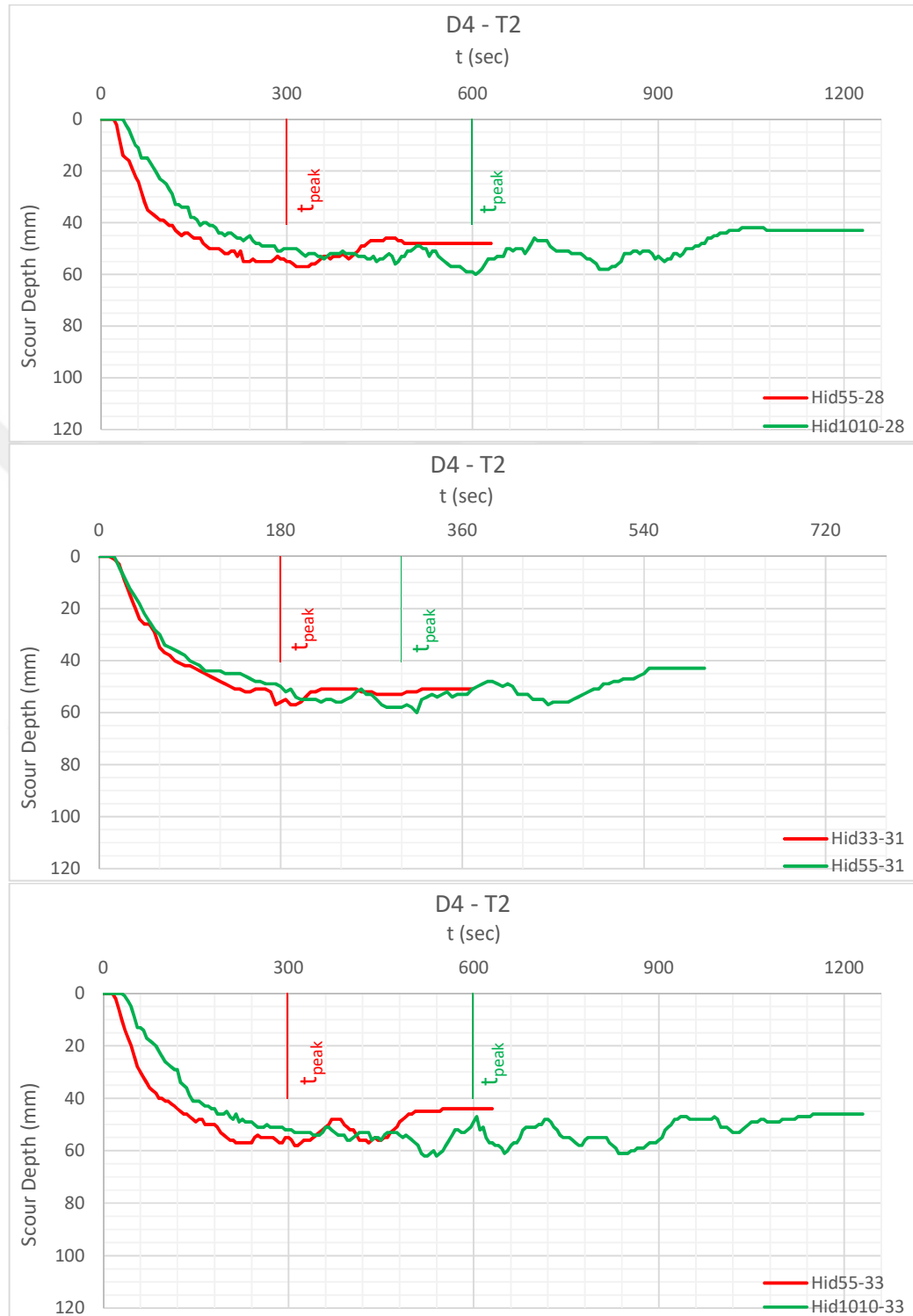


Figure 5.5 Effect of rising and falling durations for the pier D4/transducer T2

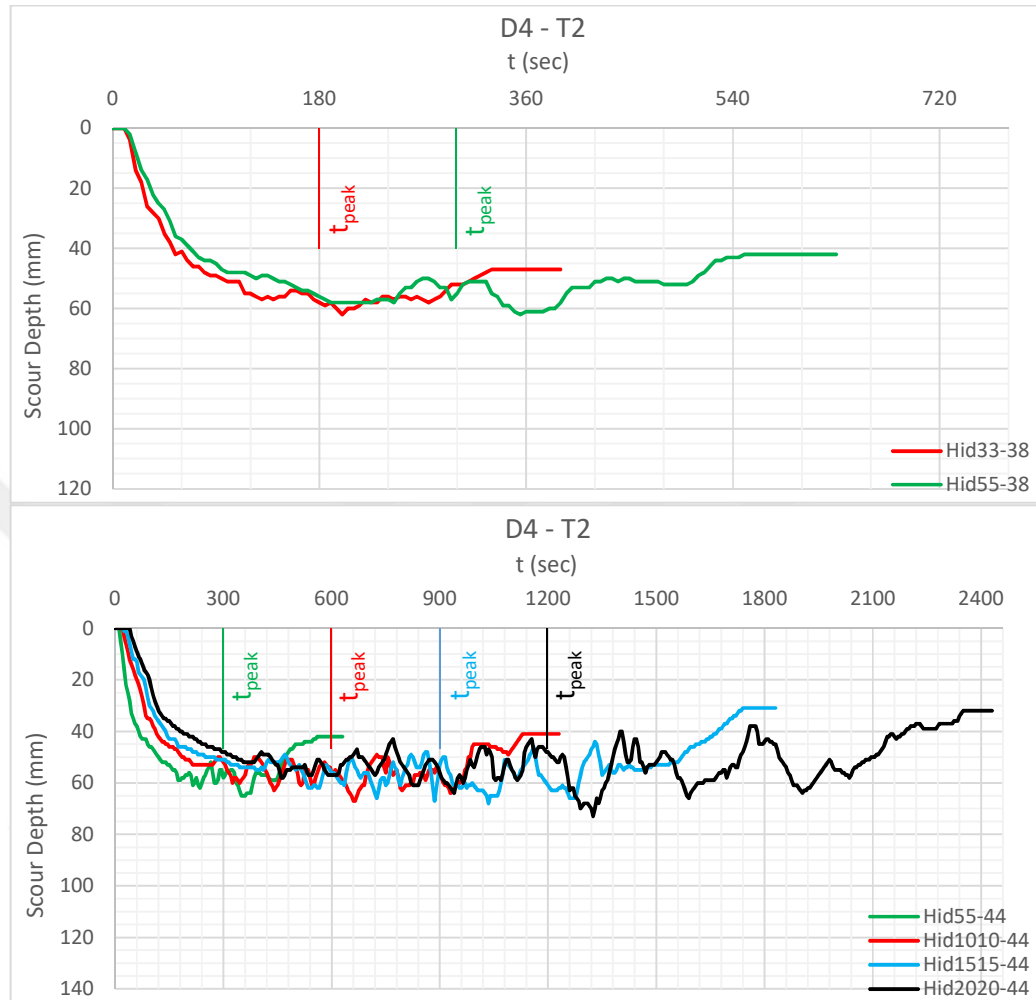


Figure 5.5 continues

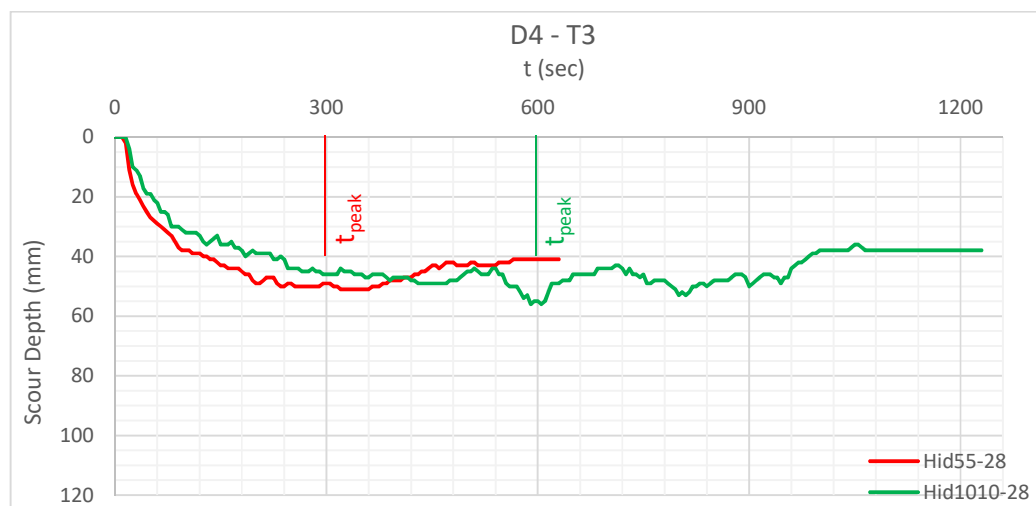


Figure 5.6 Effect of rising and falling durations for the pier D4/transducer T3

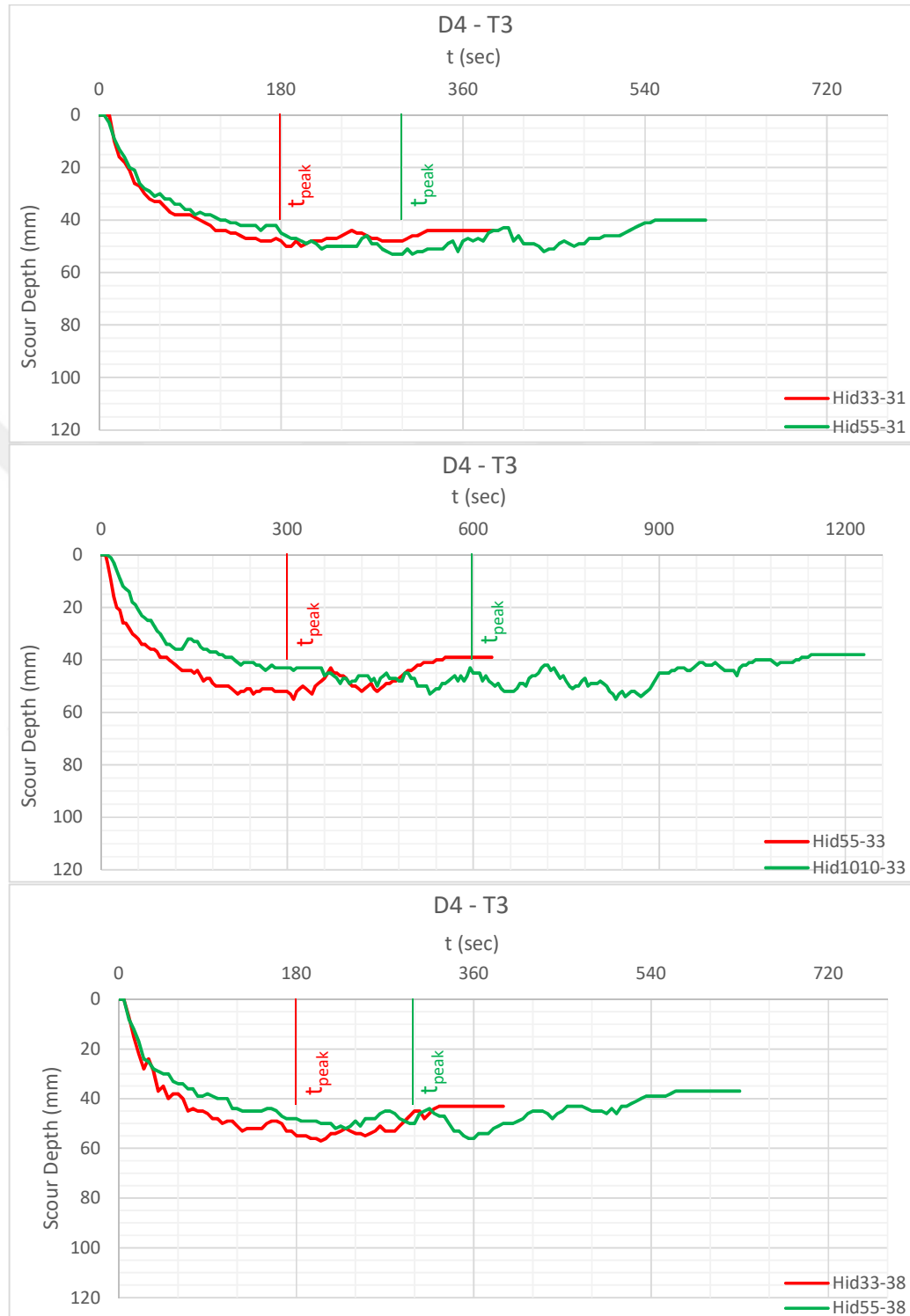


Figure 5.6 continues

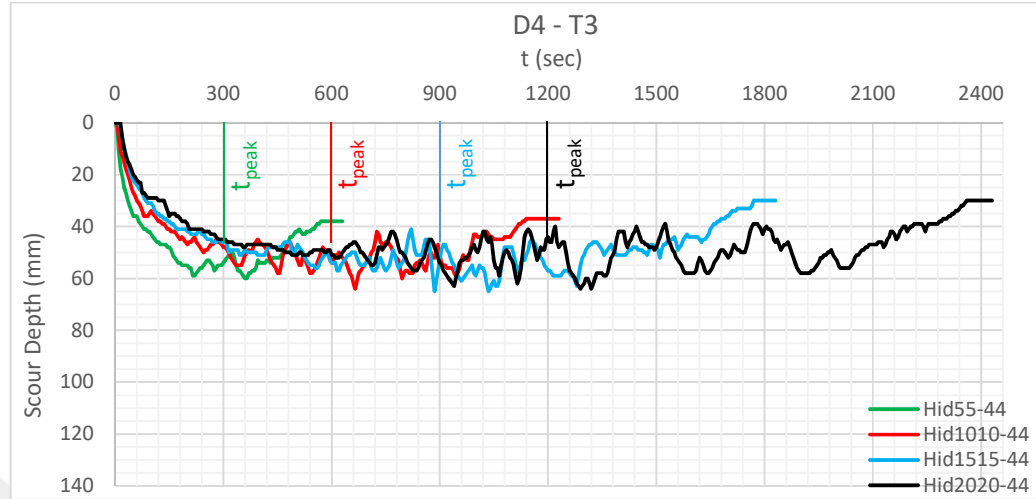


Figure 5.6 continues

5.3.2 Scour Depths for the Pier D8

Figure 5.7, 5.8 and 5.9 illustrate the effect of rising and falling durations for the pier of 8 cm diameter at T1, T2 and T3 transducers respectively.

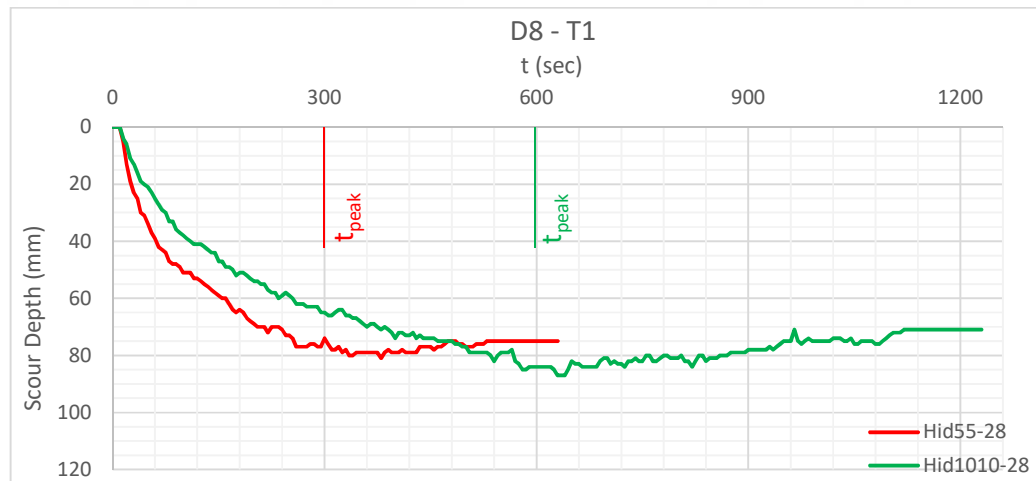


Figure 5.7 Effect of rising and falling durations for the pier D8/transducer T1

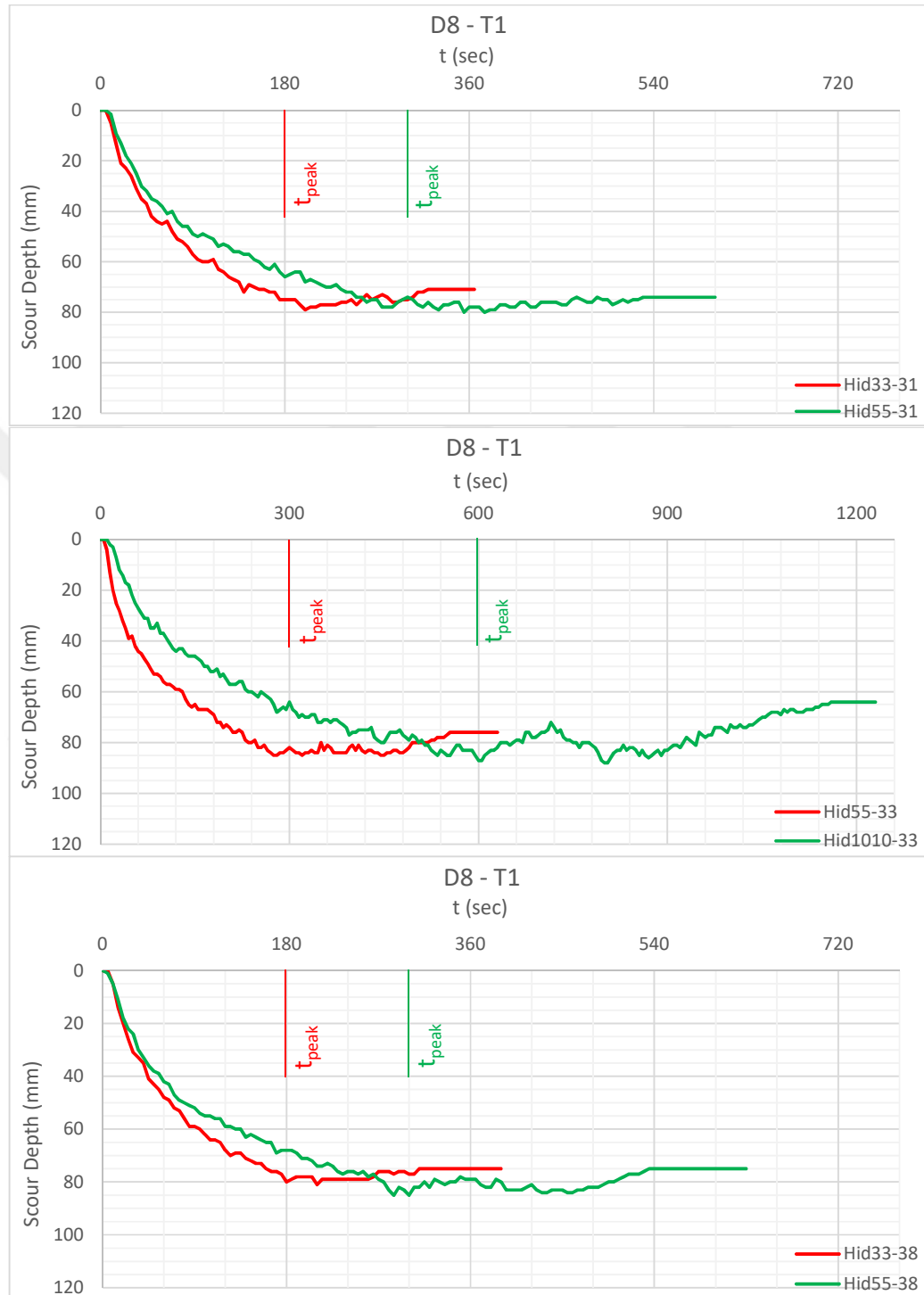


Figure 5.7 continues

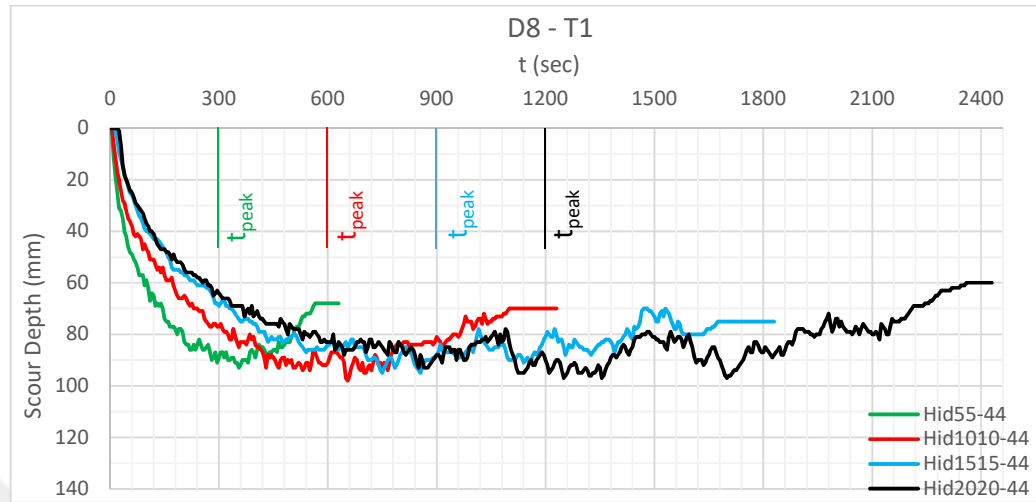


Figure 5.7 continues

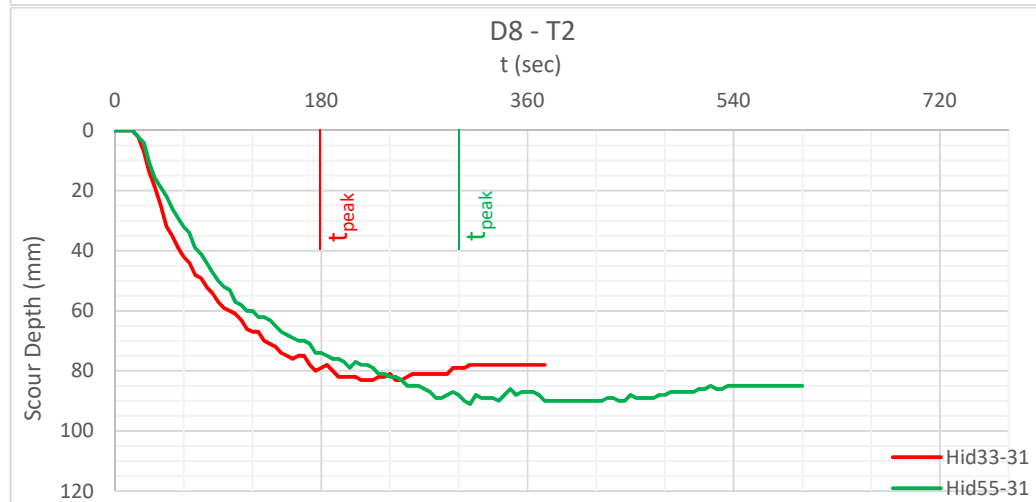
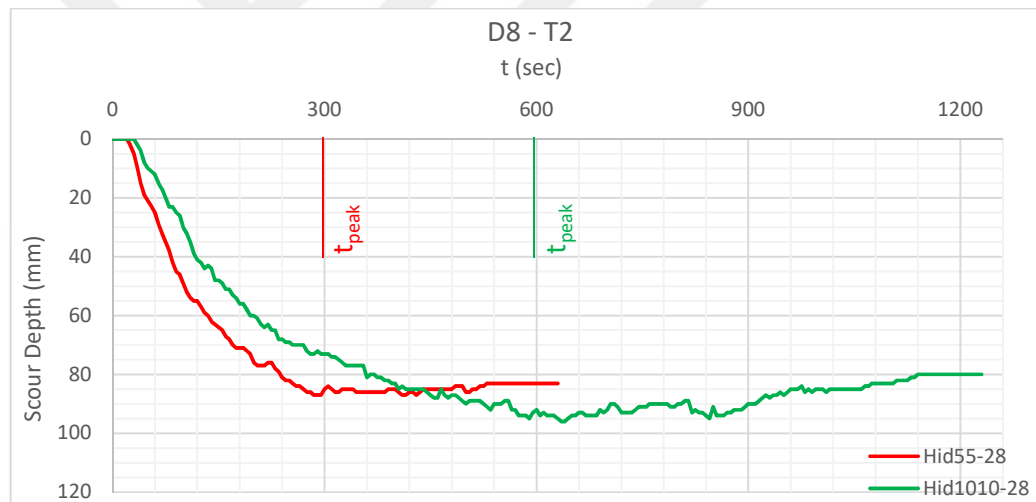


Figure 5.8 Effect of rising and falling durations for the pier D8/transducer T2

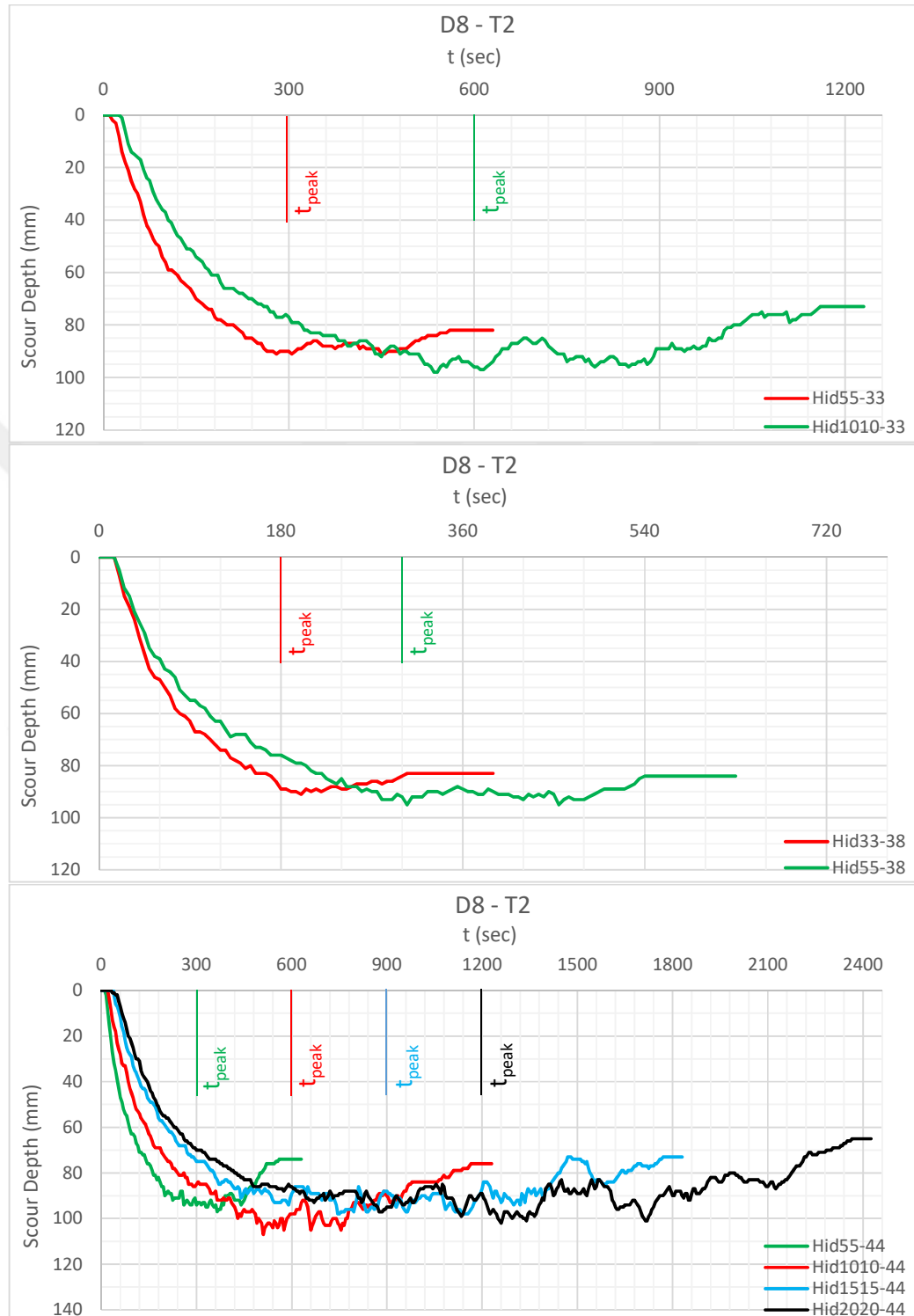


Figure 5.8 continues

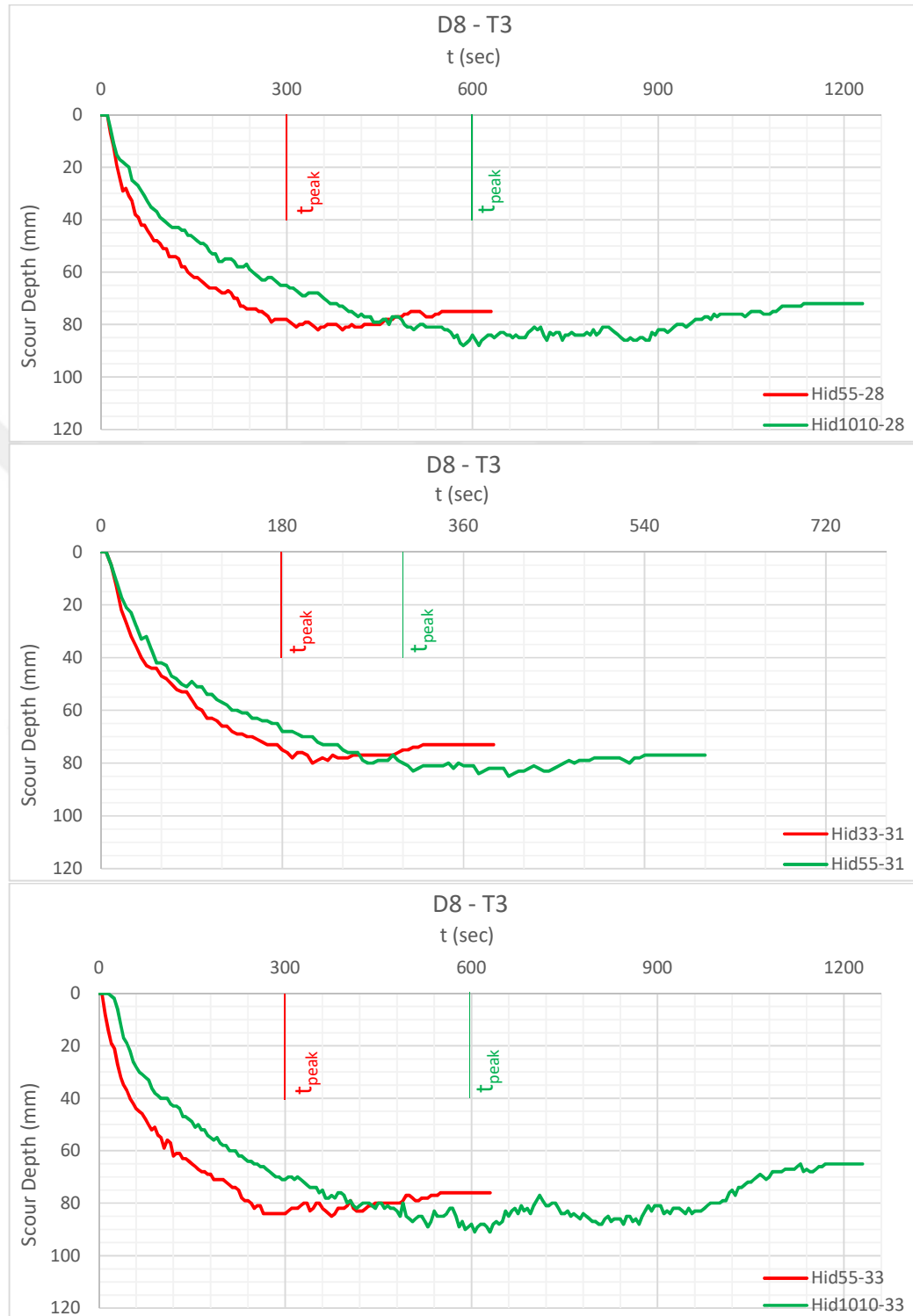


Figure 5.9 Effect of rising and falling durations for the pier D8/transducer T3

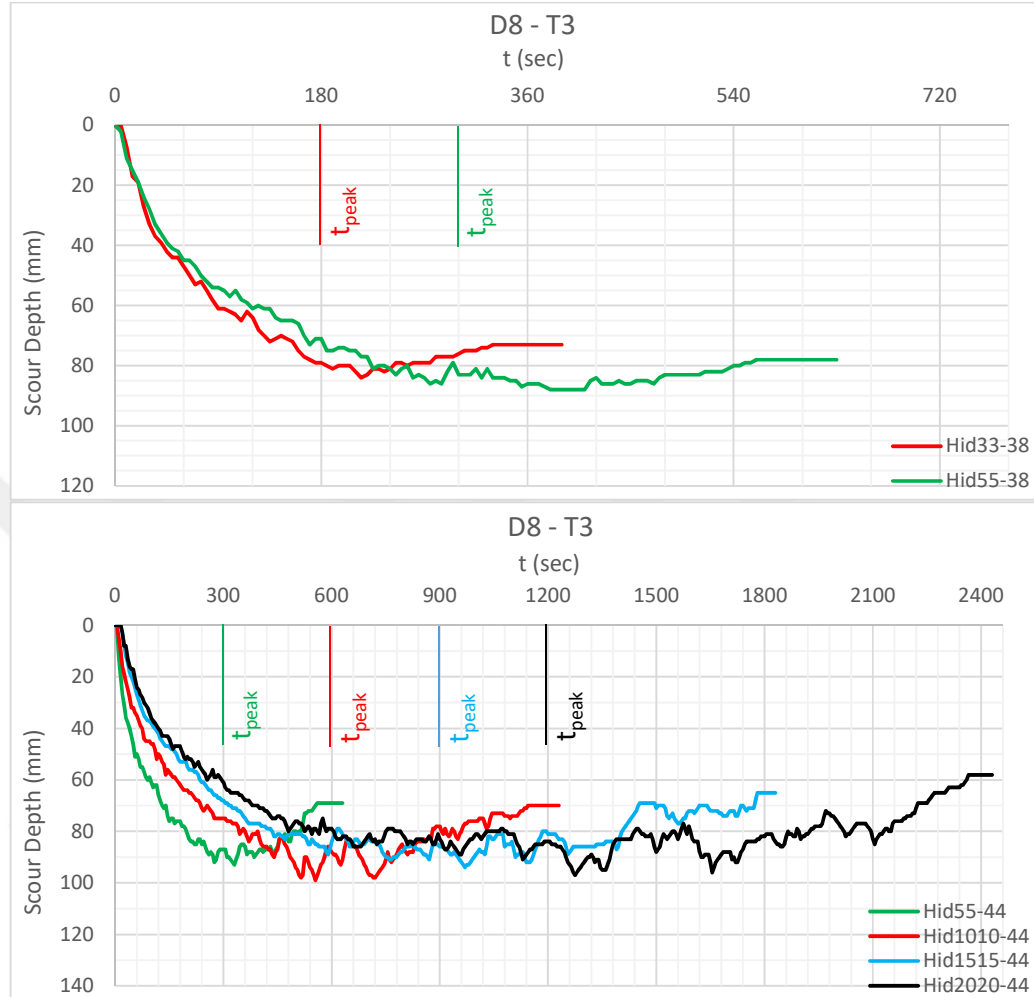


Figure 5.9 continues

5.3.3 Scour Depths for the Pier D11

Figure 5.10 5.11 and 5.12 illustrate the effect of rising and falling durations for the pier of 11 cm diameter at T1, T2 and T3 transducers respectively.

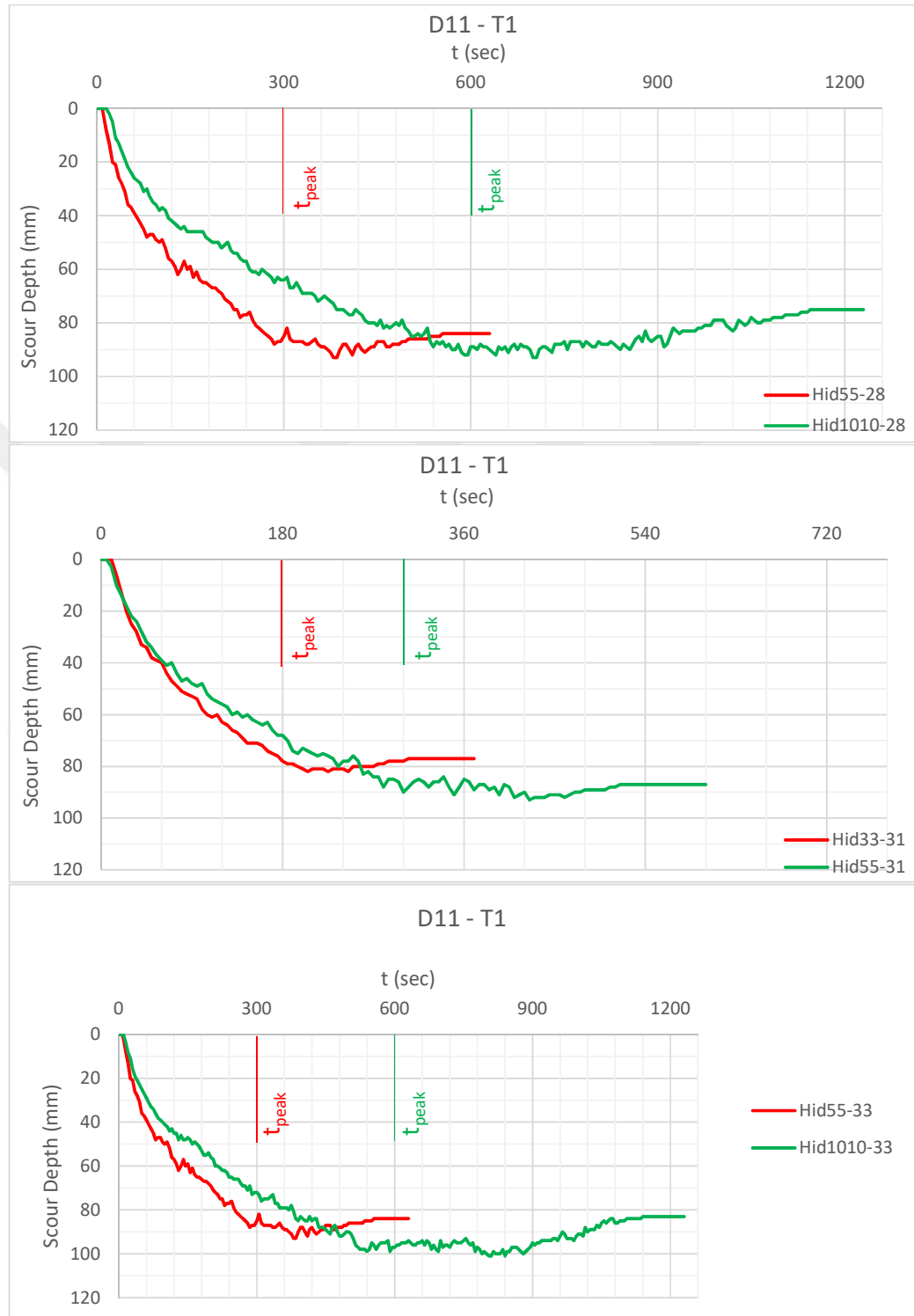


Figure 5.10 Effect of rising and falling durations for the pier D11/transducer T1

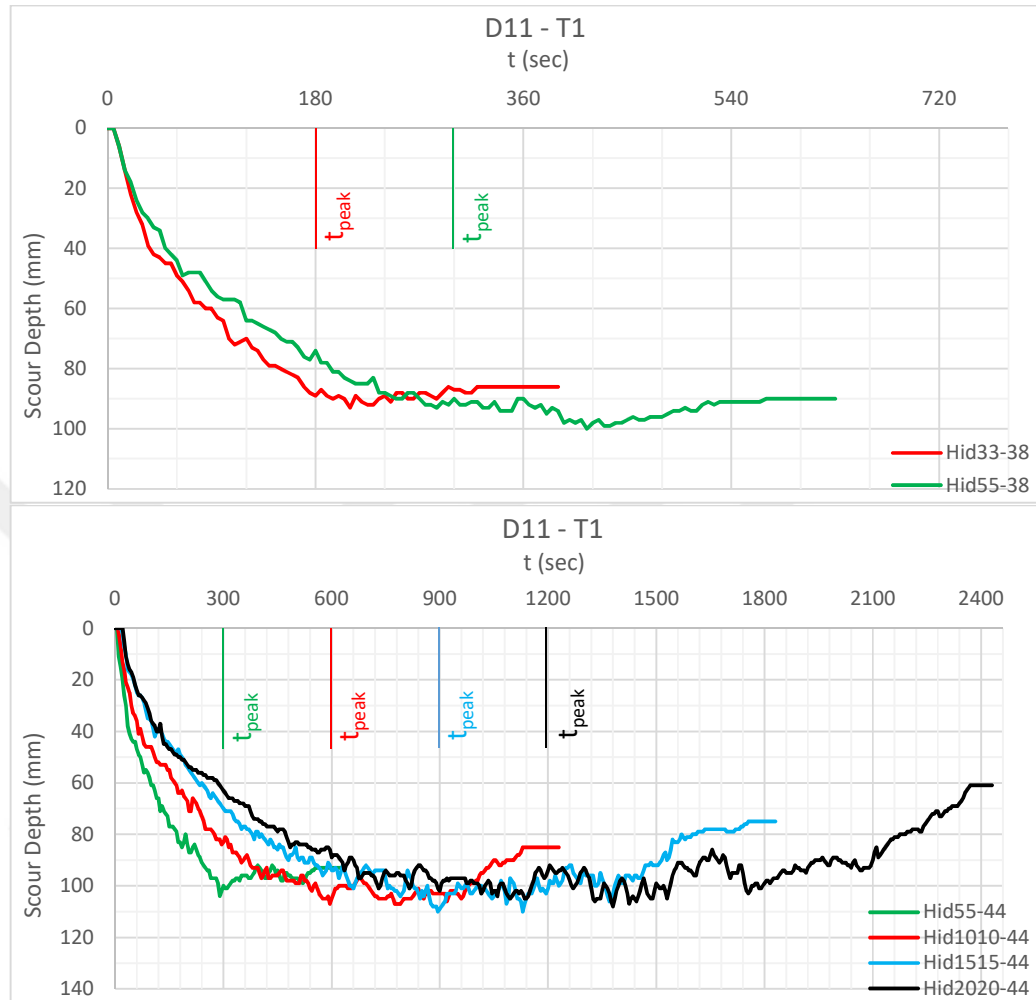


Figure 5.10 continues

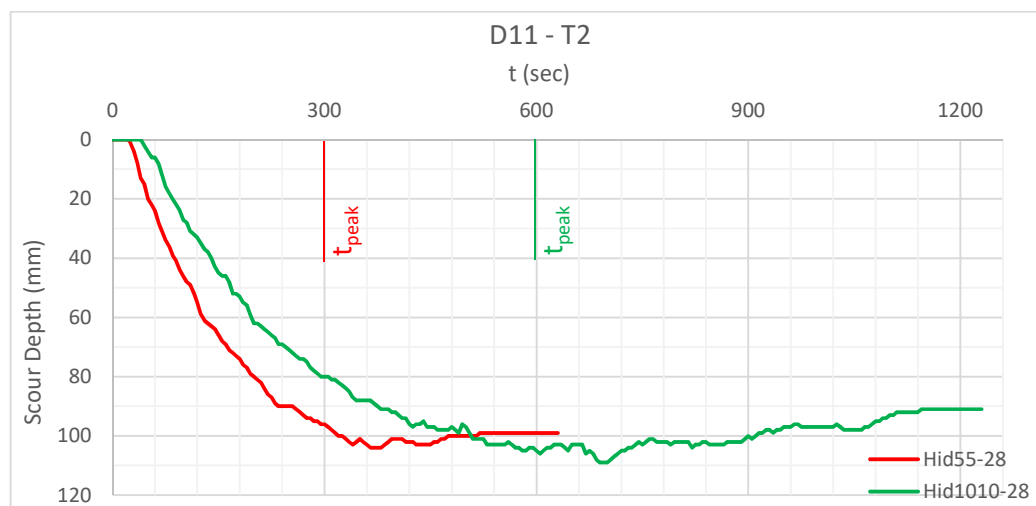


Figure 5.11 Effect of rising and falling durations for the pier D11/transducer T2

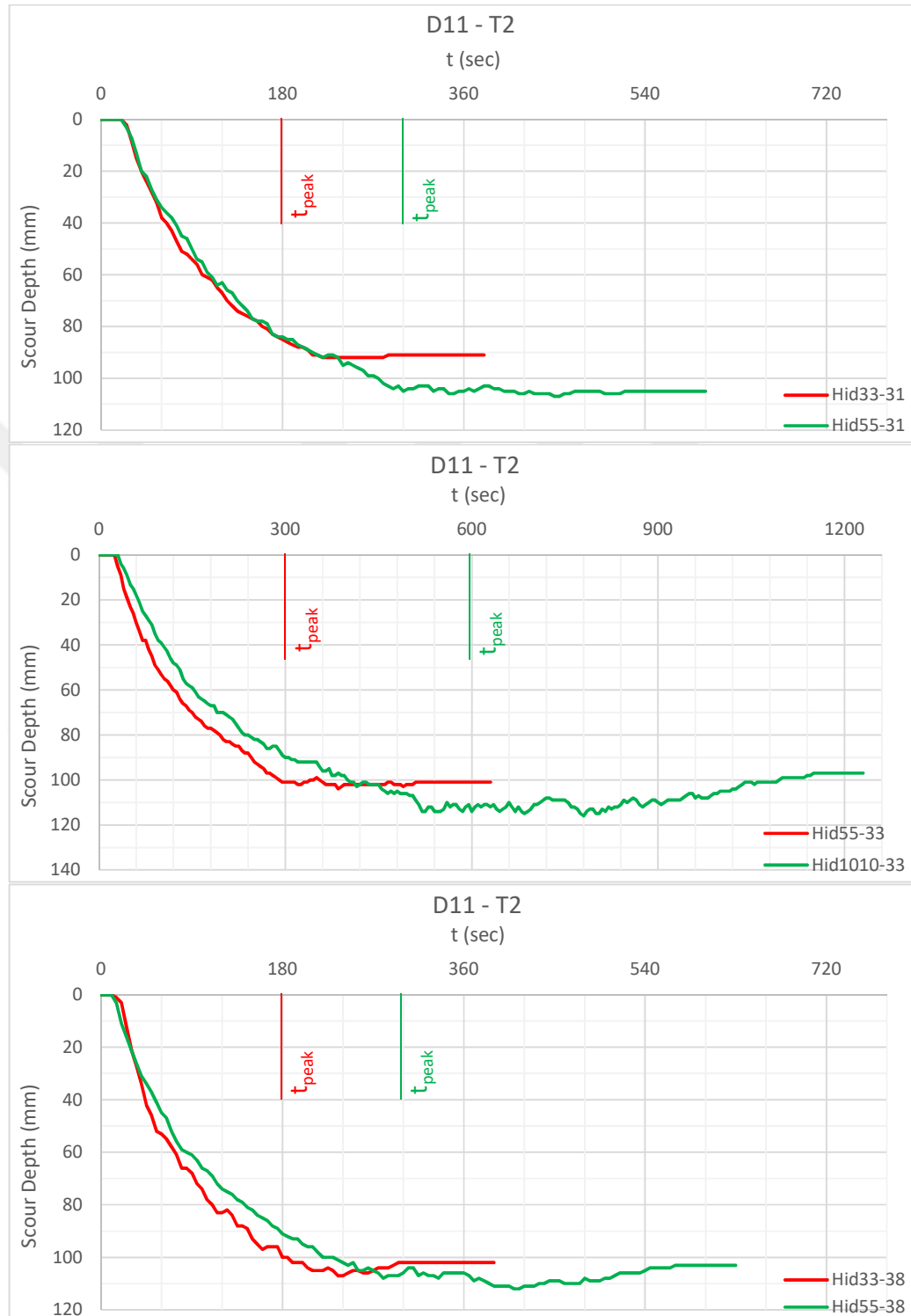


Figure 5.11 continues

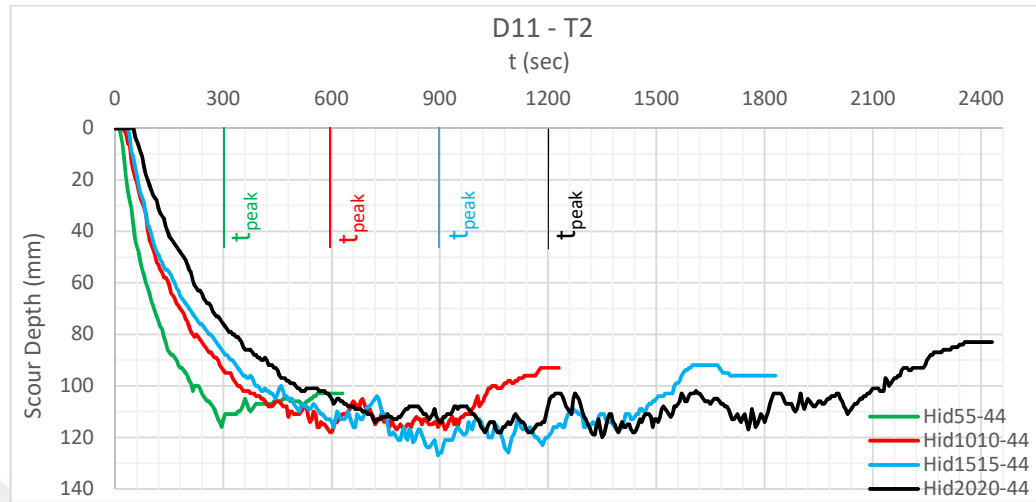


Figure 5.11 continues

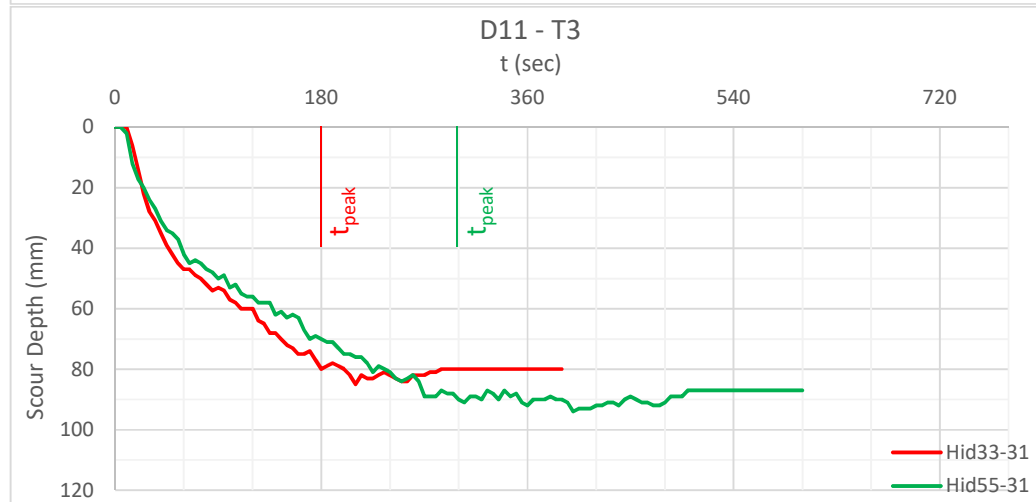
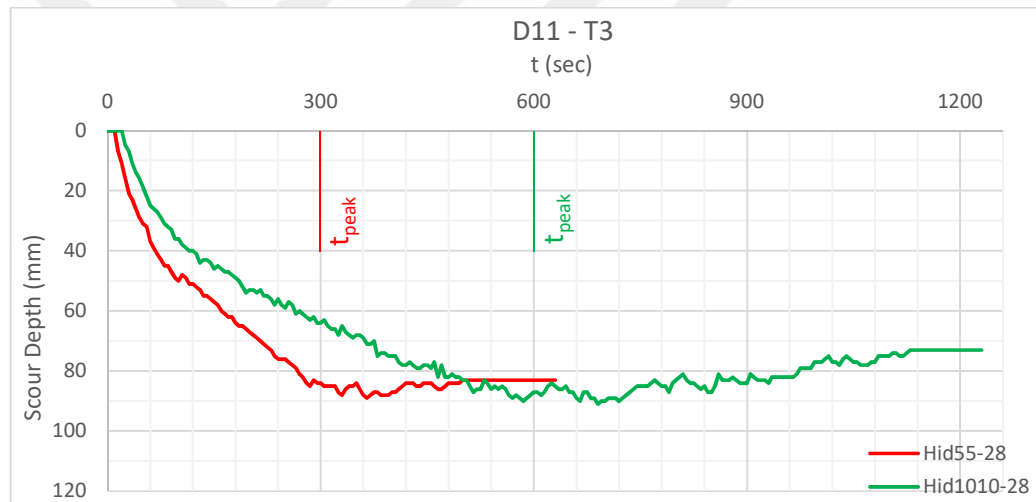


Figure 5.12 Effect of rising and falling durations for the pier D11/transducer T3

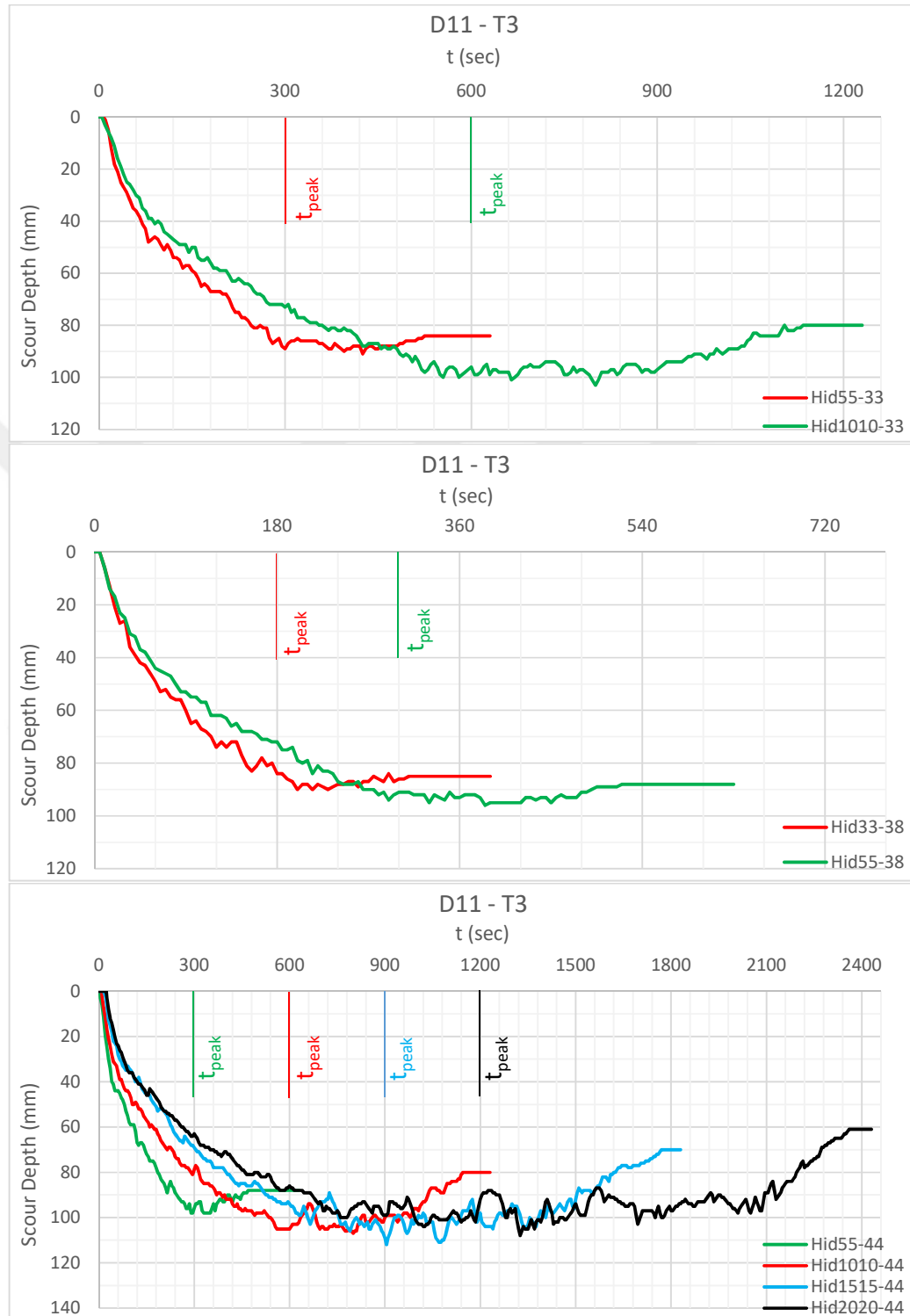


Figure 5.12 continues

5.3.4 Effect of the Duration of Asymmetric Rising and Falling Limbs

Hid33-31, Hid36-31 and Hid39-31 share the same durations of rising limb (3 mins) but different durations of falling limb (3, 6 and 9 mins). Figure 5.13, 5.14 and 5.15 illustrate the effect of rising and falling durations, indicate of the peak discharge of 31 L/s, for the piers of 4 cm, 8 cm and 11 cm diameters respectively.

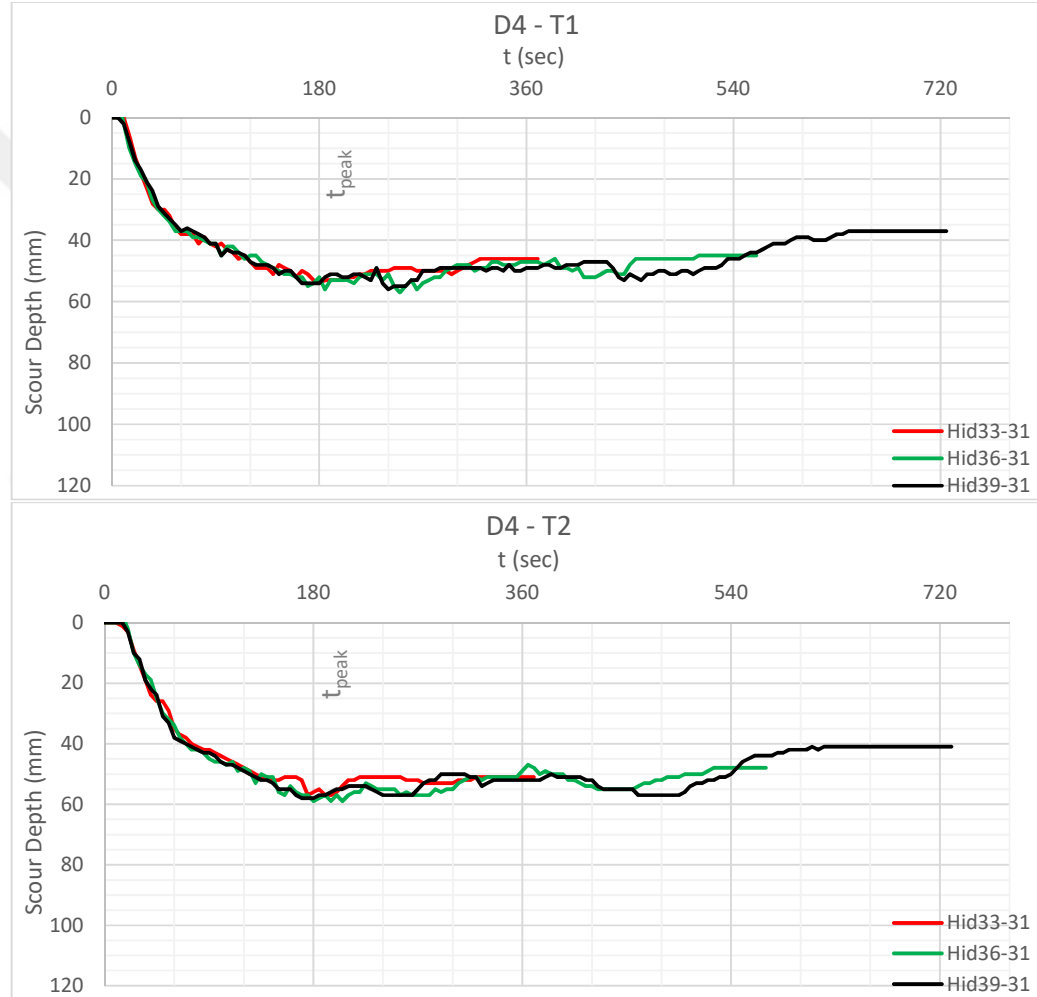


Figure 5.13 Asymmetric hydrographs with the peak discharge of 31 L/s for the pier D4

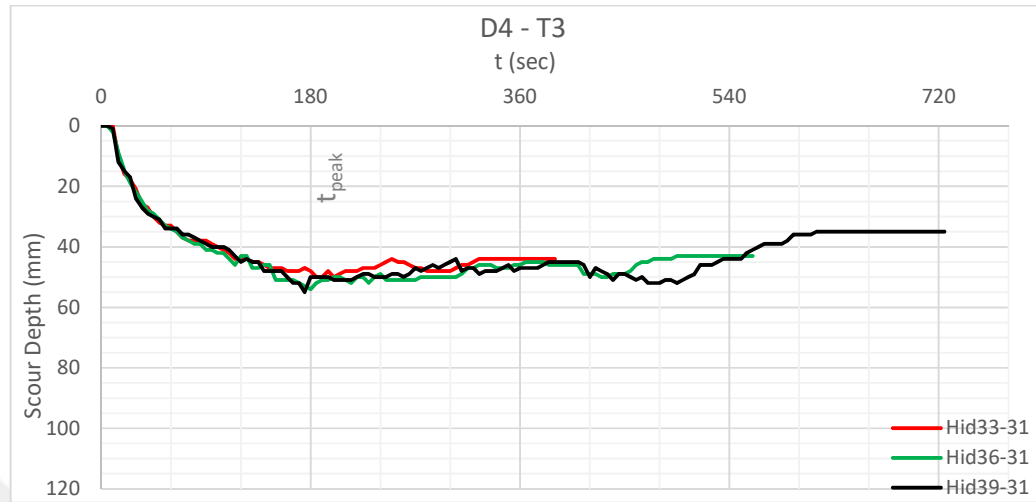


Figure 5.13 continues

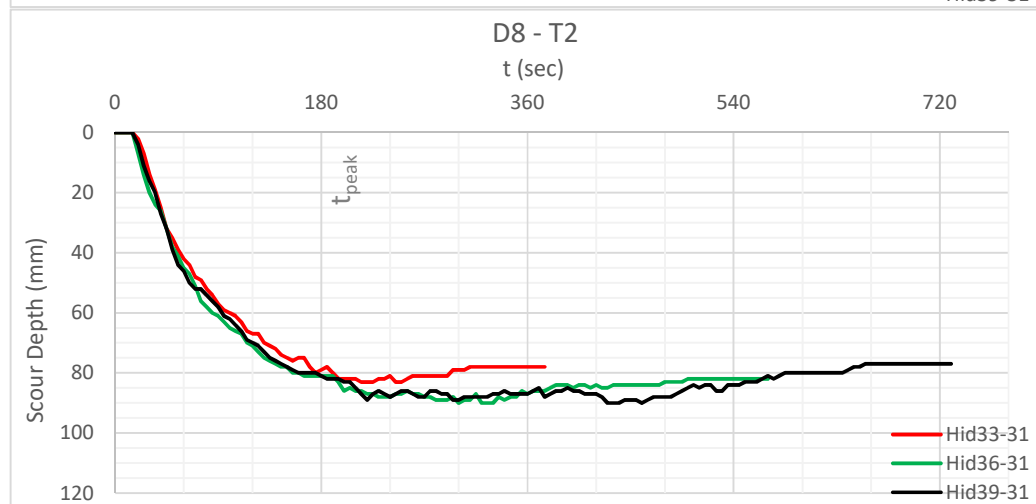
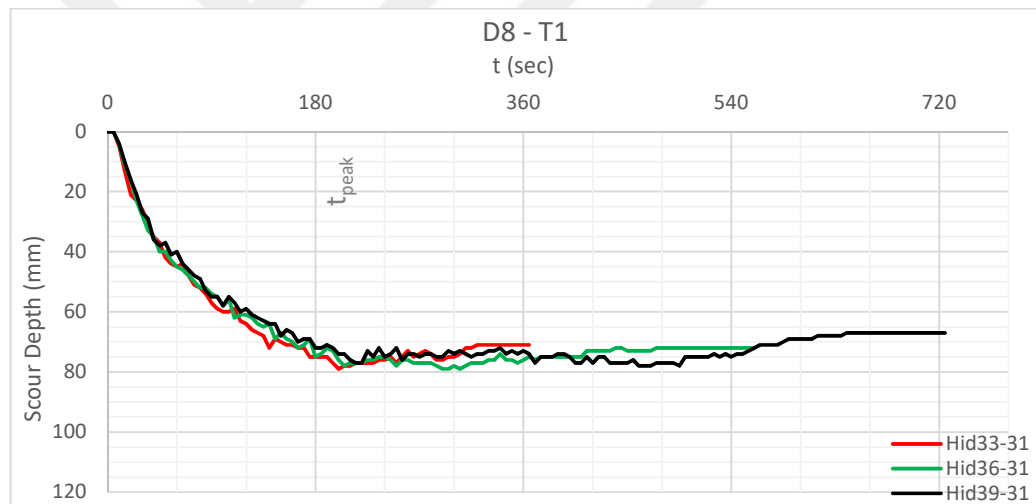


Figure 5.14 Asymmetric hydrographs with the peak discharge of 31 L/s for the pier D8

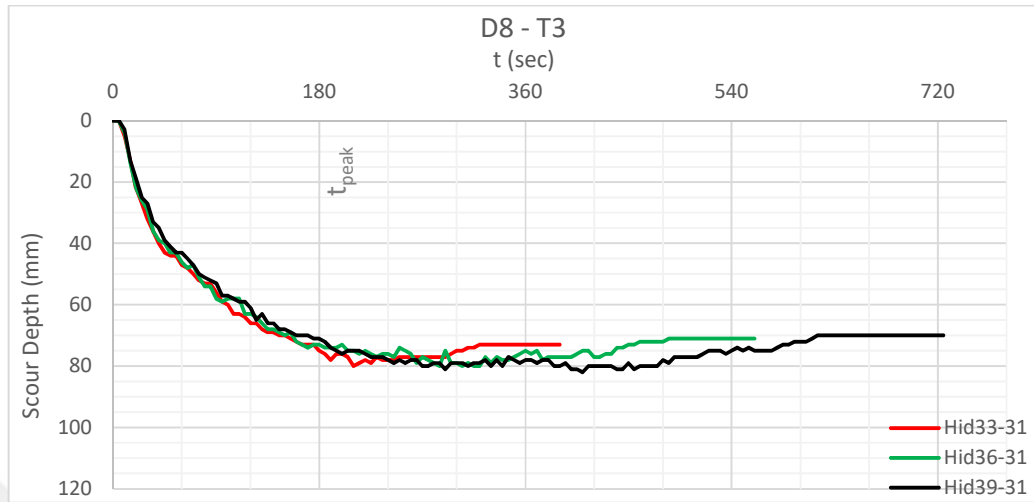


Figure 5.14 continues

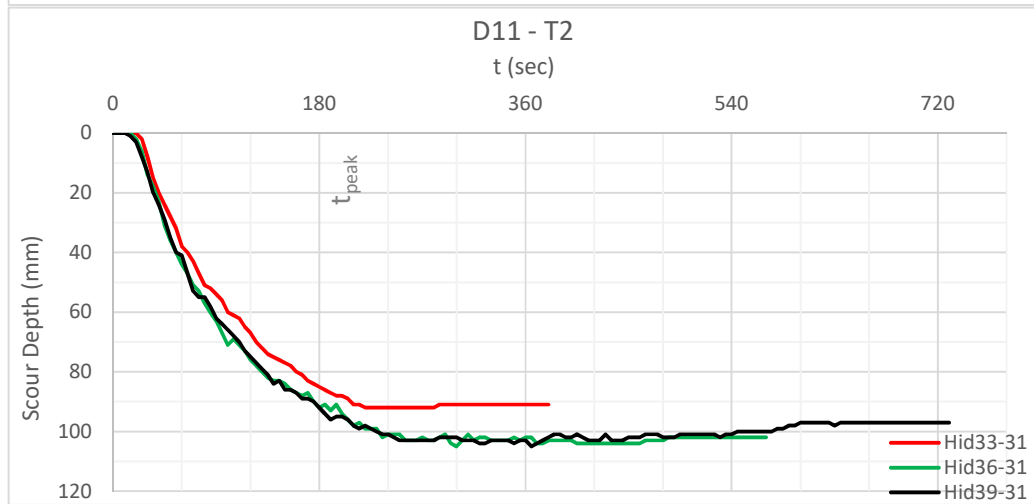
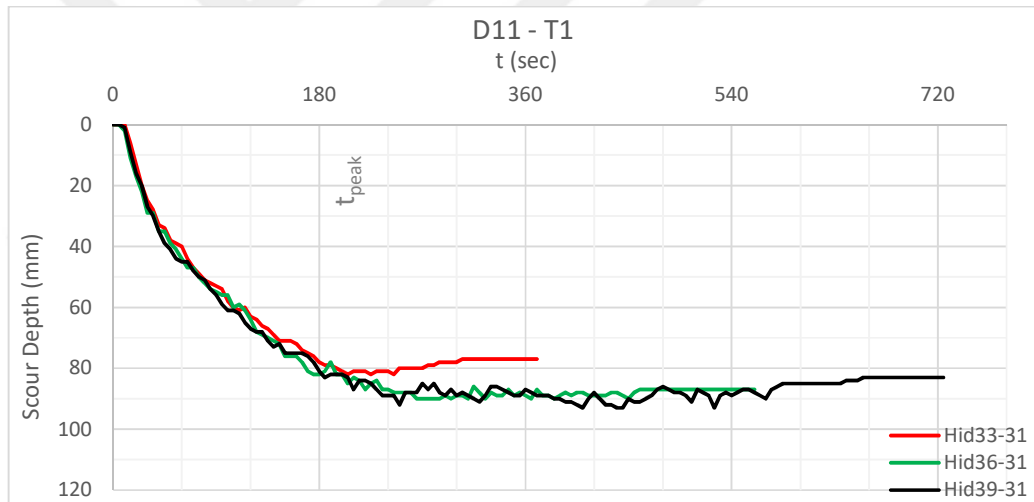


Figure 5.15 Asymmetric hydrographs with the peak discharge of 31 L/s for the pier D11

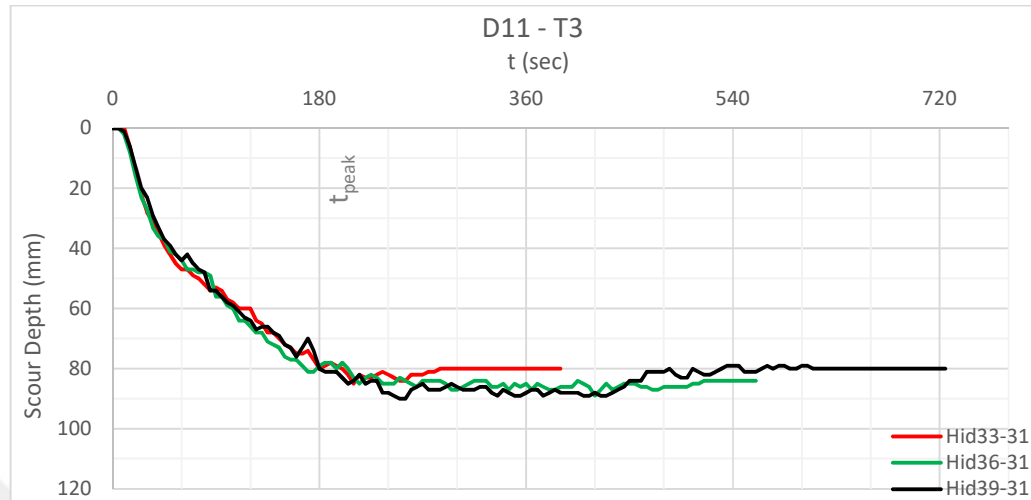


Figure 5.15 continues

Following conclusions were generally observed as a general overview of the effect of the rising and falling limb durations:

- As the duration of flood increased, sediment was exposed to high velocities for a longer time. Hence, maximum scour depth at the end of the rising limb increased with increase in the duration of rising limb.
- Due to the longer exposition to high velocities, longer duration of floods increased the magnitude of fluctuations. Maximum range of fluctuations are given in Table 5.1 for each pier, transducer and hydrograph.
- As the duration of falling limb became longer, amount of sediment supplied into the scour hole increased, so a lower final scour depth was observed.
- Difference between maximum and final scour depth ($d_m - d_f$) increased with the increase in the duration of falling limb.

Some incompatibilities observed during the experiments are thought to be a result of the stochastic character of the sediment movement.

5.4 Effect of the Peak Discharge

Hydrographs with same durations but different peak discharges were classified separately in order to analyze effect of the peak discharge.

5.4.1 Scour Depths for the Pier D4

Figure 5.16, 5.17 and 5.18 illustrate the effect of the peak discharge for the pier of 4 cm diameter at T1, T2 and T3 transducers respectively.

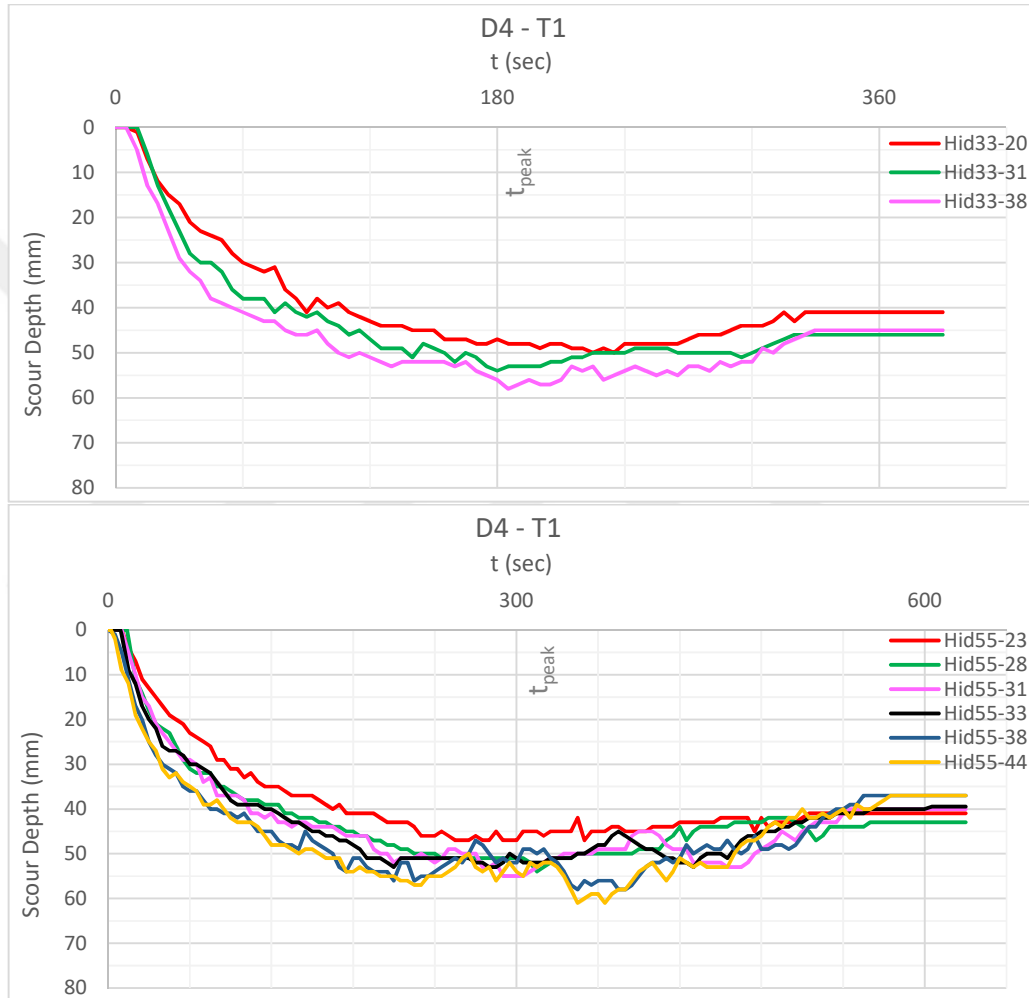


Figure 5.16 Effect of peak discharge for the pier D4/transducer T1

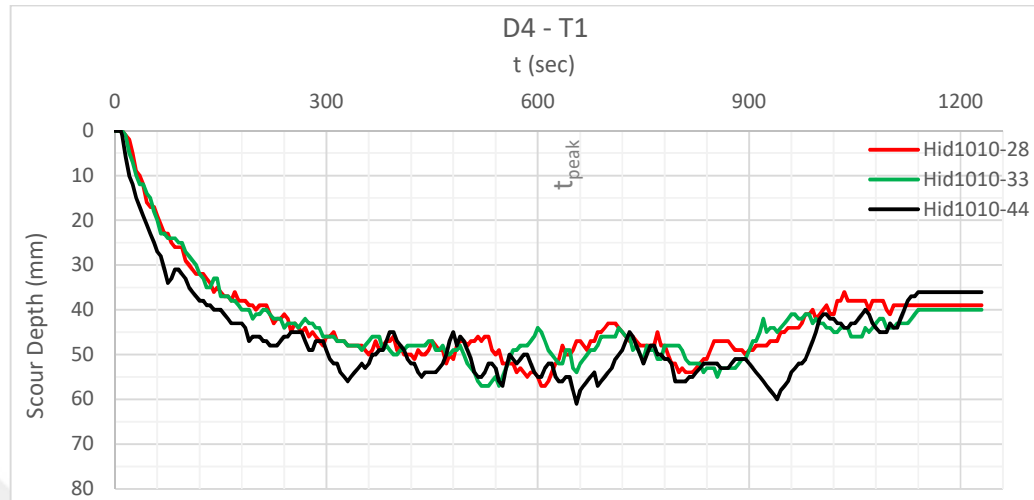


Figure 5.16 continues

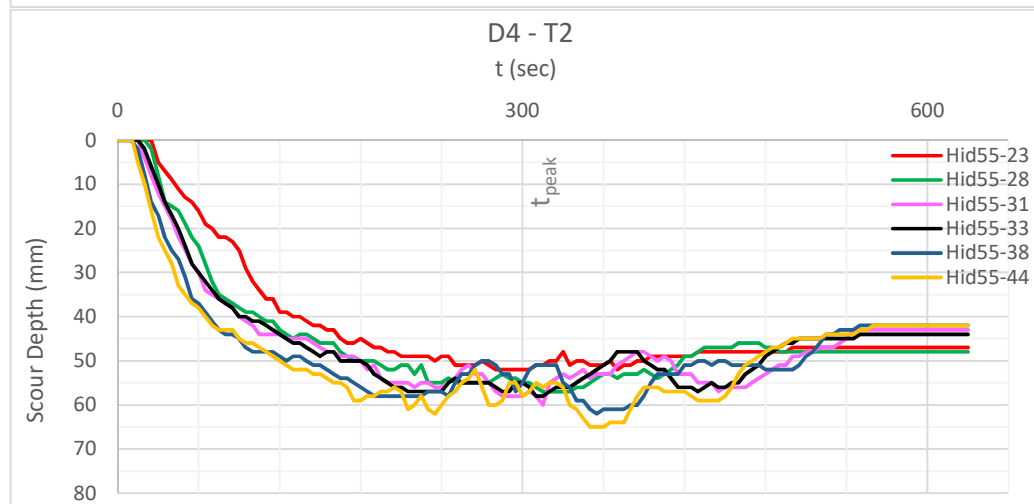
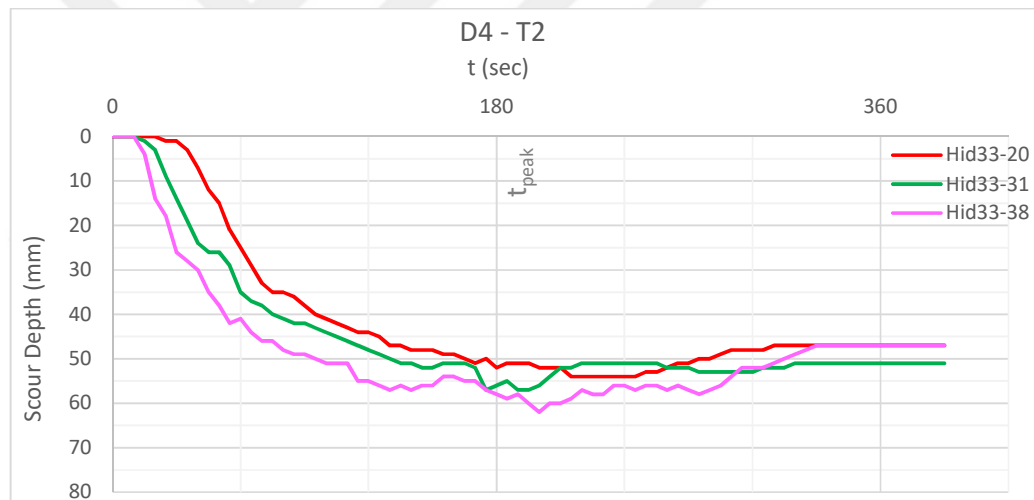


Figure 5.17 Effect of peak discharge for the pier D4/transducer T2

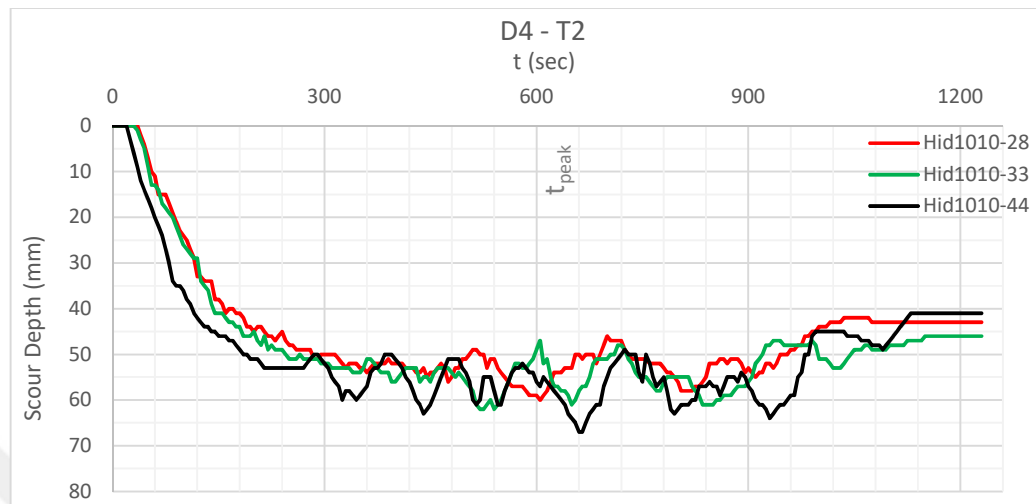


Figure 5.17 continues

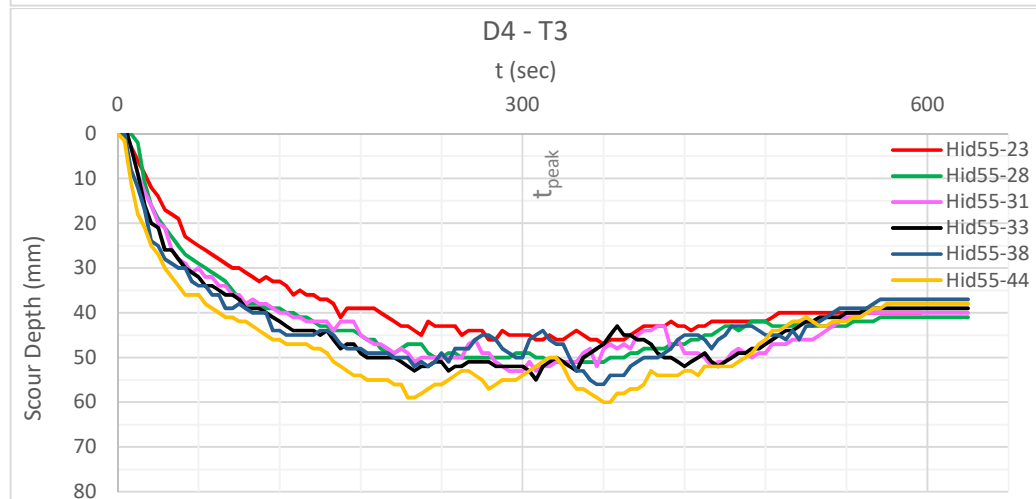
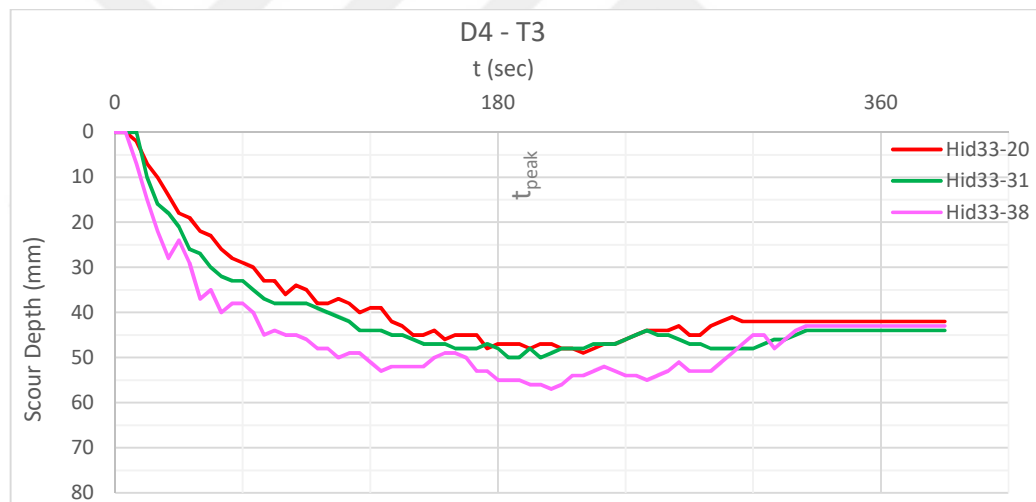


Figure 5.18 Effect of peak discharge for the pier D4/transducer T3

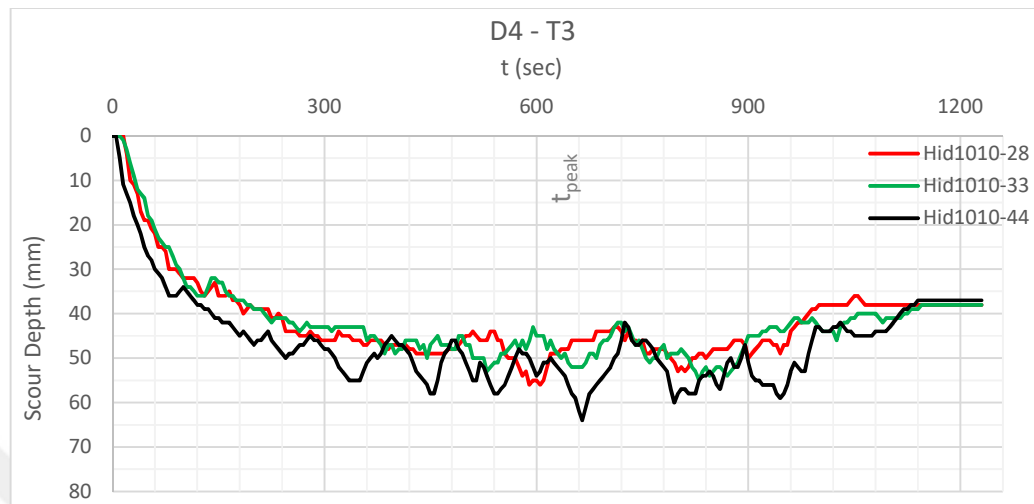


Figure 5.18 continues

5.4.2 Scour Depths for the Pier D8

Figure 5.19, 5.20 and 5.21 illustrate the effect of the peak discharge for the pier of 8 cm diameter at T1, T2 and T3 transducers respectively.

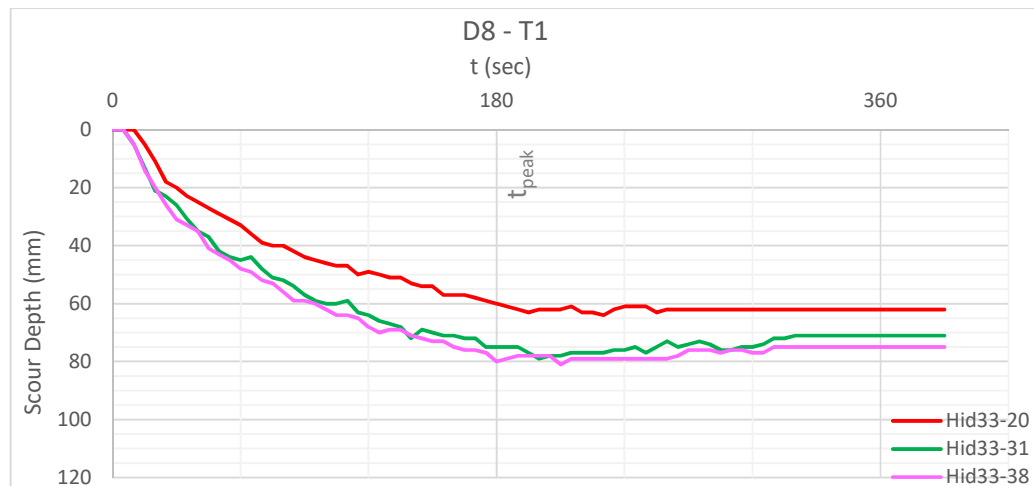


Figure 5.19 Effect of peak discharge for the pier D8/transducer T1

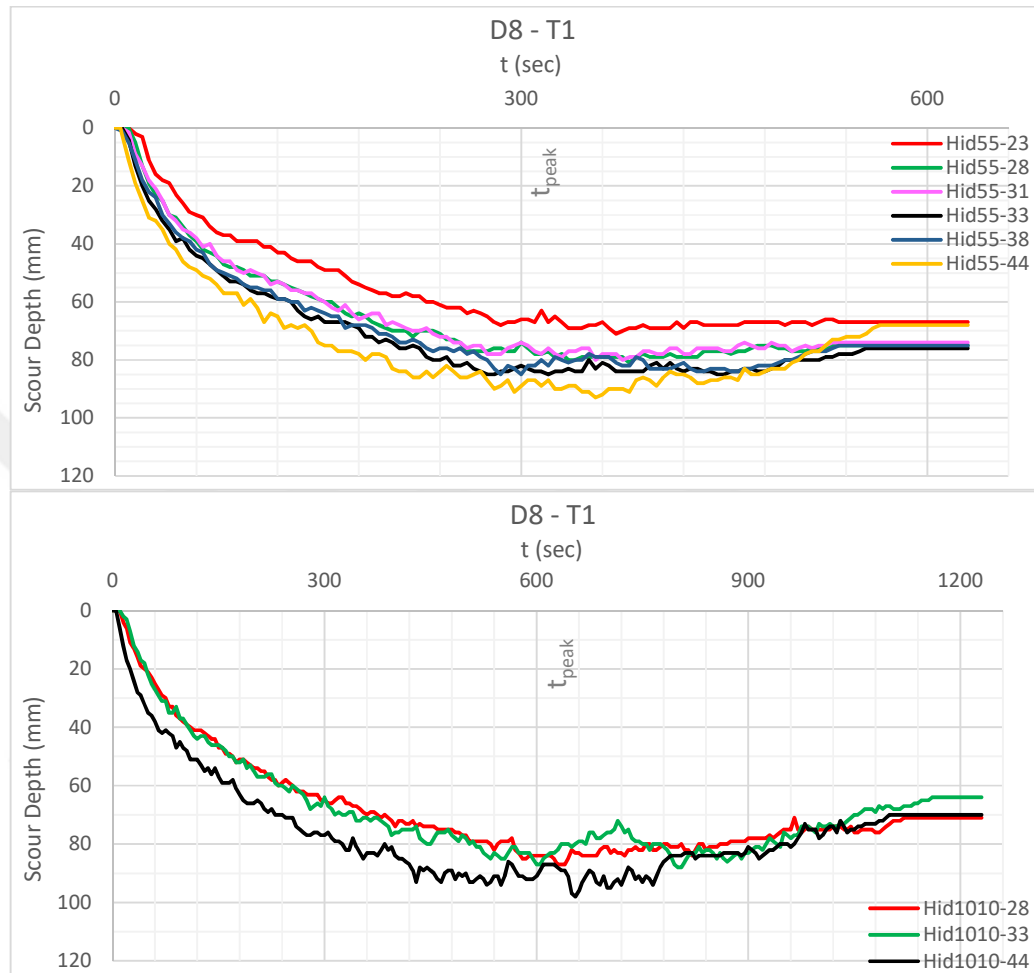


Figure 5.19 continues

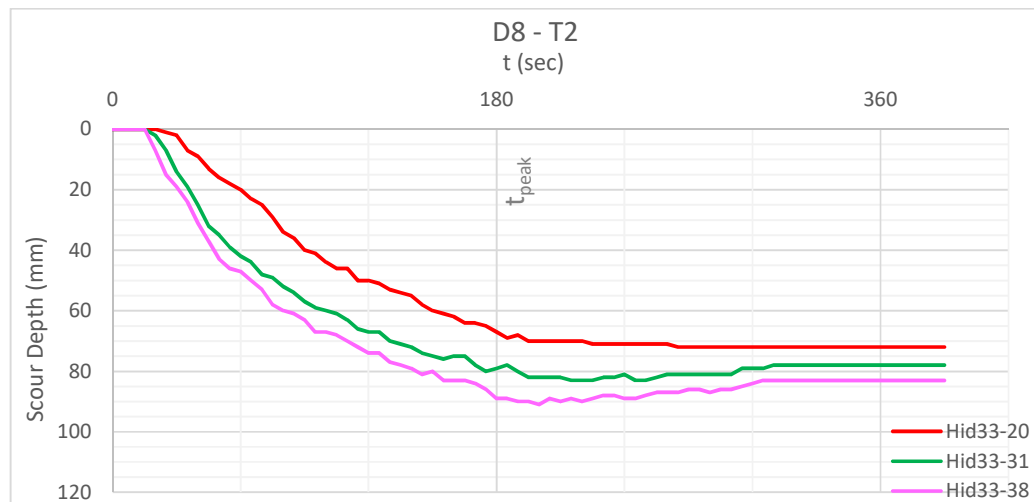


Figure 5.20 Effect of peak discharge for the pier D8/transducer T2

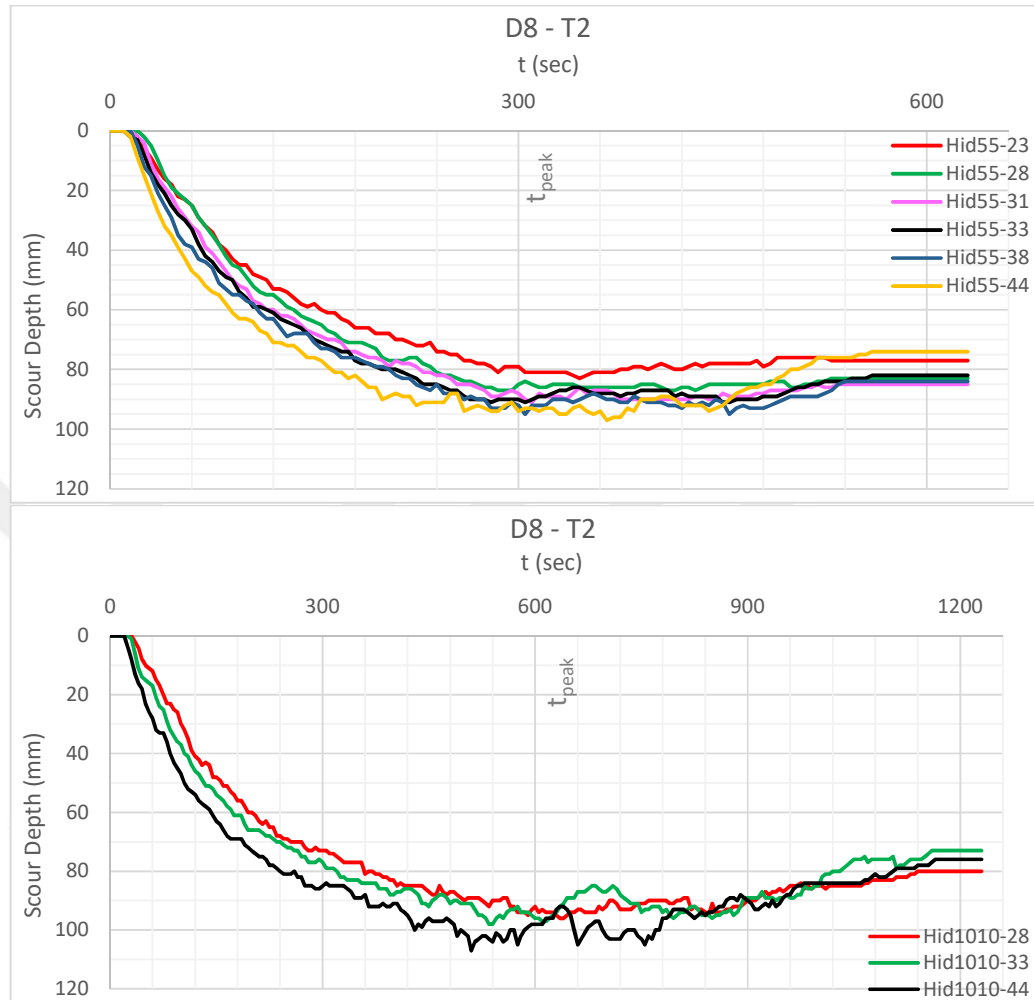


Figure 5.20 continues

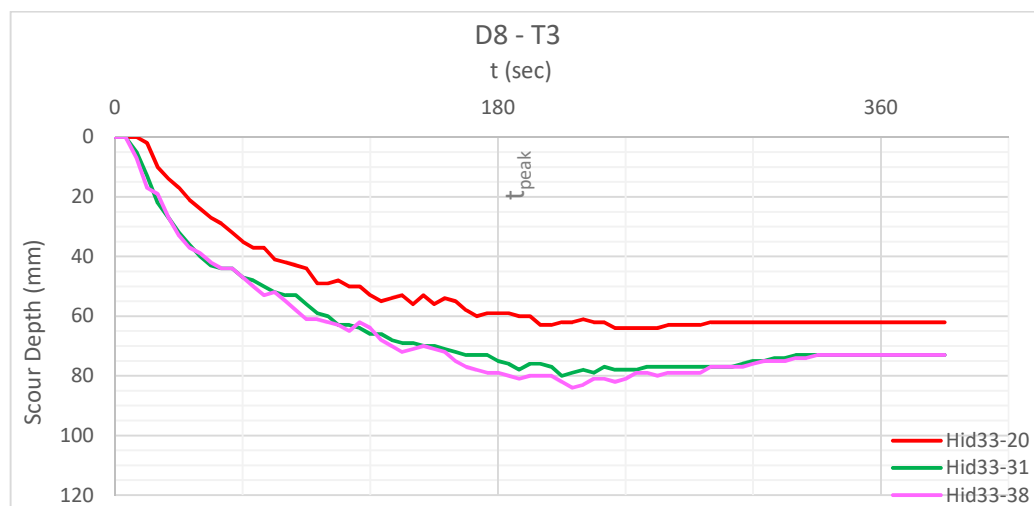


Figure 5.21 Effect of peak discharge for the pier D8/transducer T3

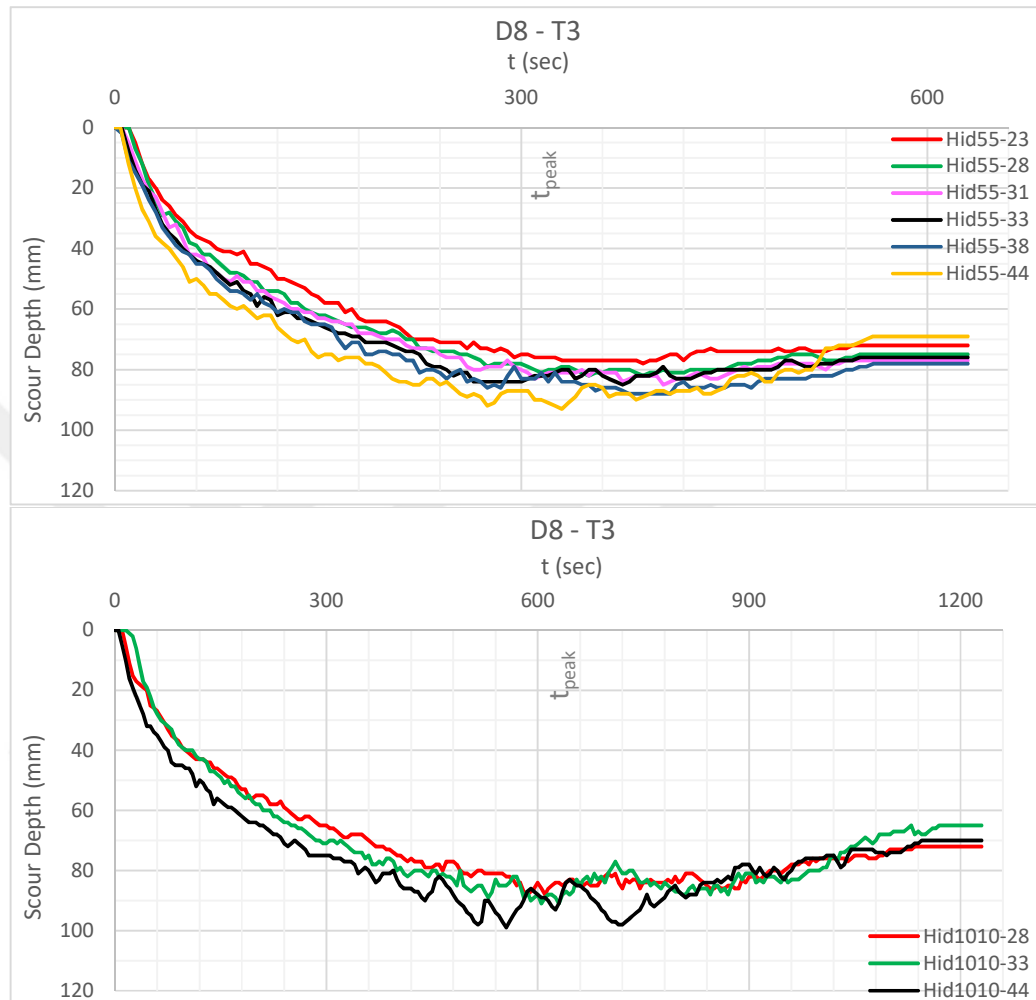


Figure 5.21 continues

5.4.3 Scour Depths for the Pier D11

Figure 5.22, 5.23 and 5.24 illustrate the effect of the peak discharge for the pier of 11 cm diameter at T1, T2 and T3 transducers respectively.

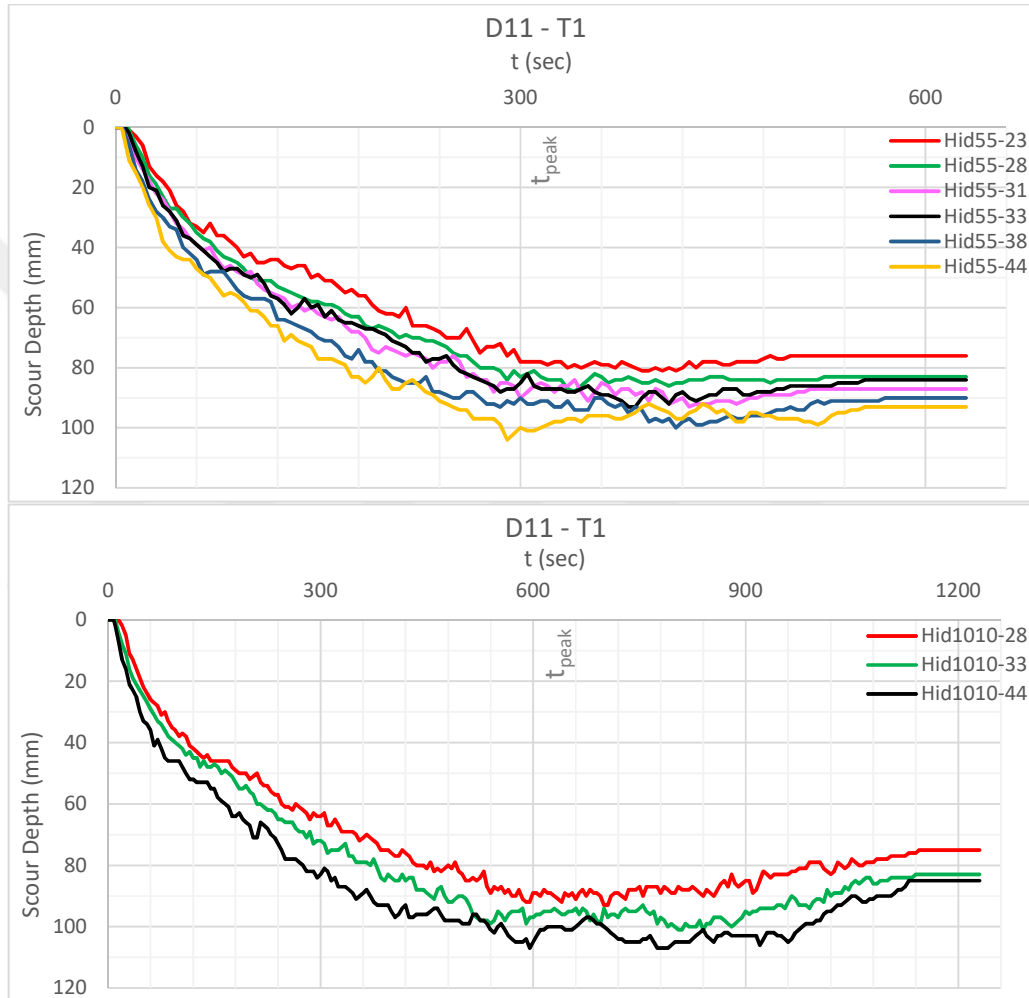


Figure 5.22 Effect of peak discharge for the pier D11/transducer T1

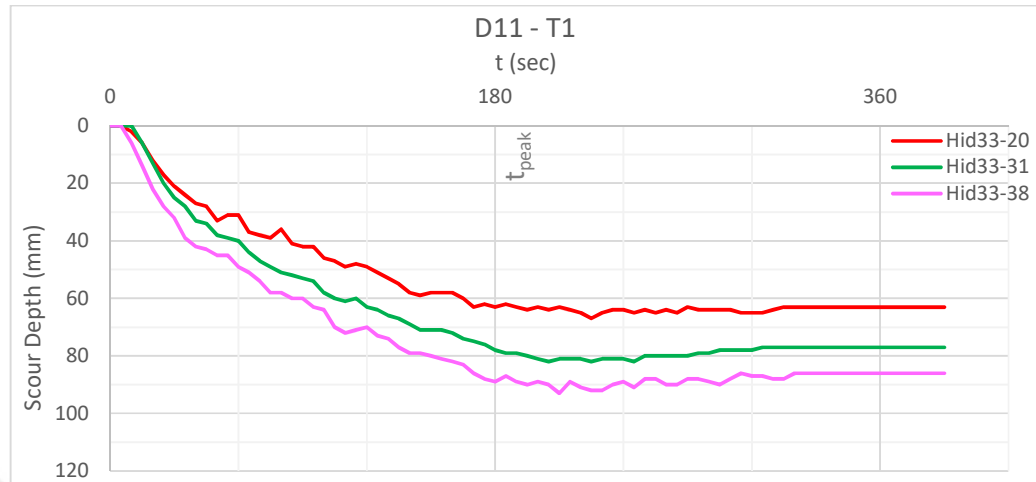


Figure 5.22 continues

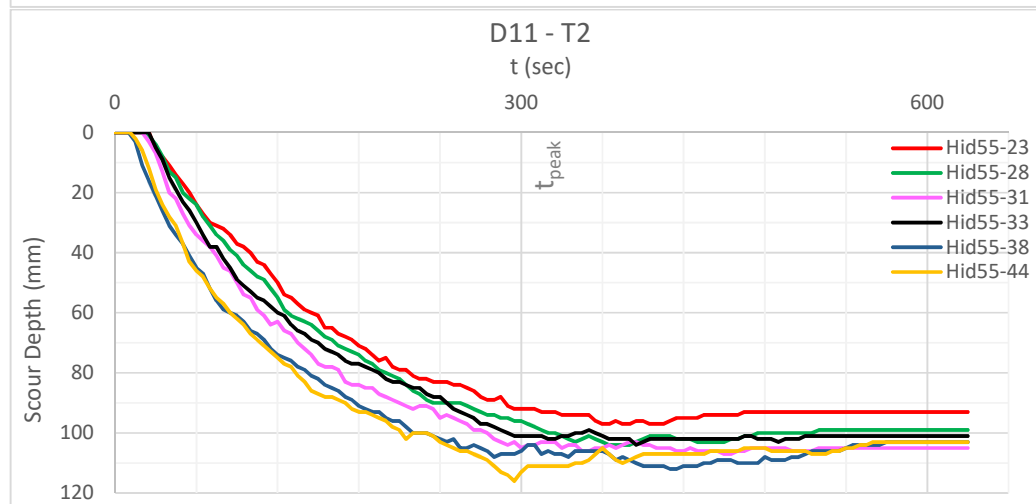
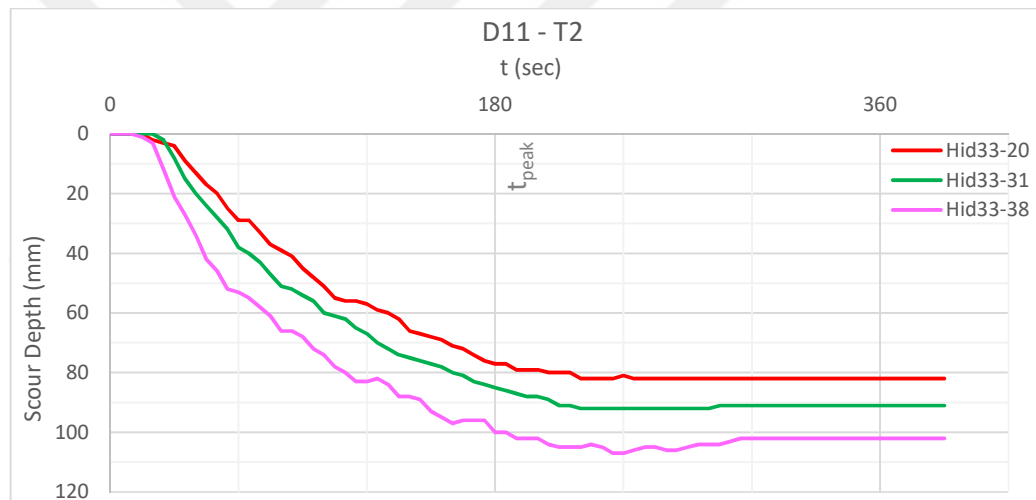


Figure 5.23 Effect of peak discharge for the pier D11/transducer T2

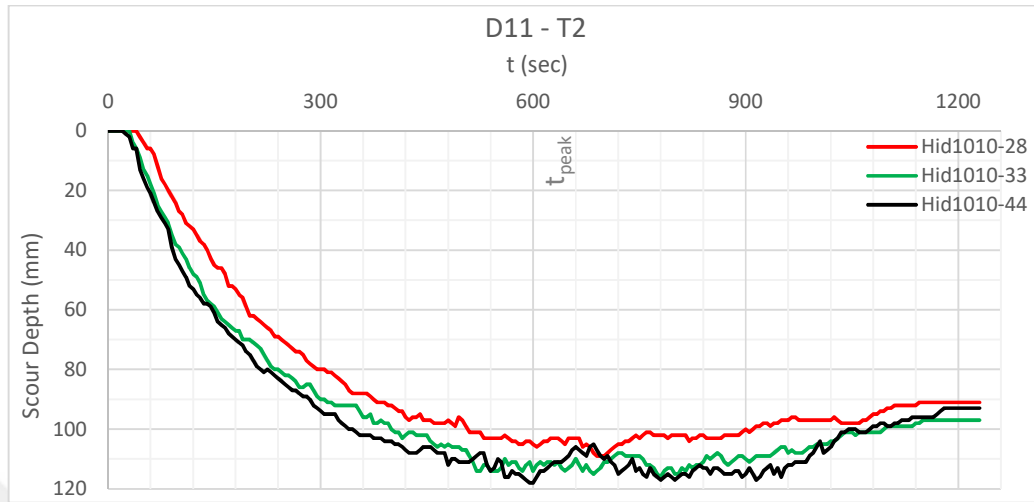


Figure 5.23 continues

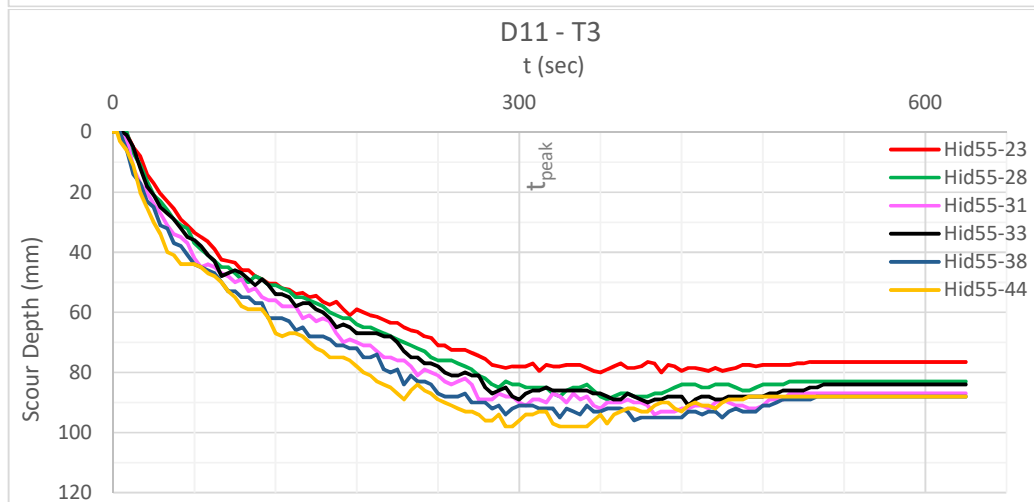
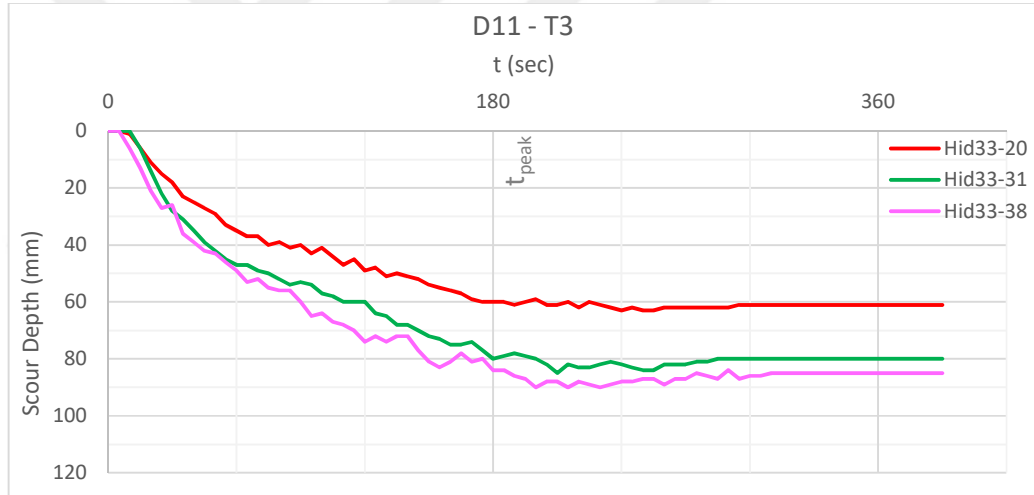


Figure 5.24 Effect of peak discharge for the pier D11/transducer T3

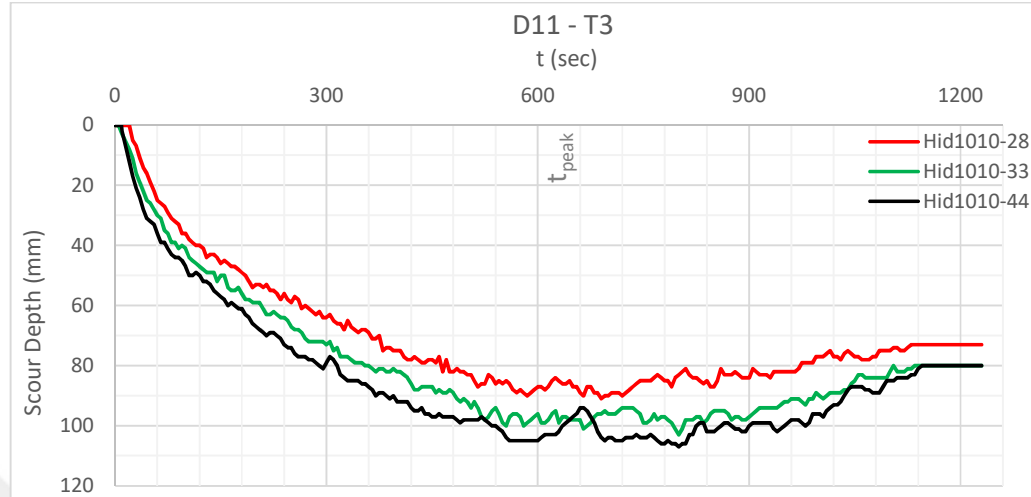


Figure 5.24 continues

Following conclusions were generally observed as a general overview of the effect of peak discharge:

- Scouring developed more rapidly with the high velocities and higher velocities caused greater maximum scour depths.
- Higher flow velocities increased the amount of transported sediment into the scour hole, therefore the magnitudes of scour depth fluctuations increased with increase in flow velocity.
- Greater velocities caused greater rate of sediment transport, so difference between maximum scour depth and final scour depth increased with increase in flow velocity. Thus, high peak discharges provided lower final scour depths at piers with relatively small diameters.

Some incompatibilities observed during the experiments are thought to be a result of the stochastic character of the sediment movement.

5.5 Effect of Pier Diameter and Approaching Flow Depth

Pier diameter directly affected the influence of supplied sediment on scour depth. As it is known and also observed in this study, increase of pier diameter caused increase in scour depth and in size of scour hole.

Although the amount of transported sediment was the same for a given hydrograph, it was the size of scour hole which determined the effect of supplied sediment on scour depth. In case of Hid36-31 the final sizes of the scour holes for the different piers are given in Figure 5.25, as illustrative example.

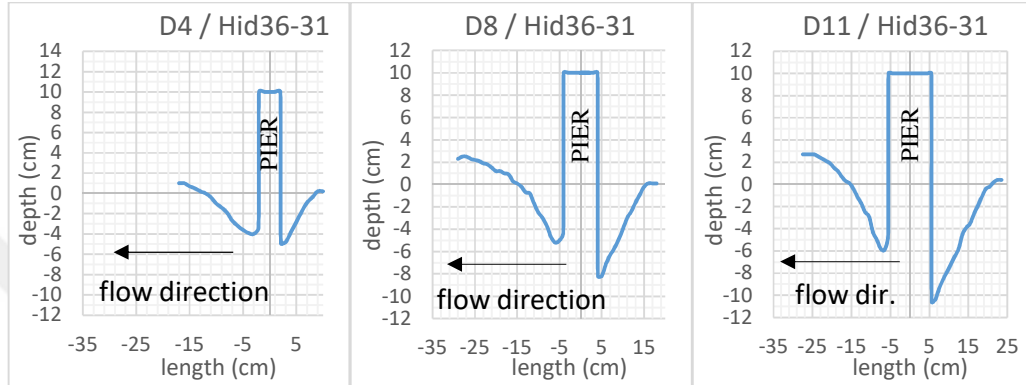


Figure 5.25 Final sizes of the scour holes for different piers

The effect of pier diameter can also be observed in Figures 5.5 to Figure 5.24. Both fluctuation heights and difference between maximum and final scour depths decreased with increase in pier diameter. Size of scour holes at different instants of the experiments performed under Hid55-44 for the piers with 4 cm, 8 cm and 11 cm are given in Figure 5.26 a, b, c and d, respectively as an illustrative example.



Figure 5.26 a, b, c and d) Sizes of the scour holes for different piers under Hid55-44 (Personal Archive, 2018)

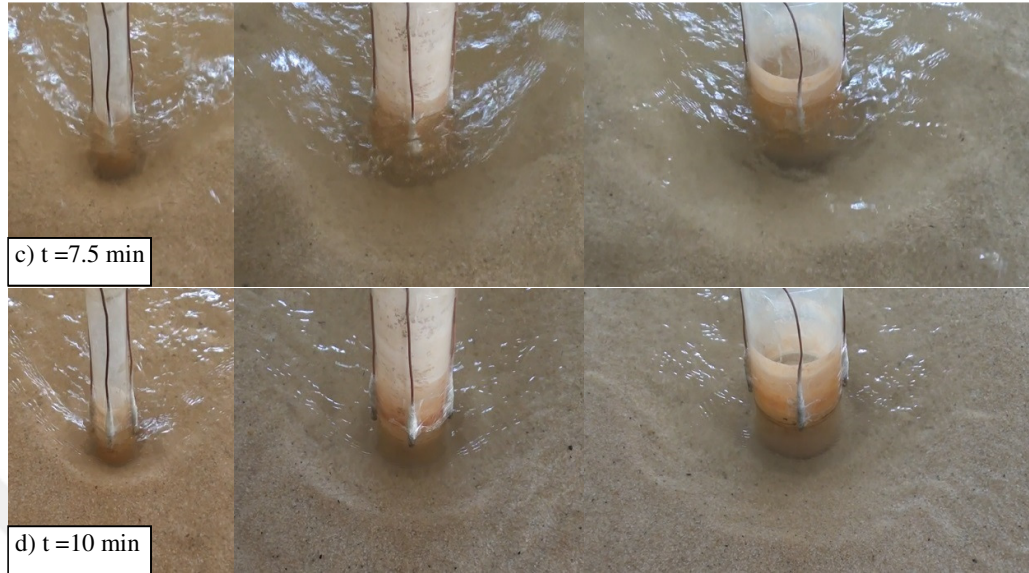


Figure 5.26 continues

A closer look to Figure 5.3 shows that at the beginning of the experiment, scour depth curves of different piers share a common line for a while. First, the curve related to the pier D4 leaves the common line, and then the pier D8. Since all other parameters but the pier width are same for scour process, relationship between pier width and flow depth must play role in this behavior. It was found that this common line lasted as long as $y/b < 0.72$ at flanks. At front of the pier, the curve related to the pier D4 left the common line after $y/b = 0.97$ and this ratio equals $y/b = 0.61$ in the case of the pier D8. It seems that scour depth was developed independently from the pier width below these values. More study with different pier widths and flow types will be useful for the clarification of this point.

5.6 Effect of Relative Grain Size

Three different relative grain sizes (b/d_{50}) were used in this study, namely 24.5, 49.9 and 67.4 for the piers D4, D8 and D11, respectively. Ettema (1980) and Melville (1997) suggested that grain size loses its effect on scour depth after $b/d_{50} > 25$, which is satisfied for all the piers used. Chiew (1984) suggested this value as 50 as a result of his live bed experiments. This criterion is satisfied for the piers D8 and D11.

5.7 Bed Forms and Scour Hole Profile

2D dune formations were observed during all stages of the experiments with variable height and wavelength depending on flow parameters. No ripple formation was observed. A sample picture of bed formations is given in Figure 5.27 a. 2D dune formations are consistent with the experiments conducted in Tsukuba University (Figure 5.27 b).

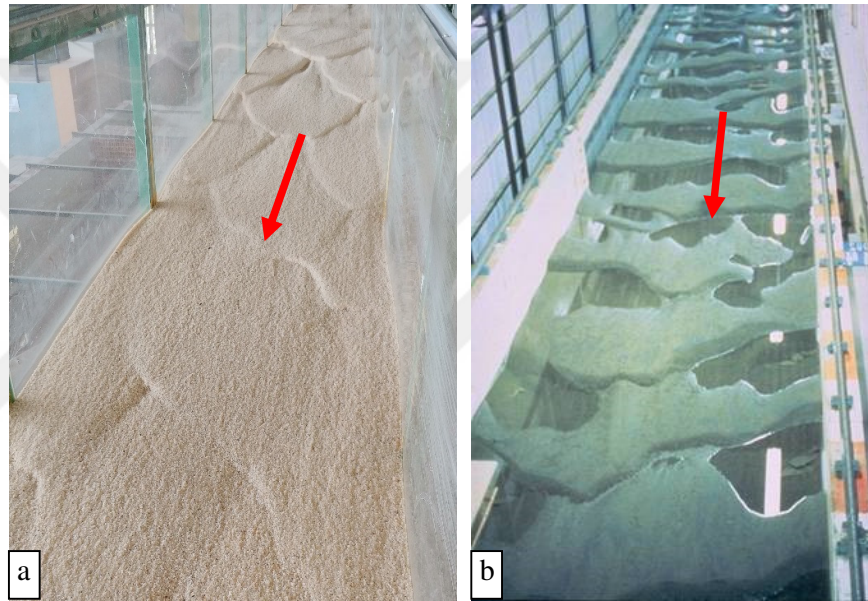


Figure 5.27 a) and b) 2D dune forms of sighted in present experiments and experiments in Tsukuba University (Parker, 2006)

The final longitudinal profile for the pier D11 in the case of Hid55-44 is presented in Figure 5.28. This profile was generated by means of a laser meter. Measurement started at upstream side of scour hole and stopped when the crest height at downstream became constant. The profiles concerning all other piers and hydrographs are given in Appendix 2.

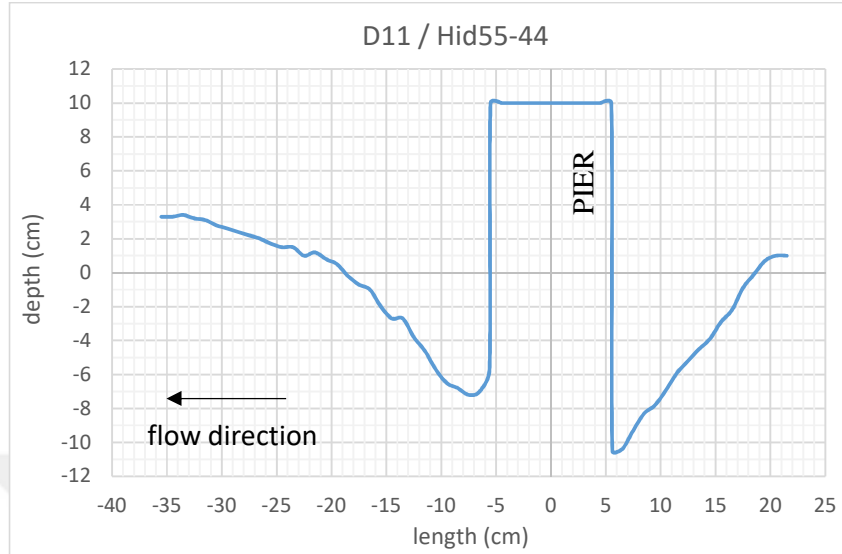


Figure 5.28 Scour hole profile of the pier with 11 cm diameter under Hid55-44

The origin corresponds to the undisturbed bed elevation measured before the experiments. In Figure 5.28, a dune cluster with height of 1 cm is seen at upstream edge of the scour hole at the end of the experiment. Upstream slope of the scour hole is approximately 0.76 which corresponds to an angle of 37° . Melville (1975) stated that this angle is equal to dynamic angle of repose of the bed material. Scoured bed material around the pier was entrained by wake vortices to downstream. A mound was formed at downstream by entrained bed material with a crest about 3-4 pier diameters away from the pier.

5.8 Time Scale

Time scale of maximum scour depths in front of the pier were calculated by means of slope method for each hydrograph and pier. First a horizontal line was drawn, starting from maximum scour depth and to y axis. Then a tangent line was drawn, starting from initial scour depth and intersected with this horizontal line at point A. Last, a vertical line starting from initial scour depth was intersected with the horizontal line at point B. Time scale (T_s) of maximum scour depth was determined as the distance between point A and B. An illustrative example is given in Figure 5.29 for Hid33-55 hydrograph and the pier D4. All measured time scales for all hydrographs and transducers are given in Table 5.3. It was found that time scale of maximum scour

depth increased when time to peak or pier diameter increased and decreased if the peak flow velocity increased.

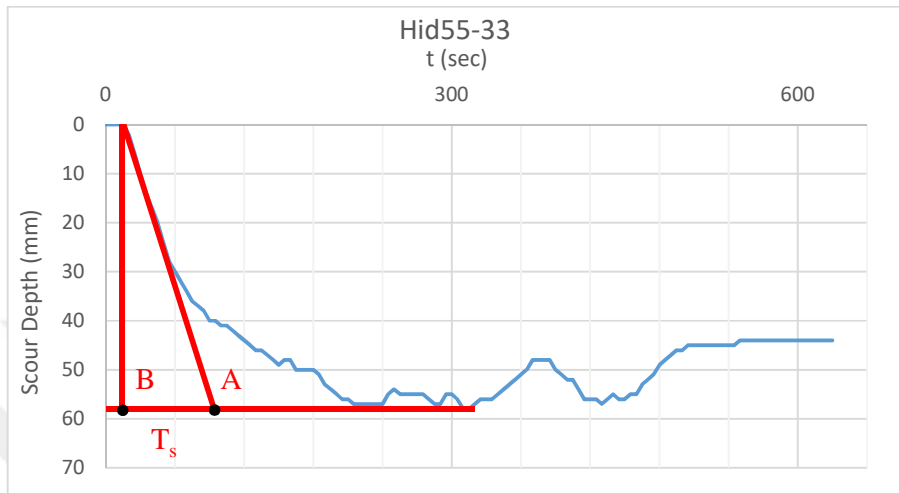


Figure 5.29 Slope method for calculating time scale

Table 5.3 Measured time scales for each pier and hydrograph

Hydrograph	$T_{s,D4}$ (sec)	$T_{s,D8}$ (sec)	$T_{s,D11}$ (sec)
Hid33-20	66.43	124.23	90.63
Hid33-31	56.39	75.94	87.69
Hid36-31	63.29	84.85	92.11
Hid39-31	62.07	84.90	103.80
Hid33-38	46.23	76.03	75.43
Hid55-23	121.98	137.99	146.79
Hid55-28	87.04	137.41	157.46
Hid55-31	85.74	136.65	115.00
Hid55-33	86.47	111.96	119.86
Hid55-38	75.24	97.67	104.80
Hid55-44	60.02	92.37	105.71
Hid1010-28	148.25	208.04	210.65
Hid1010-33	164.53	190.81	180.39
Hid1010-44	133.36	183.37	184.10
Hid1515-44	172.48	195.56	189.08
Hid2020-44	179.44	285.90	251.98

5.9 Derived Empirical Equations

Root Mean Square Error (RMSE) was adopted to derive empirical equations in the regression analysis. First, each predicted value (Y_i) subtracted from corresponding measured value (X_i) and squares of these values were summed to (SSE) in Eq. (5.1). RMSE was calculated from Eq. (5.2) by dividing SSE to data length (n).

$$SSE = \sum_i^n (Y_i - X_i)^2 \quad (5.1)$$

$$RMSE = \sqrt{SSE/n} \quad (5.2)$$

It was aimed to minimize RMSE by means of MS Excel solver. Scatter index (SI) was calculated by Eq. (5.3) to determine the meaningfulness of regression analyses. $\bar{X}$ indicates the average of measured values.

$$SI = RMSE / \bar{X} \quad (5.3)$$

5.9.1 Predicting Time Scale

Strong correlation was observed with the parameters peak flow intensity (V/V_c)_p and dimensionless time to peak discharge (T_p). An equation was derived to predict dimensionless time scale of maximum scour depth:

$$T_s^* = 3.74 \left(\frac{V}{V_c} \right)_{(p)}^{-2.24} T_p^{0.717} \quad (5.4)$$

Dimensionless time to peak discharge can be obtained from Eq. (5.5). Dimensionless time scale of maximum scour depth obtained from Eq. (5.4) can also be transformed to exact time scale of maximum scour depth with Eq. (5.5)

$$T = \frac{\sqrt{g(s-1)d_{50}}}{b} t \quad (5.5)$$

Coefficients for determination (R^2) were calculated for the piers with 4, 8 and 11 cm as 0.95, 0.93 and 0.92 respectively. Measured and calculated time scale values according to Eq. (5.4) were plotted and given with their scatter indices according to the piers in Figure 5.30. $T_{s,c}$ and $T_{s,m}$ indicate calculated and measured time scale respectively.

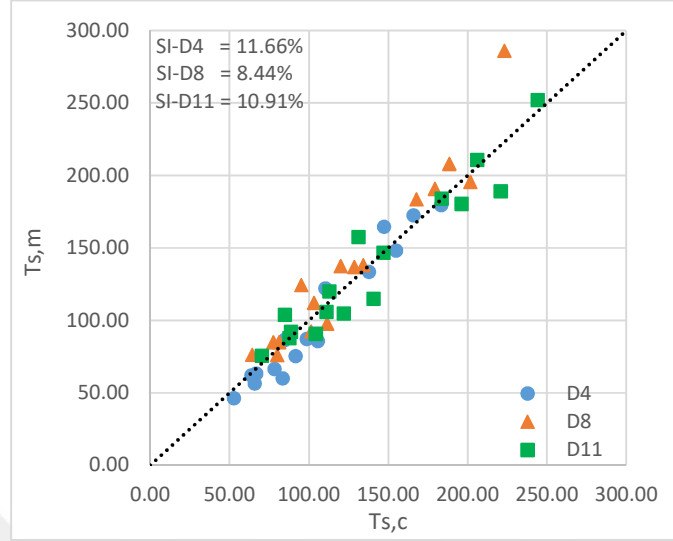


Figure 5.30 Measured and calculated time scales

5.9.2 Predicting Temporal Scour Depth

With 16 different hydrographs, 3 different pier diameters and timestep of 5 seconds, huge amount of data was studied during regression analysis in order to derive an equation for temporal live bed scour depth. Various combinations of parameters effecting scour and their mathematical transformations such as logarithmic or exponential were also tested. Consequently, flow intensity (V/V_c), dimensionless flow depth (y/b) and dimensionless time were chosen as primary parameters. Effect of dimensionless grain size (b/d_{50}) was also investigated with various combinations. The latter was totally ineffective without logarithmic transformation. With logarithmic transformation, SSE decreased about 1% percent. Melville (1997) stated that the parameter b/d_{50} becomes ineffective if $b/d_{50} > 25$ after all. In this study, b/d_{50} values for the piers with diameters of 4 cm, 8 cm and 11 cm are 24.5, 50 and 67.4 respectively. Therefore the parameter b/d_{50} is disregarded. Another issue is the change of scour behavior after peak discharge. Therefore, the following equation was derived with the purpose of predicting temporal scour depth within the range of $0 \leq t \leq t_p$.

$$\frac{d(t)}{b} = 1.43 \left(\frac{V}{V_c} - I_{vc} \right)^{0.37} \left(\frac{y}{b} \right)^{0.36} (1 - e^{-0.0077t}) \quad (5.6)$$

where, I_{vc} is the threshold value of flow intensity that causes the incipient scouring around pier and it was found 0.6 in experiments. This value was suggested as 0.5 by

Breusers et al. (1977) and Raudkivi (1986), 0.4 by Melville & Chiew (1999), Chang et al. (2004) and Lai et al (2009). Gao et al. (1993) stated that I_{vc} can change between 0.4 and 0.6. T is dimensionless time parameter, obtained from Eq. (5.5) for each timestep of flood. Reason of stating time parameter by an exponential function is; when $t = 0$, $d(t) = 0$; with continuously increasing flow velocity during the rising limb of a flood, when t_p approaches to ∞ , $d(t) = 1$. This expression of time parameter was also used for temporal scour below pipelines by Dogan and Arisoy (2014).

The scour depths calculated from Eq. (5.6) were compared with those obtained from the experimental ones. In Figure 5.31, d_m and d_c indicate the measured and calculated time dependent scour depths, respectively.

The measured scour depths were also compared with those obtained from the following equation proposed by Oliveto & Hager (2002);

$$Z = 0.068 \cdot N \cdot \sigma^{-1/2} \cdot F_d^{1.5} \log(T) \quad (5.7)$$

where, Z is dimensionless scour depth ($Z = z(t)/L_R$), N is shape factor (K_s in this study) and 1 for circular piers, F_d is densimetric Froude number $F_d = V/(g'd_{50})^{1/2}$, T is dimensionless time ($T = [(g'/d_{50})^{1/2}/L_R]t$), $g' = (\rho_s - \rho_f)g$ and $L_R = (b^{2/3}y^{1/3})$.

The dimensionless scour depths were converted to scour depths and so obtained scour depths are plotted in Figures given in Appendix 3, together with those obtained from the experiments and the derived equation (5.6). In these figures, d_m denotes the experimental scour depth, d_{c1} and d_{c2} indicate the scour depths calculated from Eq. (5.6) and Eq. (5.7), respectively.

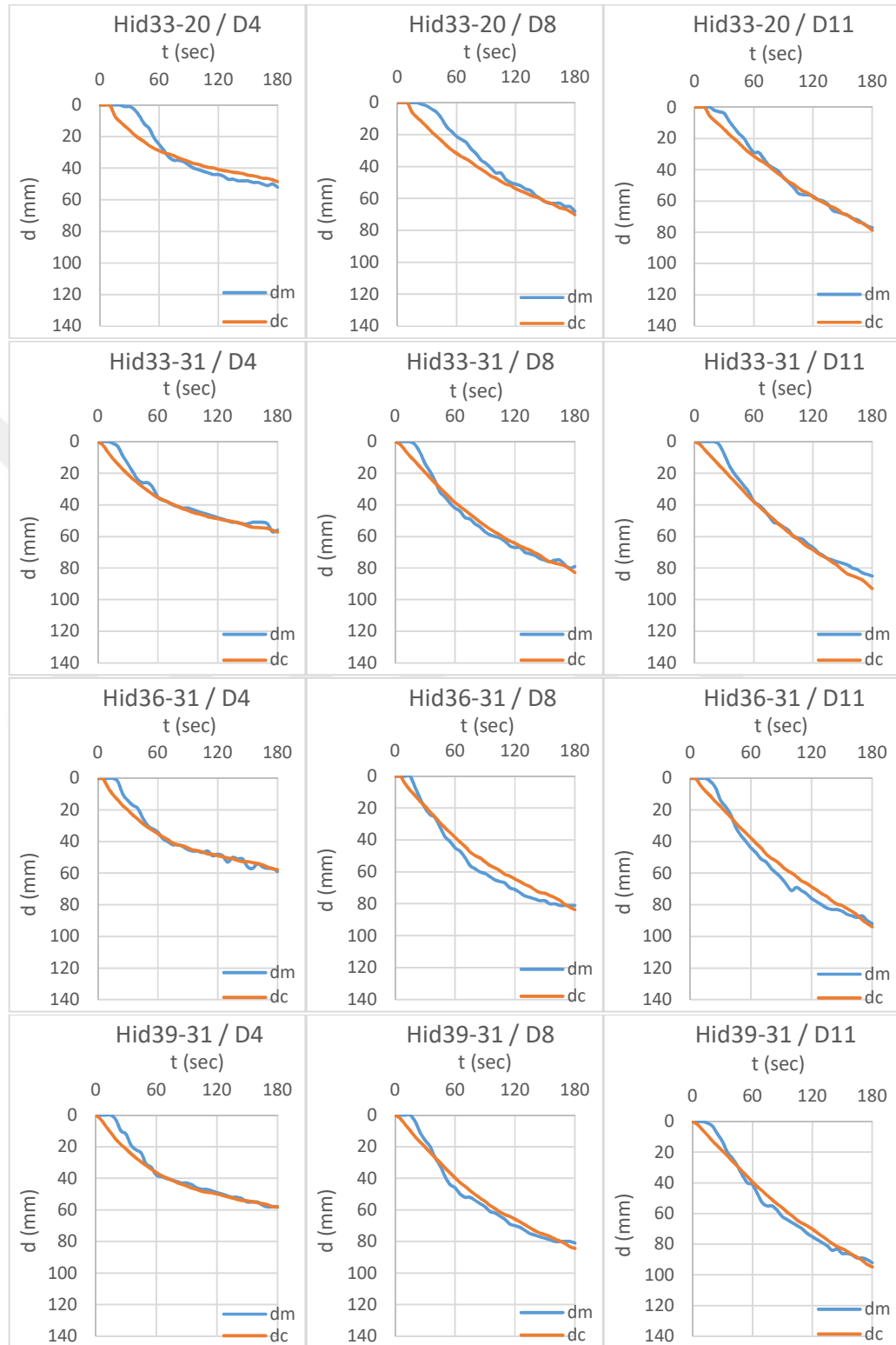


Figure 5.31 Comparison of calculated scour depths with measured scour depths

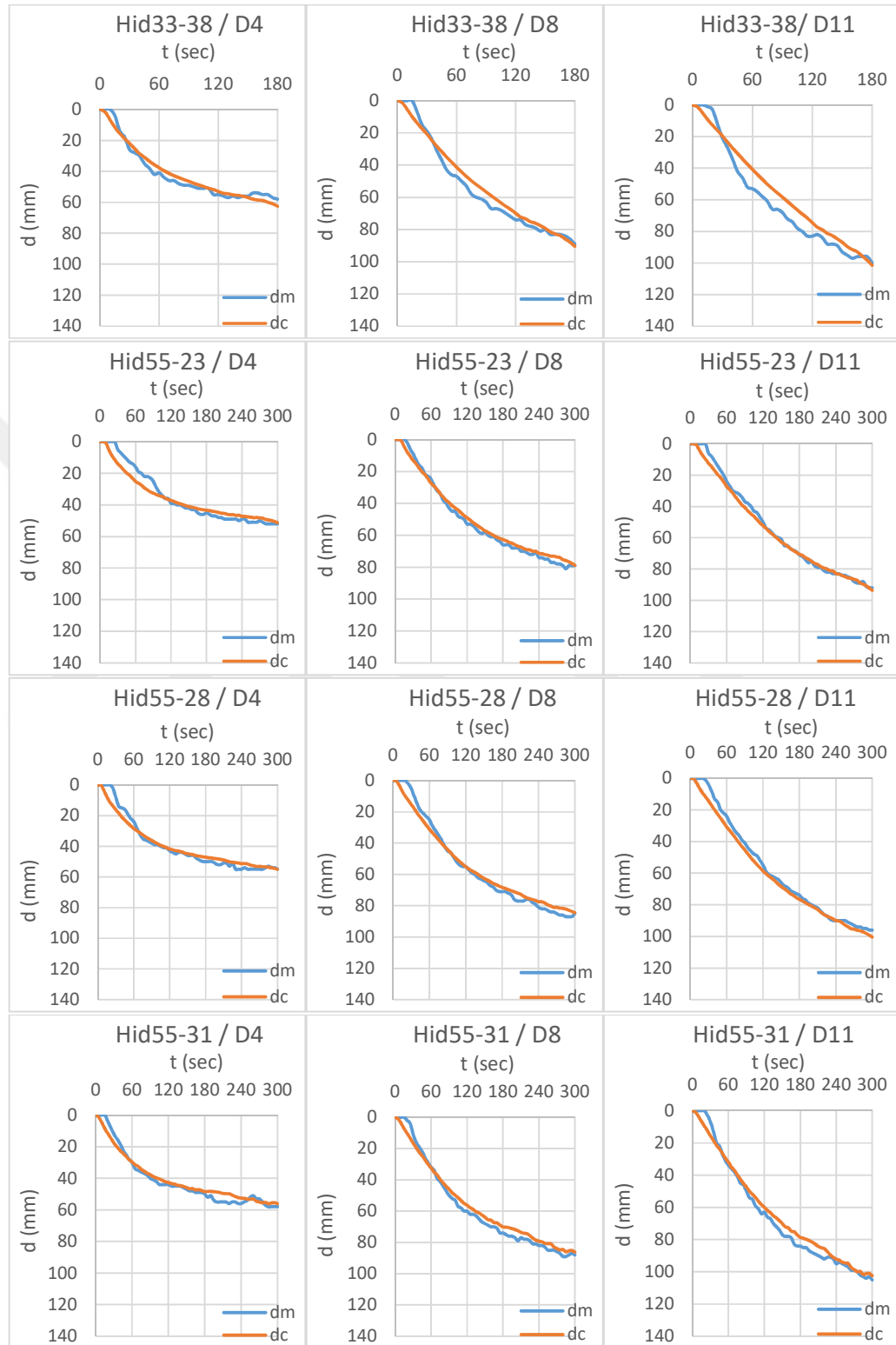


Figure 5.31 continues

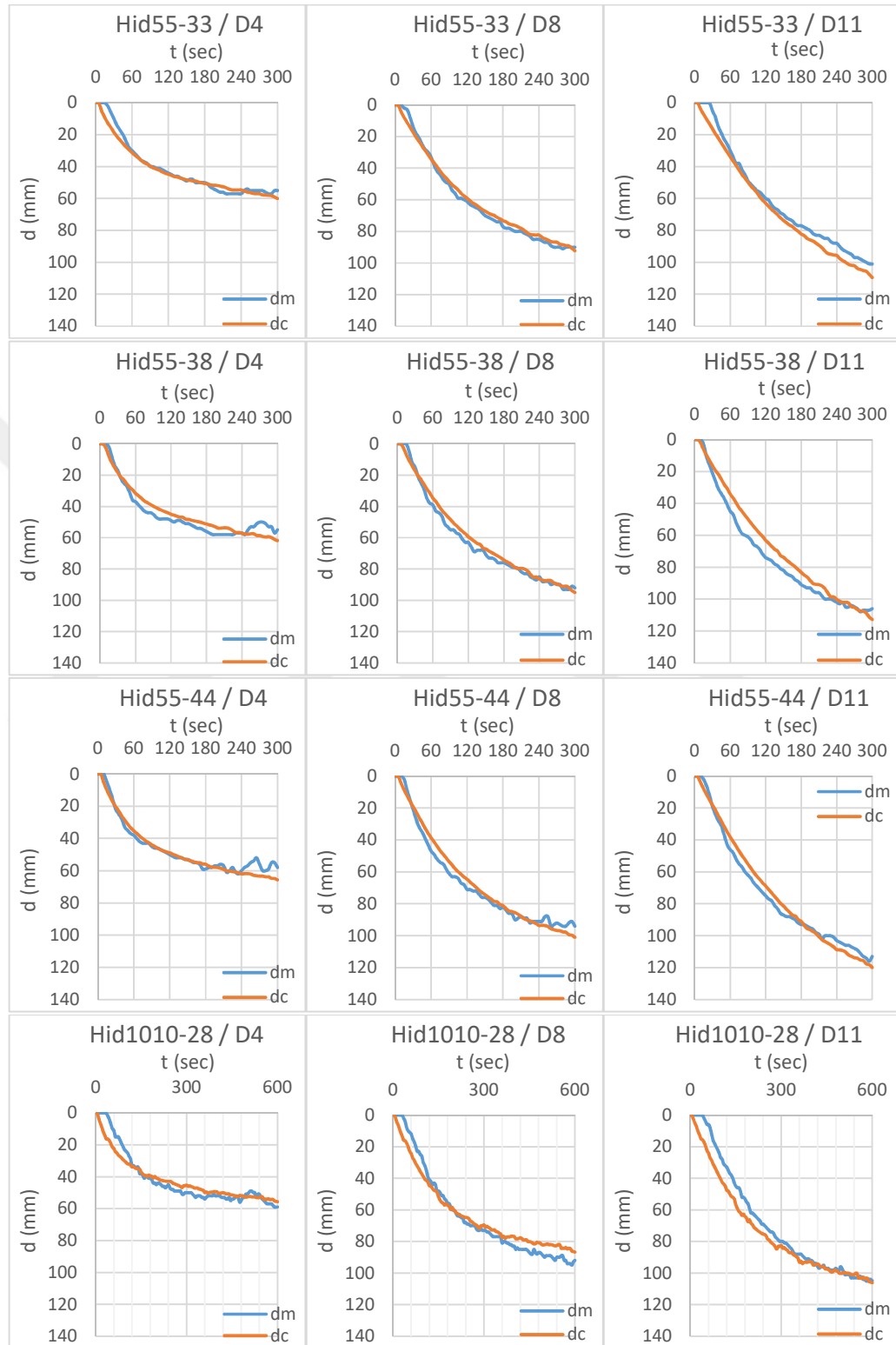


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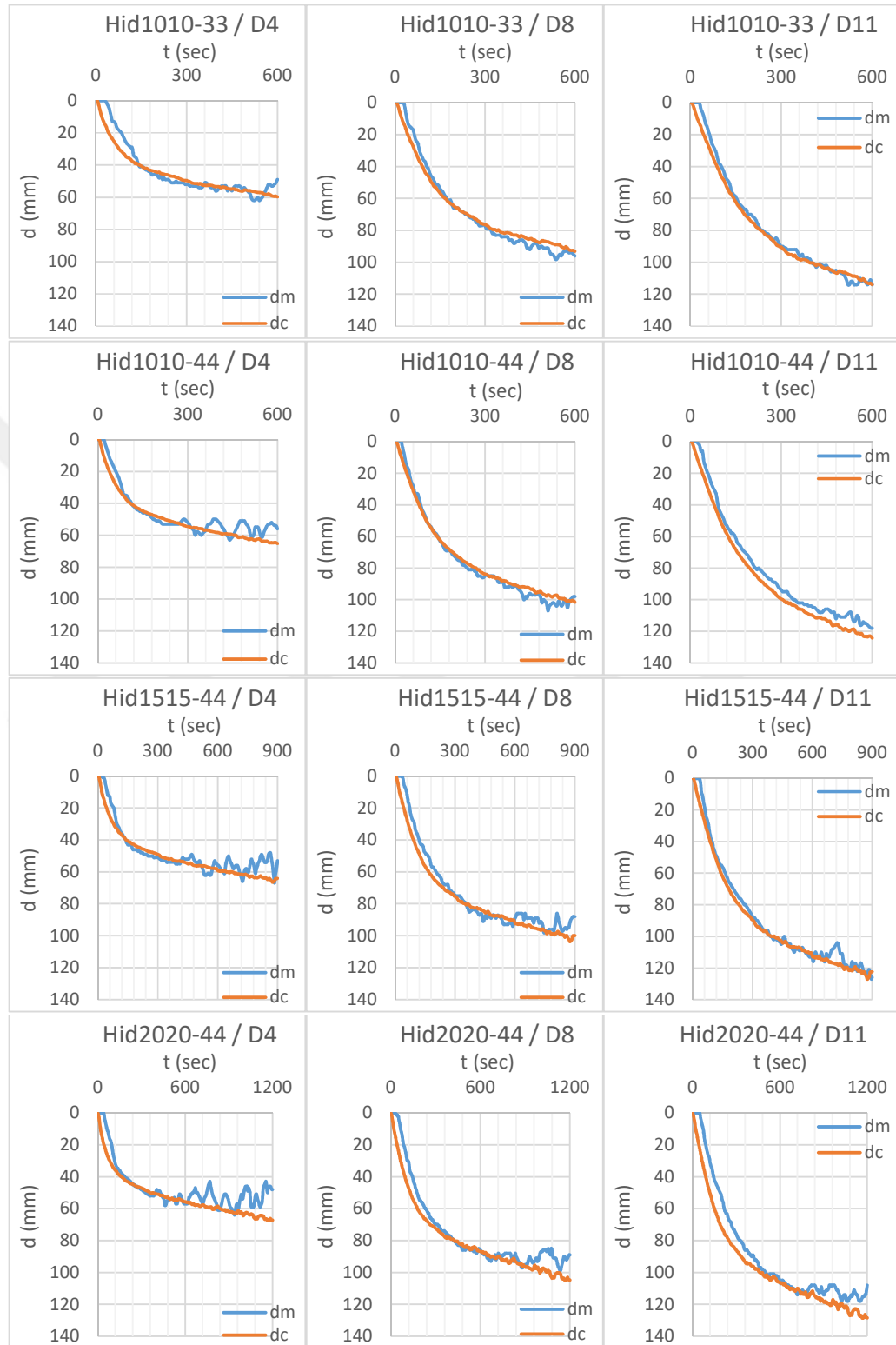


Figure 5.31 continues

Calculated and measured dimensionless scour depths (indicated as dc/b and dm/b , respectively) were given in Figure 5.32 for each pier and hydrograph with their scatter index (SI) and determination coefficients (R^2).

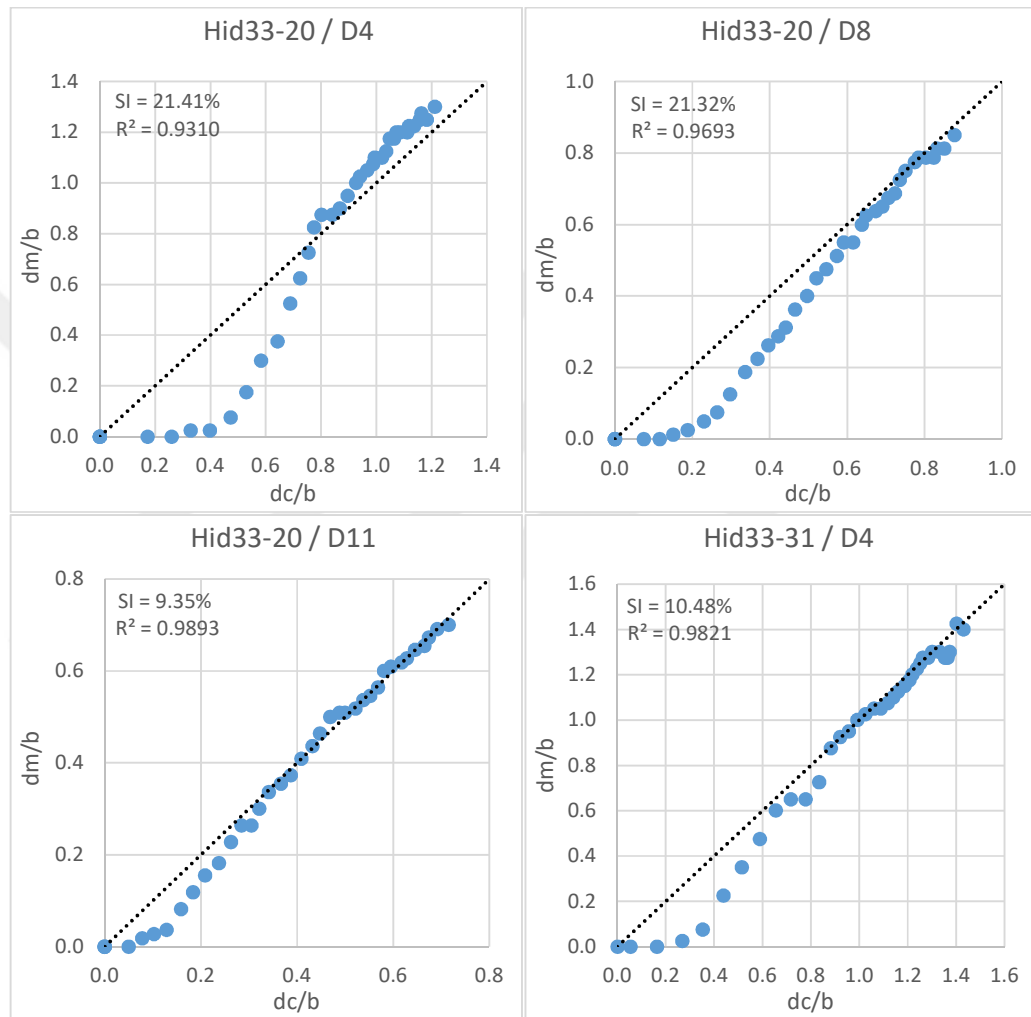


Figure 5.32 Measured and calculated temporal scour depths

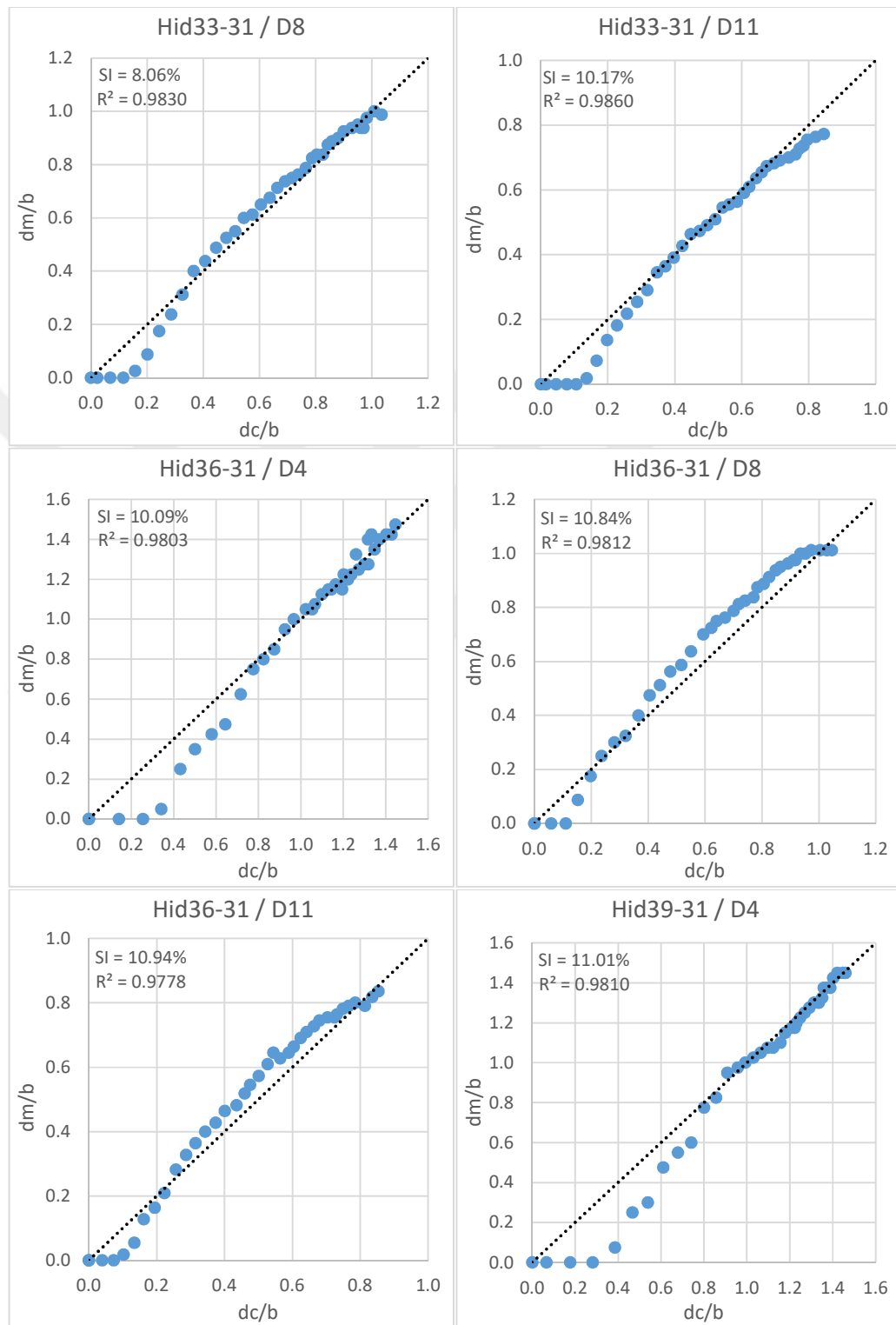


Figure 5.32 continues

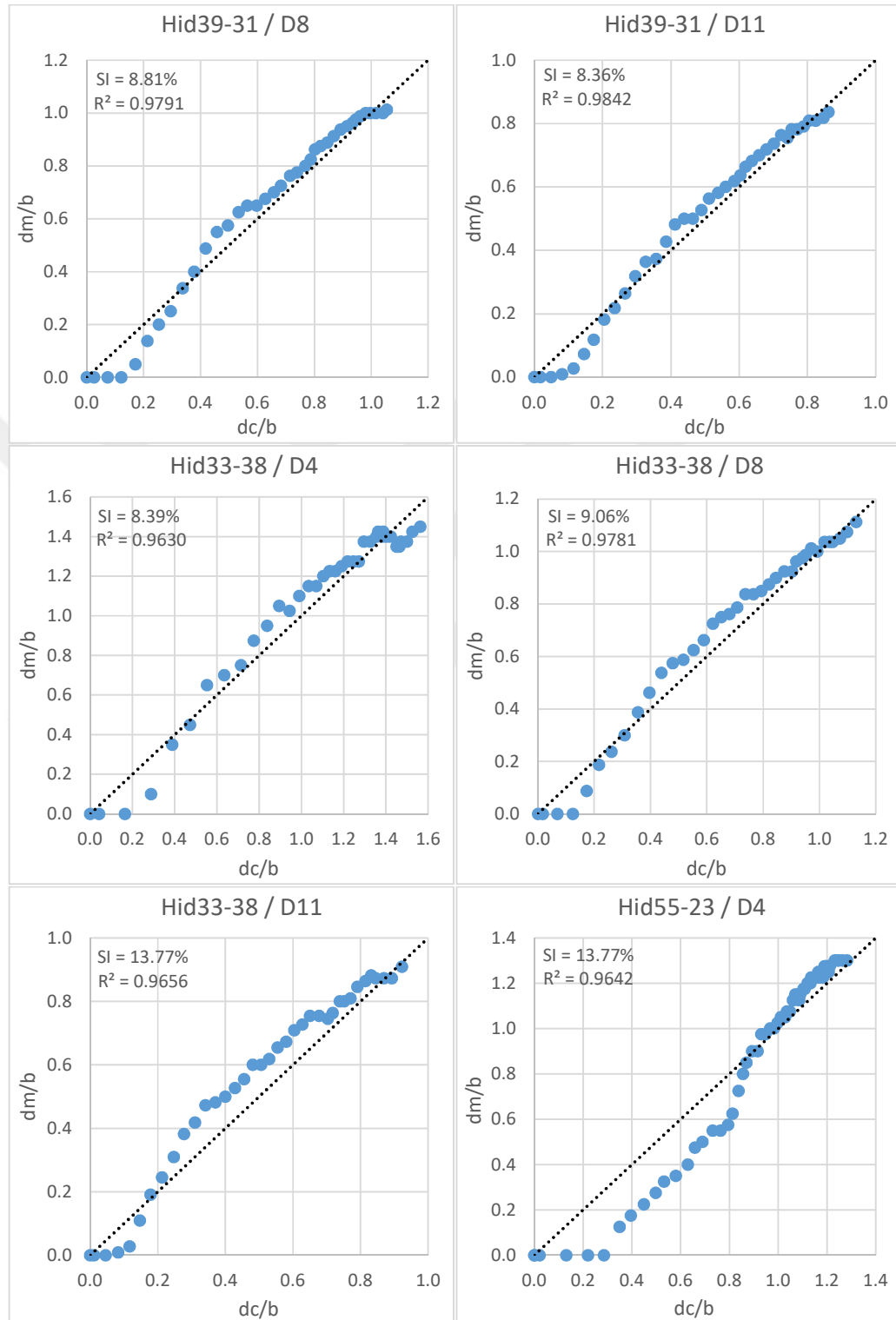


Figure 5.32 continues

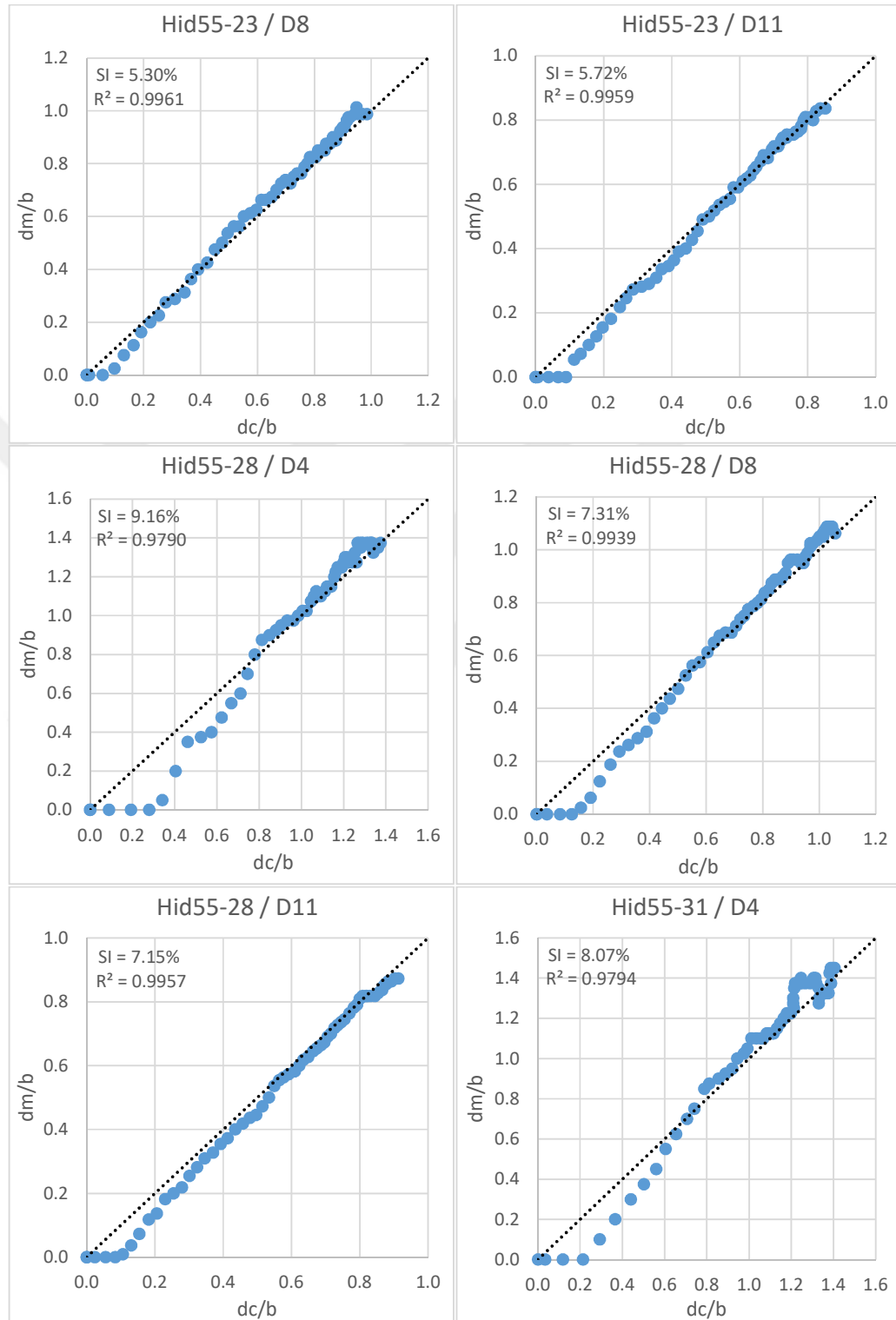


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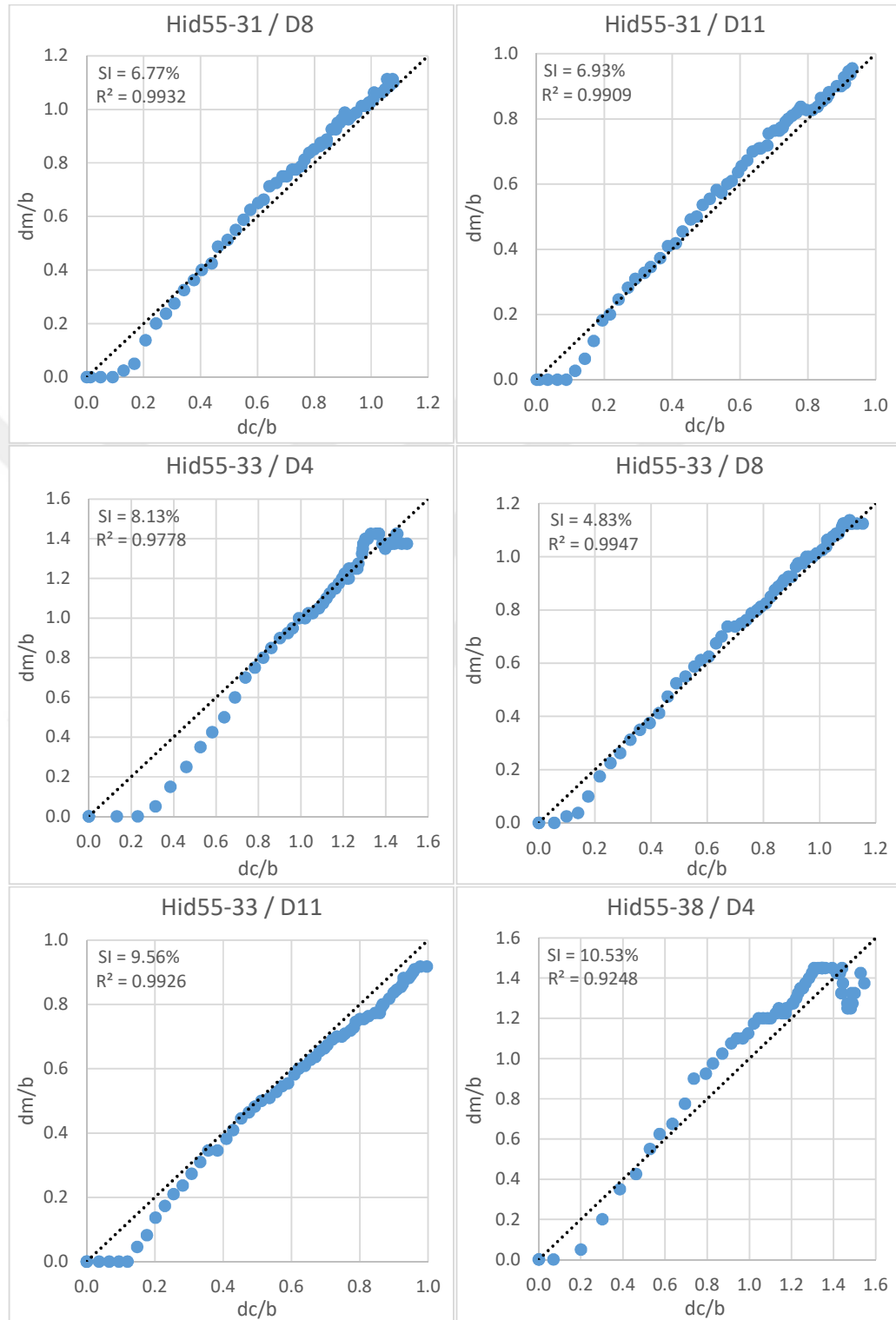


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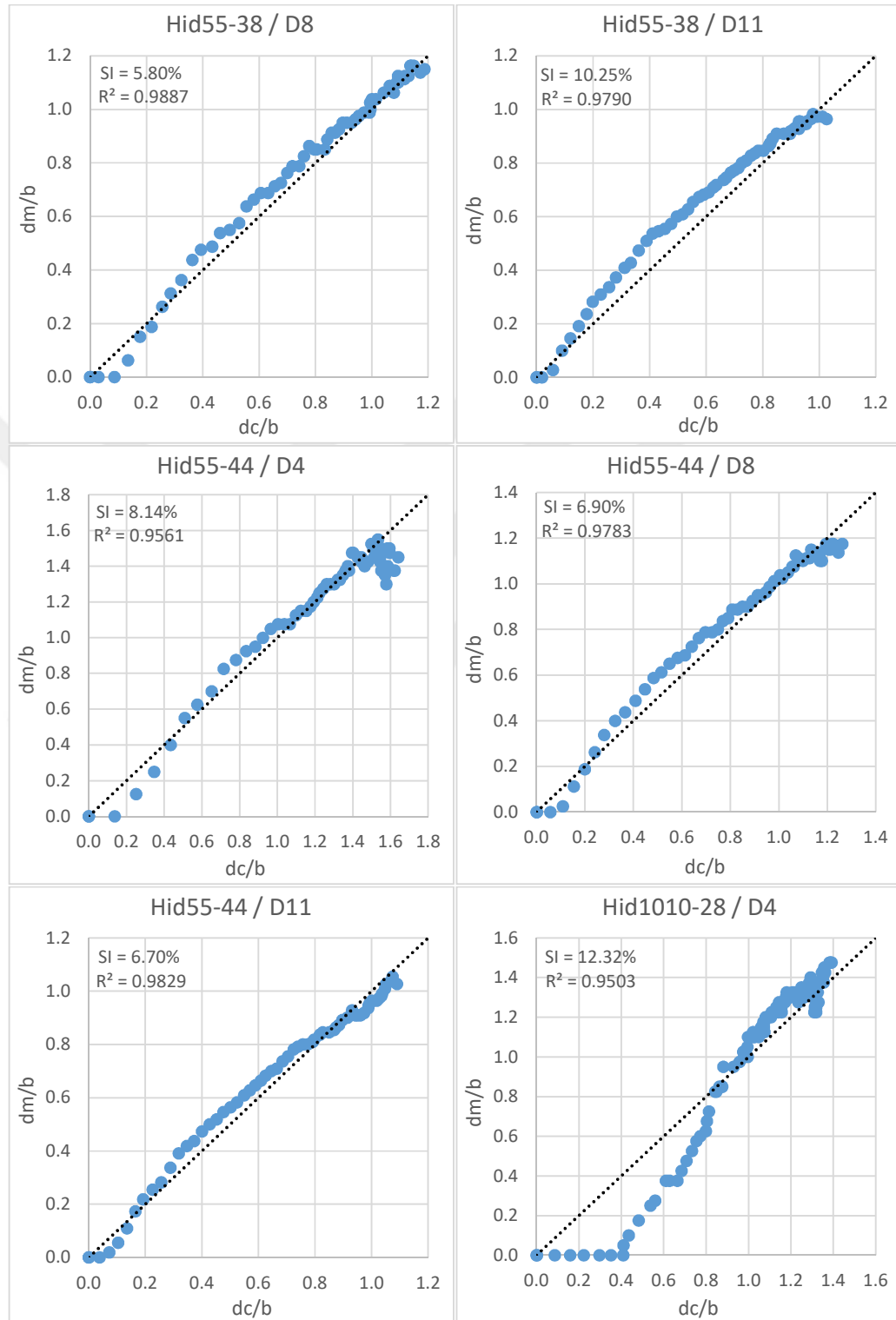


Figure 5.32 continues

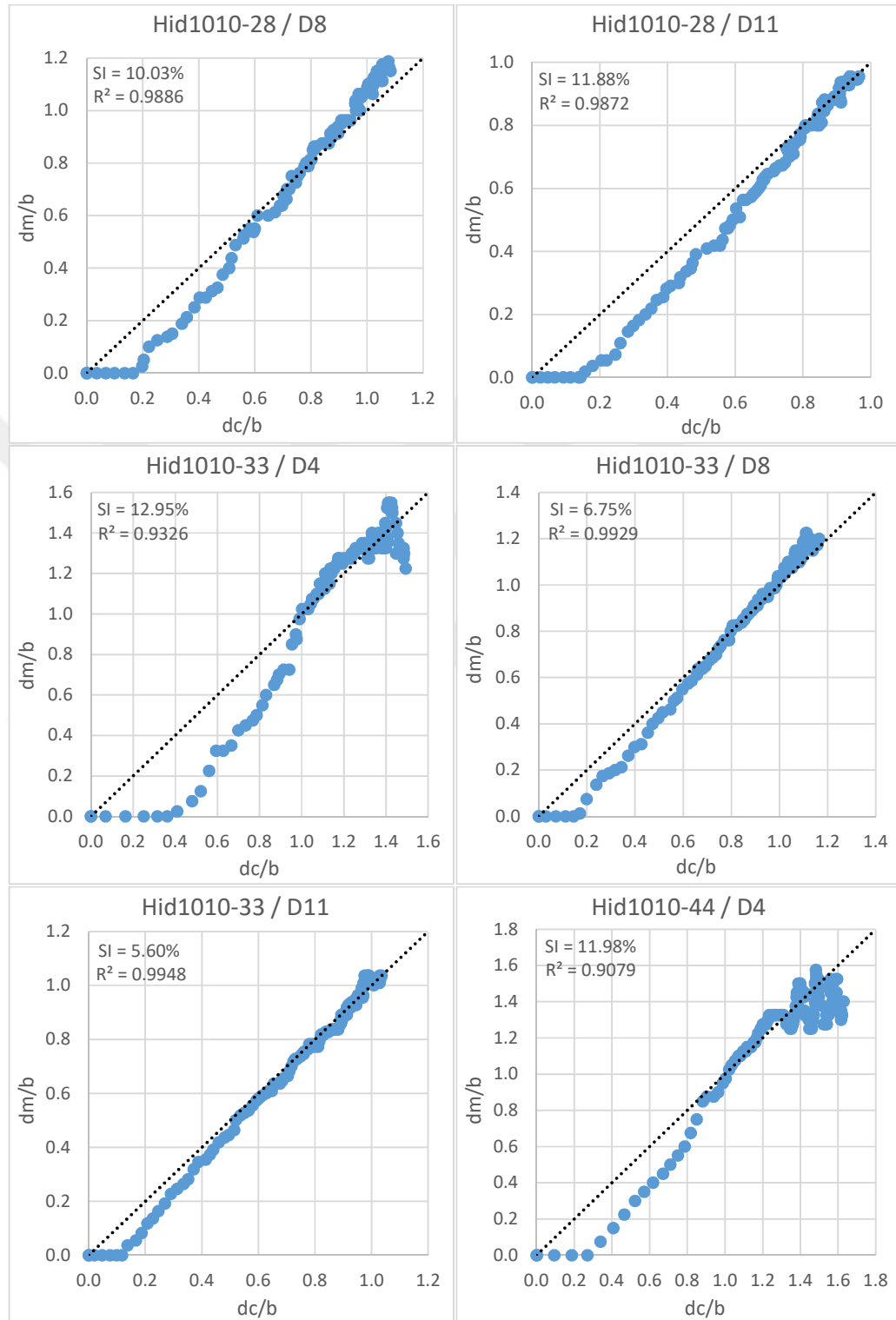


Figure 5.32 continues

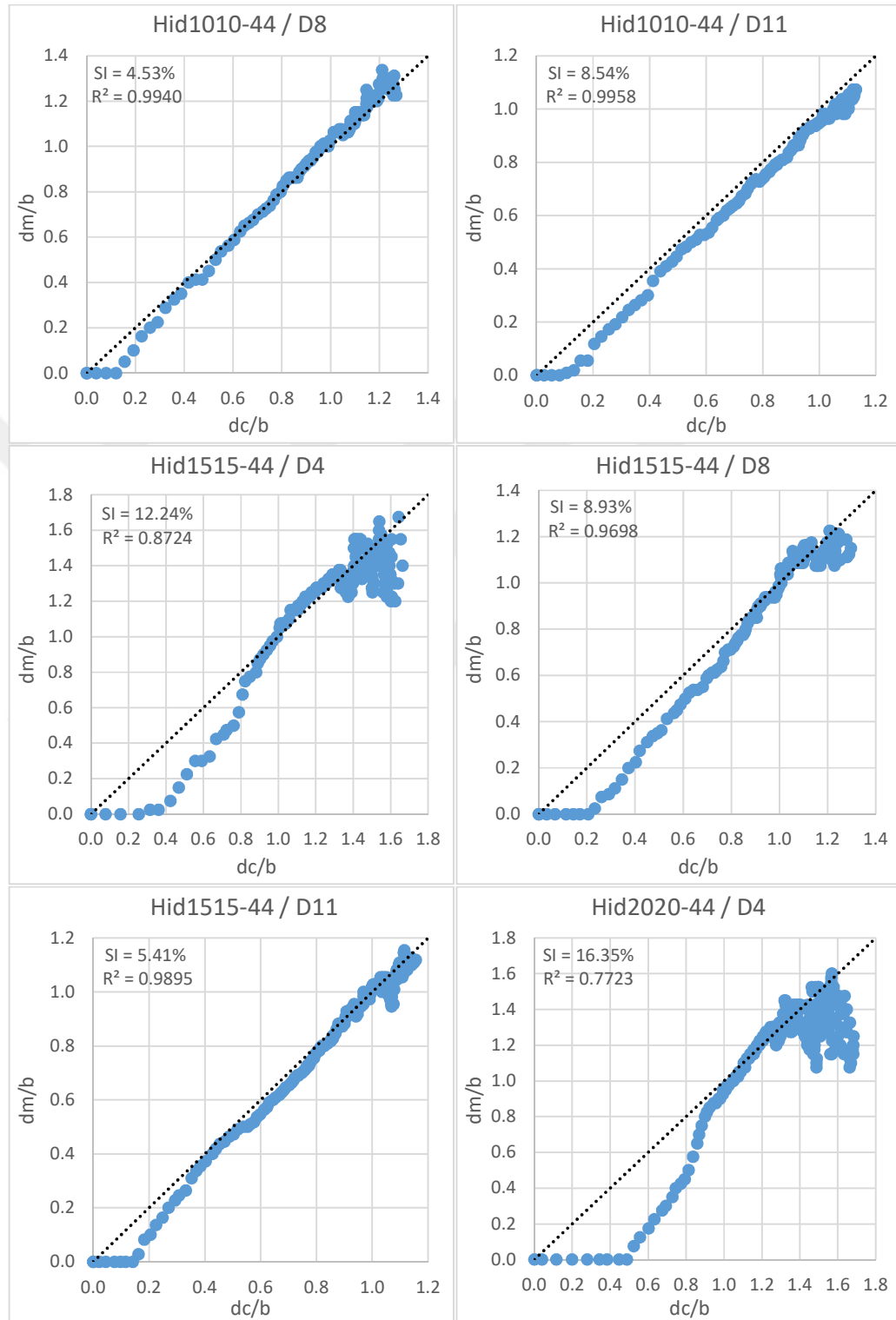


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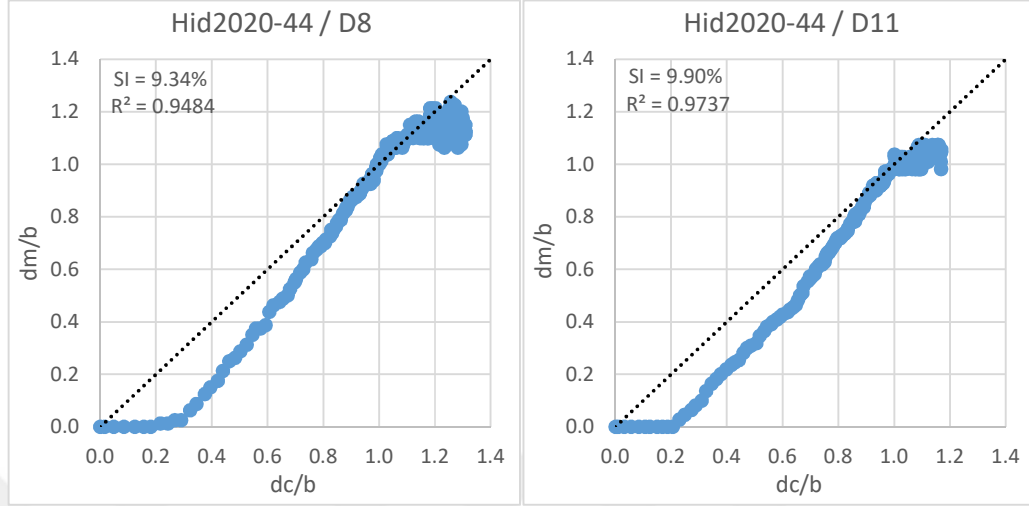


Figure 5.32 continues

5.9.3 Predicting Maximum Scour Depth

A separate equation was developed to predict maximum scour depth during floods. Effective parameters were found as peak values of flow intensity $(V/V_c)_p$, dimensionless flow depth $(y/b)_p$ and dimensionless time scale of maximum scour depth T_s^* by means of various regression analysis.

$$\frac{d_m}{b} = 0.83 \left(\frac{V}{V_c} \right)_{(p)}^{0.23} \left(\frac{y}{b} \right)_{(p)}^{0.4} T_s^{*0.053} \quad (5.8)$$

Measured values were used for dimensionless time scales but it can be calculated from Eq. (5.4). Determination coefficients were found for the piers with diameters of 4 cm, 8 cm and 11 cm as 0.91, 0.80 and 0.87 respectively. Calculated and measured dimensionless scour depths (indicated as dc/b and dm/b , respectively) were given in Figure 5.33 for each pier with their scatter indices.

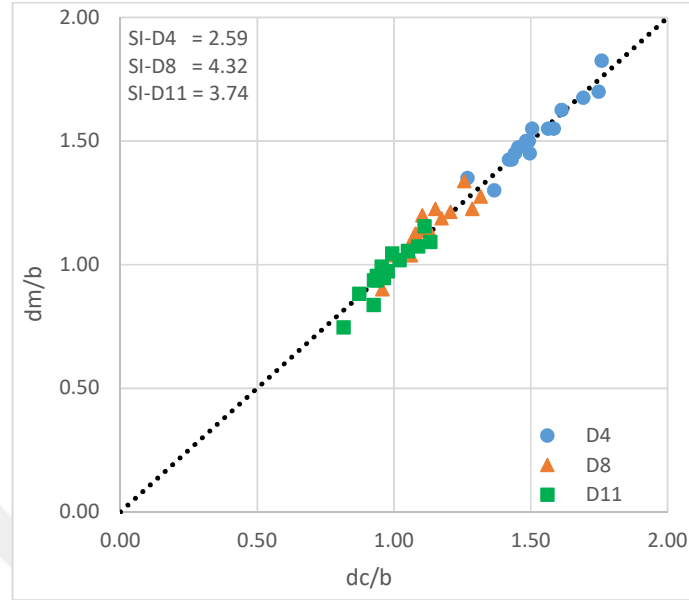


Figure 5.33 Measured and calculated maximum scour depths

5.10 Comparison of Experimental Maximum Scour Depths with Formulas Proposed in the Case of Live Bed

Although the equilibrium scour depth concept is not applicable in live bed scour under unsteady flow conditions, the maximum scour depths were compared with the equilibrium scour depths calculated from the formulas existing in literature. In these formulas, the experimental peak flow parameters were used. The so obtained results are given in Figure 5.34 a, b and c, for the piers D4, D8 and 11, respectively.

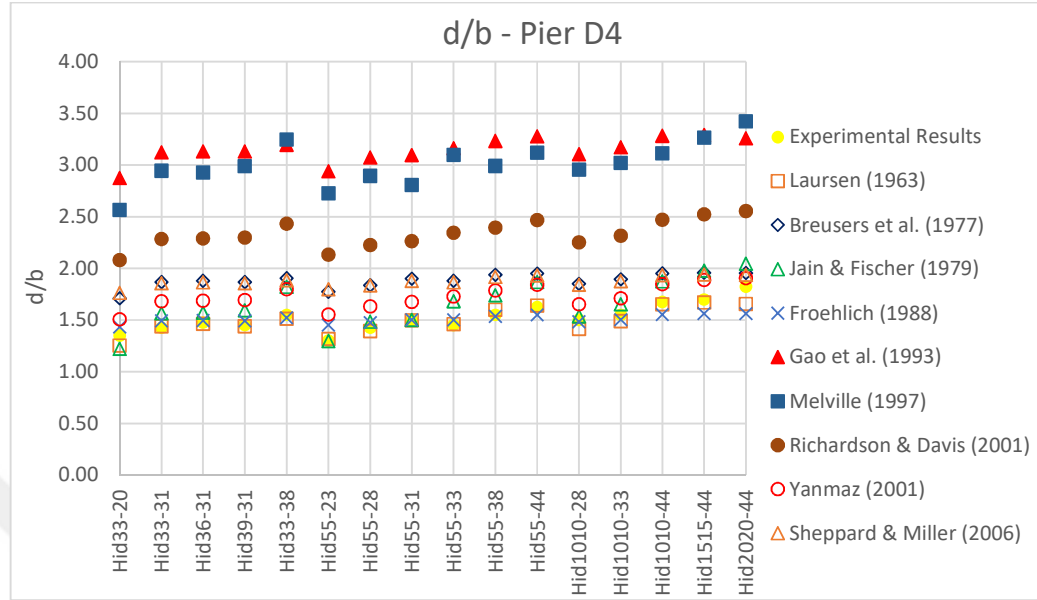


Figure 5.34 a) Equilibrium scour depths calculated by present formulas in literature and maximum scour depths of experimental results for the pier D4

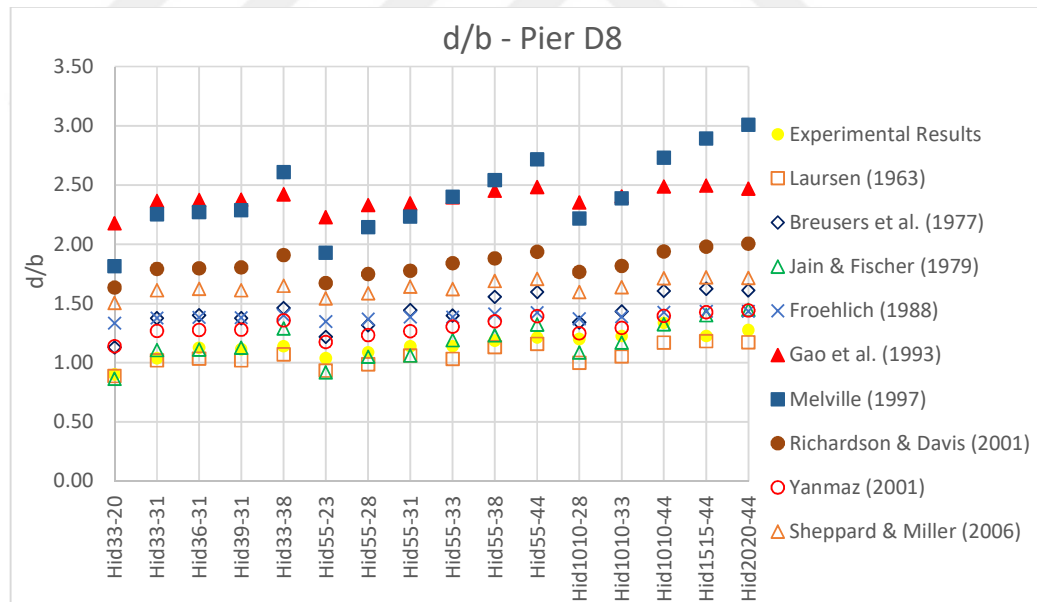


Figure 5.34 b) Equilibrium scour depths calculated by present formulas in literature and maximum scour depths of experimental results for the pier D8

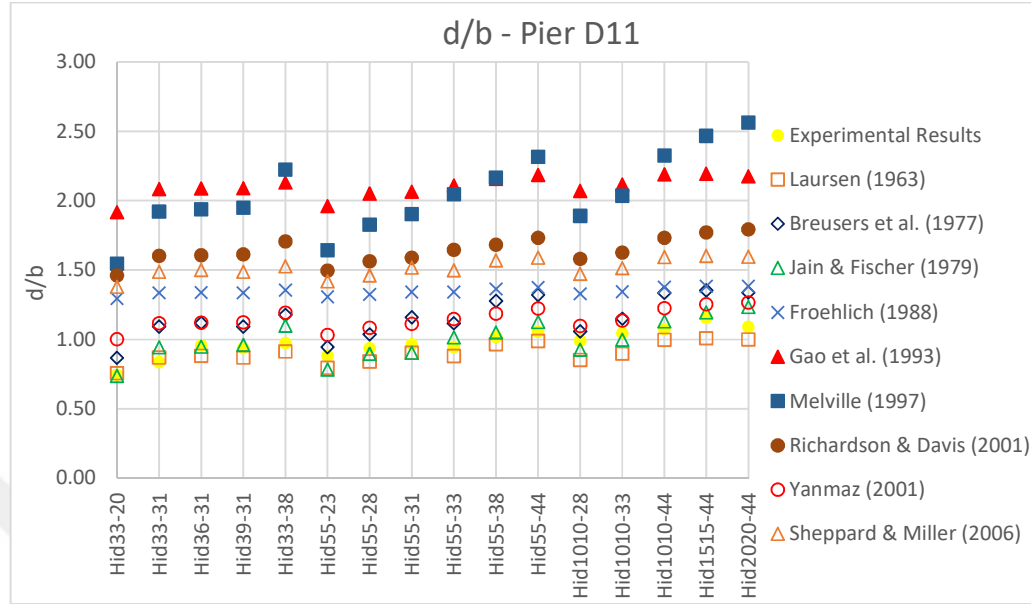


Figure 5.34 c) Equilibrium scour depths calculated by present formulas in literature and maximum scour depths of experimental results for the pier D11

Measured maximum scour depths are closer to the scour depths obtained by the equations of Laursen (1963), Jain & Fischer (1979). Equations of Breusers et al. (1977) and Yanmaz (2001) also gave the results which are not far. Larger values obtained from the most of the above mentioned formulas are expected and reasonable for steady flow conditions, since the scour depths calculated from the formulas corresponding to steady flow conditions may be overestimated (Kothyari et al. 1992a). Hence, it is actually the formulas predicted scour depths closer to experimental results were underestimated for steady flow conditions.

5.11 Prototype Scour Depth Values in the case of physical models

It was attempted to determine the corresponding pier diameters, flow rate, time to reach the peak flow rate, the hydrograph duration and scour depth values, when the piers used in the experiments are considered as 1/10 and 1/20 scaled physical models.

Since the gravitational force is dominant, the experimental findings are converted to the prototype ones by using the Froude similarity law. In the case of different model scale (L_r) the following time scale (T_r) and flow rate scale (Q_r) values were obtained:

For $L_r = 1/10$, $T_r = \sqrt{L_r} = 0.316$, $Q_r = L_r^{5/2} = 0.00316$

For $L_r = 1/20$, $T_r = \sqrt{L_r} = 0.224$, $Q_r = L_r^{5/2} = 0.00056$

The so obtained prototypes values are given in Table 5.4, together with those concerning the physical model. In this table; D_m and D_p indicate pier diameters in model and prototype, Q_{mp} and Q_{pp} denote peak flow rates in model and prototype, t_{mp} and t_{pp} are time to reach the peak flow rate in model and prototype, t_{md} and t_{pd} mean hydrograph duration in model and prototype, d_{ms} and d_{ps} denote maximum scour depth in model and prototype.

Table 5.4 Obtained prototype values together with concerning the physical model

	Model						Prototype (1/10)						Prototype (1/20)					
	Qp (L/s)	tp (min)	te (min)	dm (cm)			Qp (m3/s)	tp (min)	te (min)	dm (m)			Qp (m3/s)	tp (min)	te (min)	dm (m)		
				D4	D8	D11				D=40 cm	D=80 cm	D=110 cm				D=80 cm	D=160 cm	D=220 cm
Hid33-20	20	3	6	5.4	7.2	8.2	6.32	9.49	18.97	0.54	0.72	0.82	35.78	13.42	26.83	1.08	1.44	1.64
Hid33-31	31	3	6	5.7	8.3	9.2	9.80	9.49	18.97	0.57	0.83	0.92	55.45	13.42	26.83	1.14	1.66	1.84
Hid36-31	31	3	9	5.9	9	10.5	9.80	9.49	28.46	0.59	0.90	1.05	55.45	13.42	40.25	1.18	1.80	2.10
Hid39-31	31	3	12	5.8	8.9	10.3	9.80	9.49	37.95	0.58	0.89	1.03	55.45	13.42	53.67	1.16	1.78	2.06
Hid33-38	38	3	6	6.2	9.1	10.7	12.02	9.49	18.97	0.62	0.91	1.07	67.98	13.42	26.83	1.24	1.82	2.14
Hid55-23	23	5	10	5.2	8.3	9.7	7.27	15.81	31.62	0.52	0.83	0.97	41.14	22.36	44.72	1.04	1.66	1.94
Hid55-28	28	5	10	5.7	8.7	10.3	8.85	15.81	31.62	0.57	0.87	1.03	50.09	22.36	44.72	1.14	1.74	2.06
Hid55-31	31	5	10	6	9.1	10.6	9.80	15.81	31.62	0.60	0.91	1.06	55.45	22.36	44.72	1.20	1.82	2.12
Hid55-33	33	5	10	5.8	9.1	10.4	10.44	15.81	31.62	0.58	0.91	1.04	59.03	22.36	44.72	1.16	1.82	2.08
Hid55-38	38	5	10	6.2	9.5	11.2	12.02	15.81	31.62	0.62	0.95	1.12	67.98	22.36	44.72	1.24	1.90	2.24
Hid55-44	44	5	10	6.5	9.7	11.6	13.91	15.81	31.62	0.65	0.97	1.16	78.71	22.36	44.72	1.30	1.94	2.32
Hid1010-28	28	10	20	6	9.6	10.9	8.85	31.62	63.25	0.60	0.96	1.09	50.09	44.72	89.44	1.20	1.92	2.18
Hid1010-33	33	10	20	6.2	9.8	11.5	10.44	31.62	63.25	0.62	0.98	1.15	59.03	44.72	89.44	1.24	1.96	2.30
Hid1010-44	44	10	20	6.7	10.7	11.8	13.91	31.62	63.25	0.67	1.07	1.18	78.71	44.72	89.44	1.34	2.14	2.36
Hid1515-44	44	15	30	6.8	9.8	12.7	13.91	47.43	94.87	0.68	0.98	1.27	78.71	67.08	134.16	1.36	1.96	2.54
Hid2020-44	44	20	40	7.3	10.2	12	13.91	63.25	126.49	0.73	1.02	1.20	78.71	89.44	178.89	1.46	2.04	2.40

CHAPTER SIX

NUMERICAL RESULTS AND COMPARISON WITH EXPERIMENTAL FINDINGS

Hid33-20 hydrograph was selected to perform the numerical analysis for the pier D4. LES, RNG and two equation $k-\epsilon$ turbulence models were tested separately and their effects on scour depth and shape of the scour hole were examined.

6.1 Example of Numerical Scour Depths, Average Velocities and Flow Depths

Raw output of temporal scour depths obtained from Flow-3D for LES, RNG and $k-\epsilon$ turbulence models are presented in Figure 6.1 a, b and c at T1, in Figure 6.2 a, b and c at T2, respectively. The units of all parameters are given in CGS unit system.

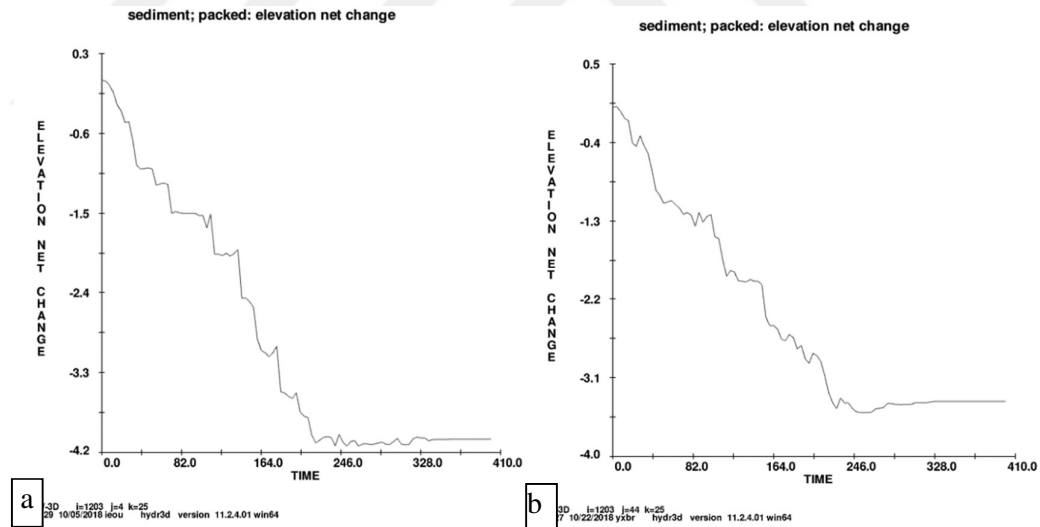


Figure 6.1 a, b and c) Raw output of temporal scour depth at T1 for LES, RNG and $k-\epsilon$ turbulence models, respectively

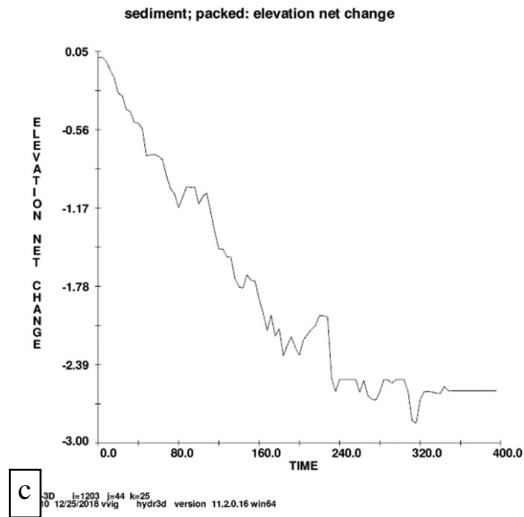


Figure 6.1 continues

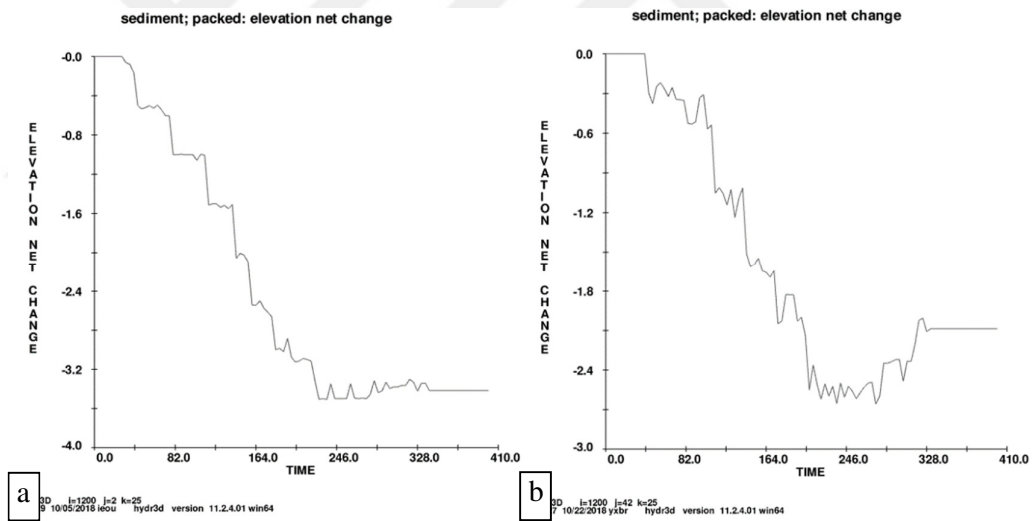


Figure 6.2 a, b and c) Raw output of temporal scour depth at T2 for LES, RNG and k- ϵ turbulence models, respectively

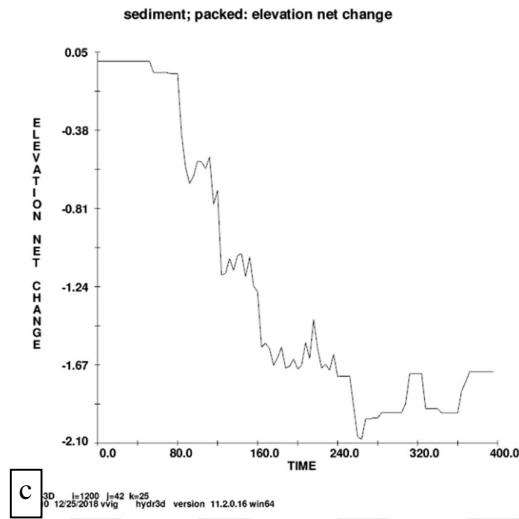


Figure 6.2 continues

At 11th meter of the flume, depth averaged velocities of each turbulence model are presented in Figure 6.3 a, b c and d, together with experimental ones, while Figure 6.4 a, b, c and d contains the numerical flow depths together with the experimental ones.

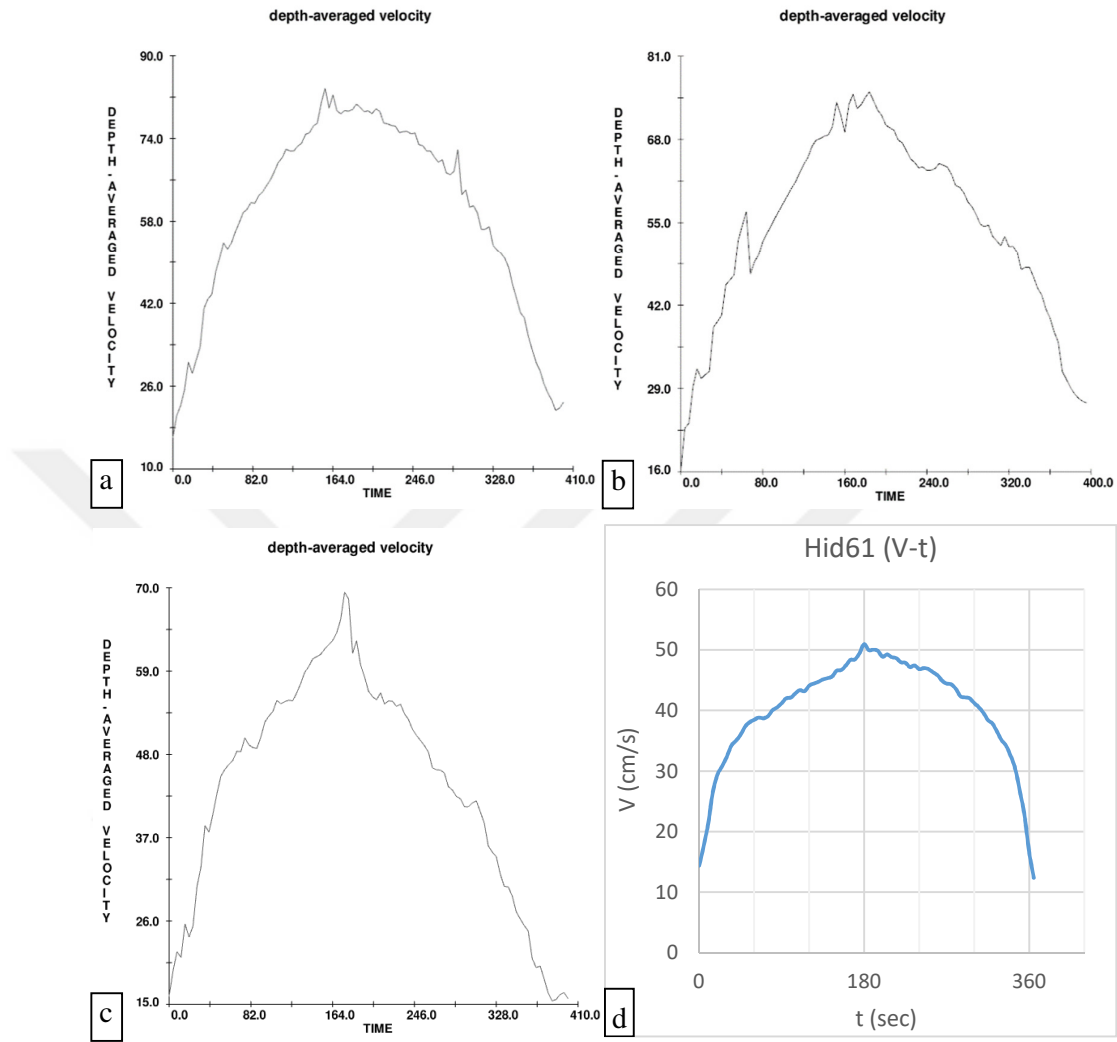


Figure 6.3 Depth average velocities of a) LES turbulence model, b) RNG turbulence model, c) k- ϵ turbulence model, d) experimental results

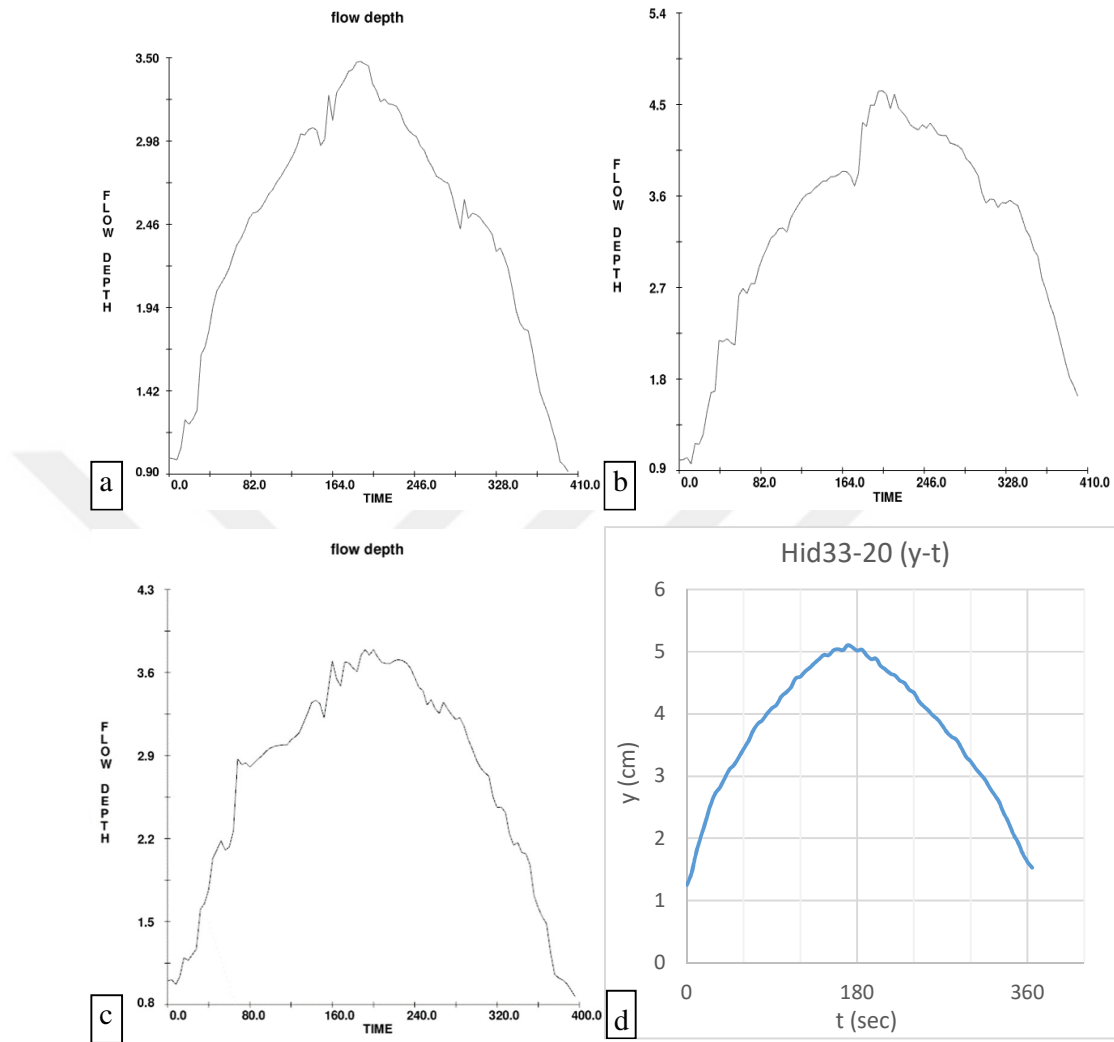


Figure 6.4 Flow depths of a) LES turbulence model, b) RNG turbulence model, c) k-ε turbulence model, d) experimental results

6.2 Example of Numerical Velocity Fields

The velocity fields in the wake region, in the surrounding of the pier and along the longitudinal section, at the peak time, are given in Figure 6.5 a, b and c, respectively, as illustrative examples.

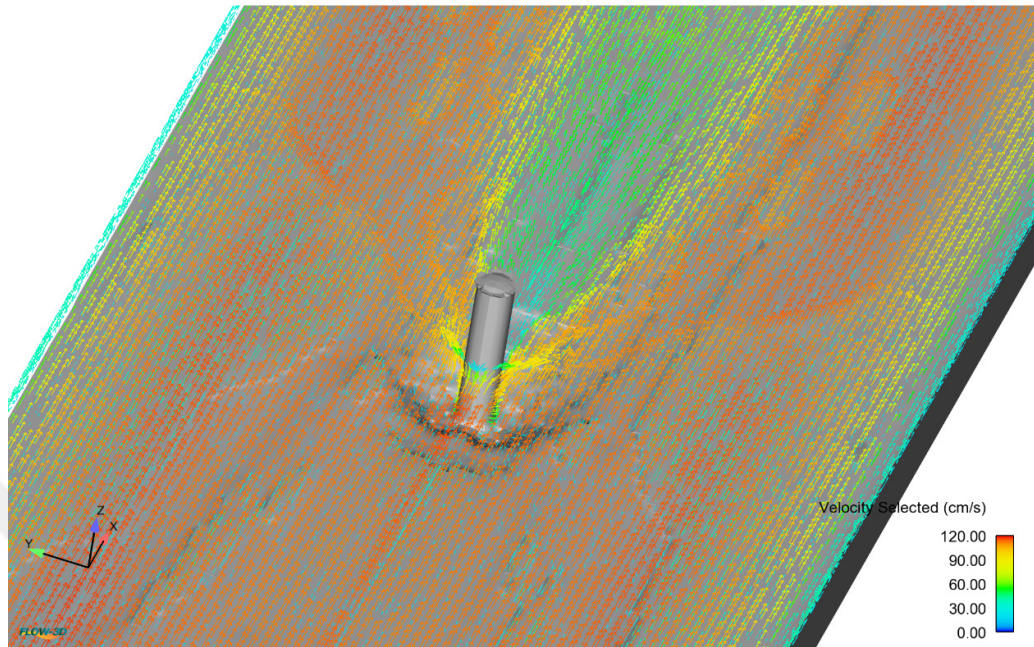


Figure 6.5 a) The velocity field in the wake region at peak time

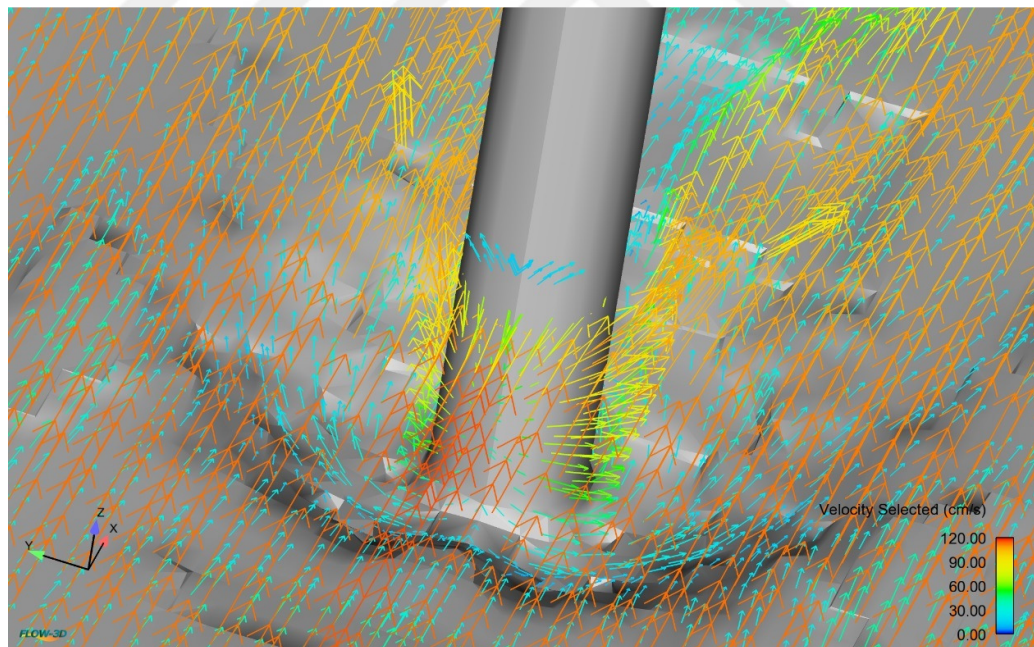


Figure 6.5 b) The velocity field in the surrounding of the pier at peak time

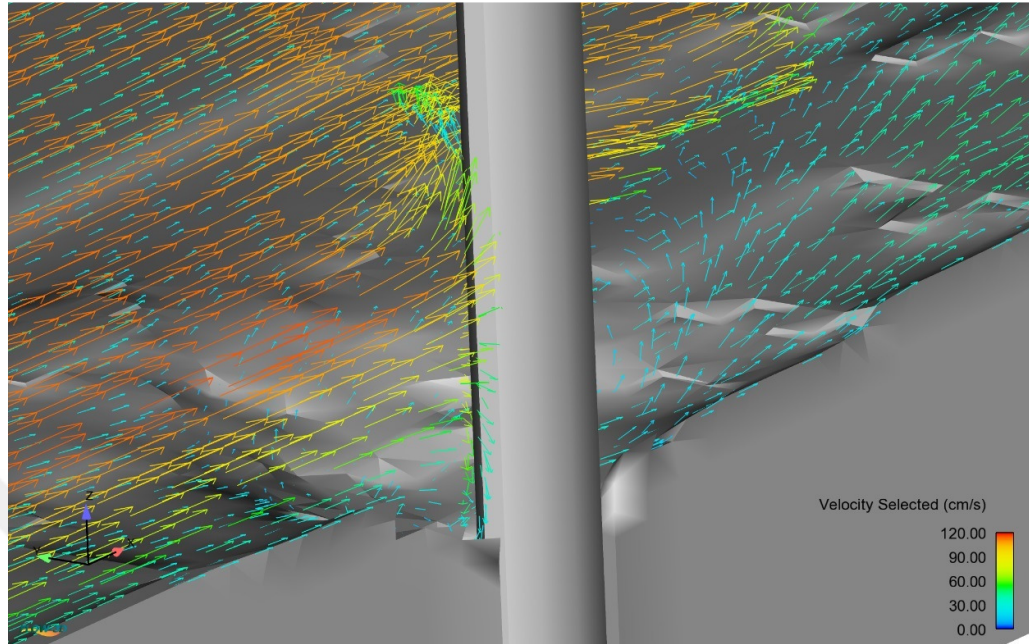


Figure 6.5 c) The velocity field along the longitudinal section at peak time

6.3 Comparison of Scour Hole Shapes

At the end of the simulations, 3D pictures of scour hole were taken for each turbulence model at every 60 secs. Figure 6.6, 6.7 and 6.8 a to 6.6, 6.7 and 6.8 f show this process for LES, RNG and two equation $k-\varepsilon$ turbulence models respectively. Figure 6.6 f, Figure 6.7 f and 6.8 f correspond to the final state of the scouring. Also a picture of scour hole at the end of the same experiment is given in Figure 6.9 for comparison.

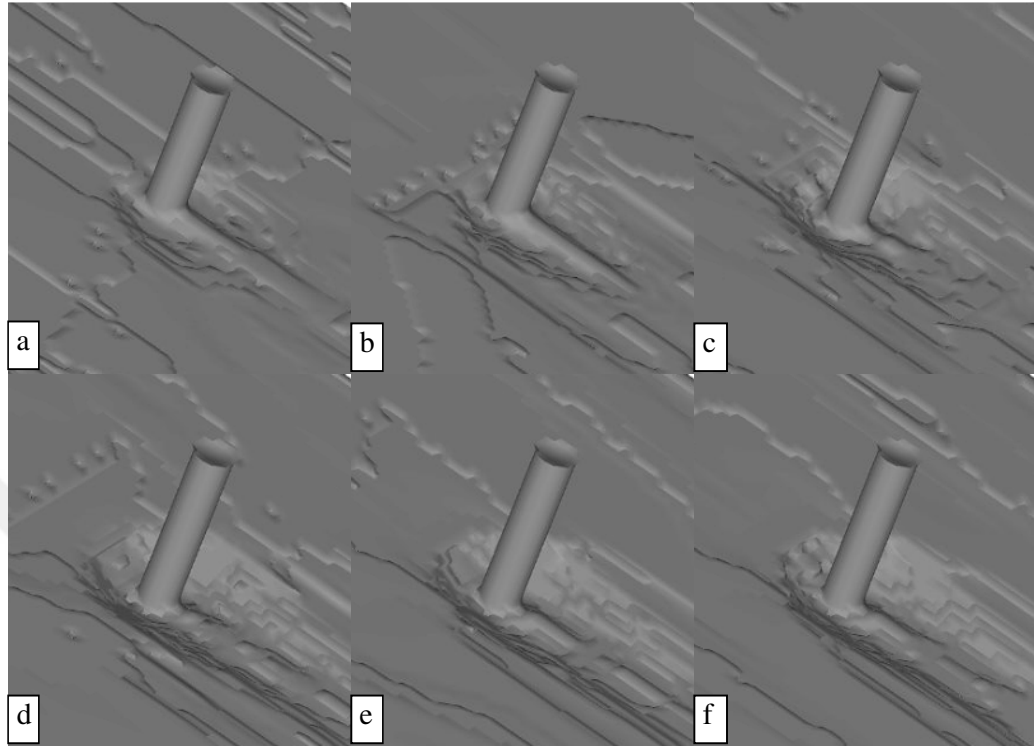


Figure 6.6 a) to 6.1 f) Pictures of scour hole at every 60 secs for LES turbulence model

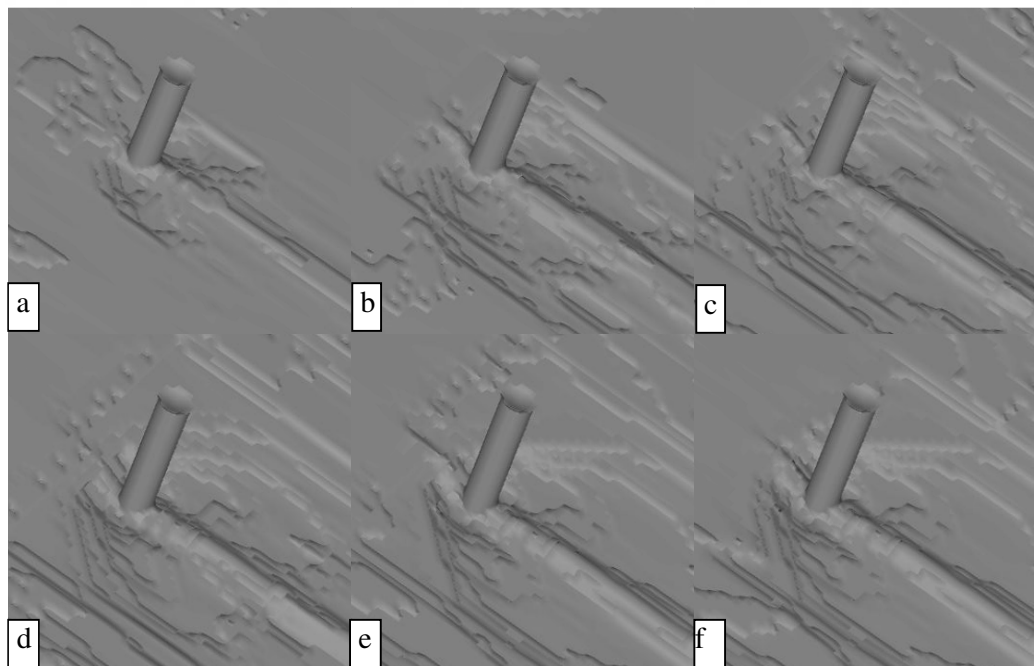


Figure 6.7 a) to 6.2 f) Pictures of scour hole at every 60 secs for RNG turbulence model

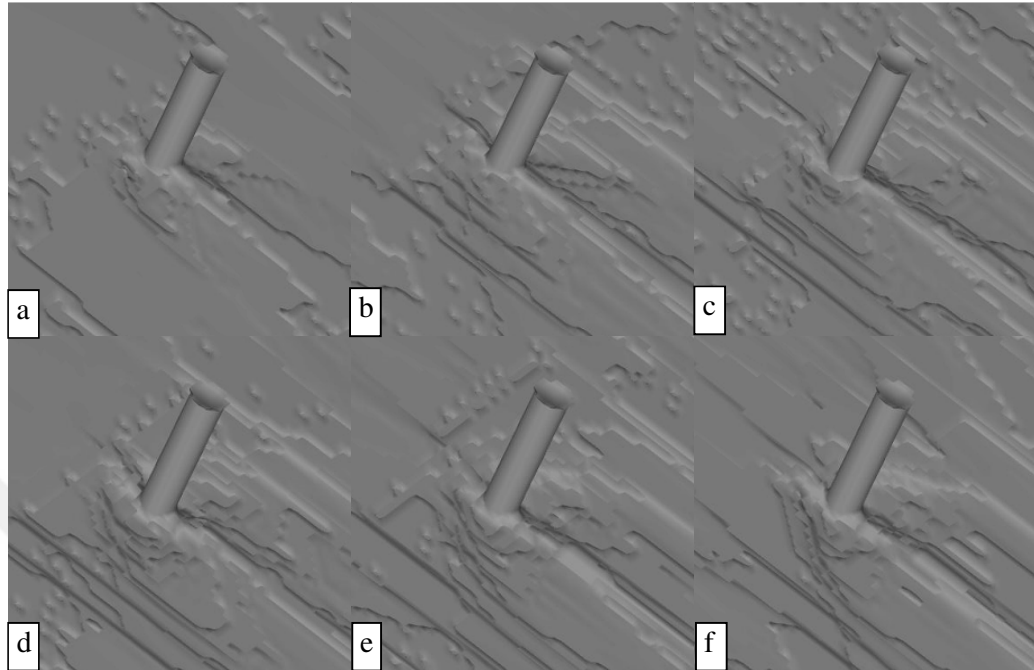


Figure 6.8 a) to 6.3 f) Pictures of scour hole at every 60 secs for k- ϵ turbulence model

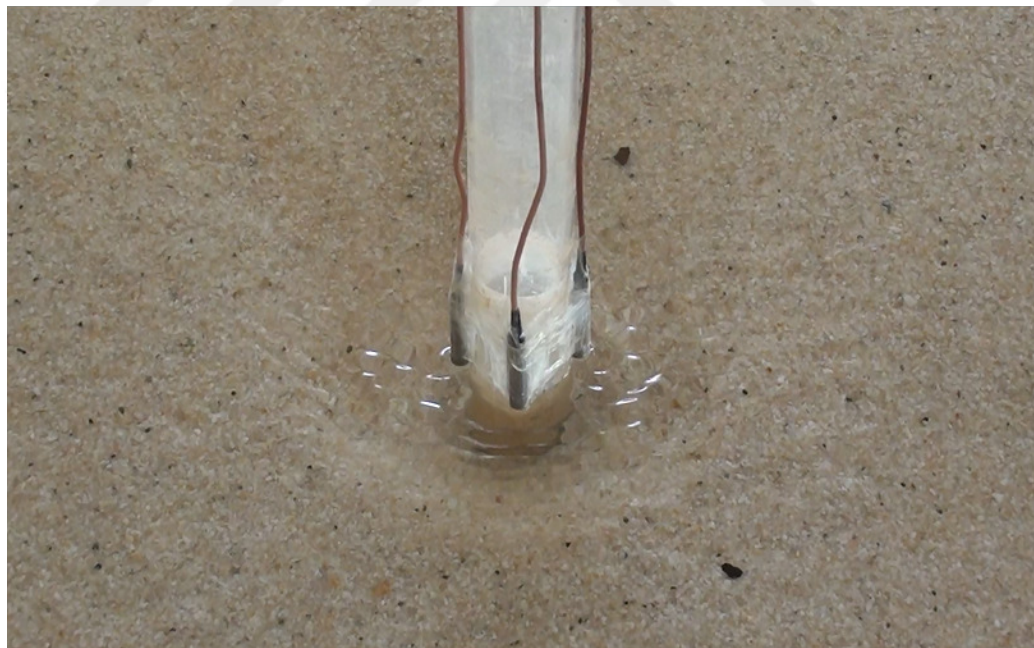


Figure 6.9 Scour hole of Hid33-20 hydrograph at the end of the experiment (Personal archive, 2017)

LES turbulence model gave the most similar shape of the scour hole compared to experiment. A “linear bump” occurred in front of the pier in all turbulence models but, this fact was not observed in experiments. The frustum shaped of the scour hole in Figure 6.9 was not be simulated in the used turbulence models, except in LES

turbulence model where this configuration occurred between 180-240 seconds. This situation caused lower scour depths in front of the pier compared to scour depths at flanks which is contradictory to the experimental results.

Figure 6.10 a, b and c show 2D view of the scour hole at 240th secs of the simulation for LES, RNG and two equation k- ϵ turbulence models, respectively. LES turbulence model simulated scouring in front of the pier much better than the other turbulence models even if it gave lower scour depths compared to those at flanks. RNG turbulence model simulated scouring in wake region better than LES model.

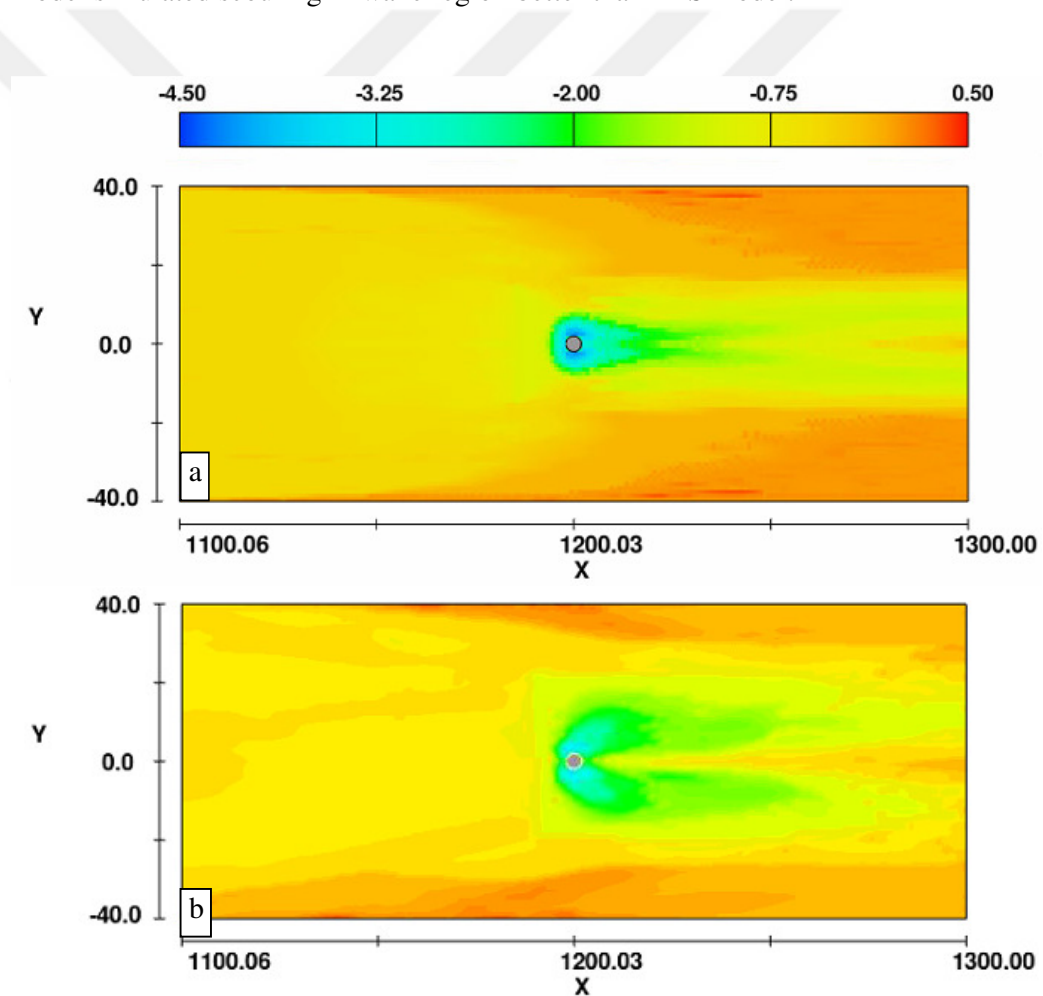


Figure 6.10 2D (top) views of scour holes for a) LES, b) RNG and c) two equation k- ϵ turbulence models respectively

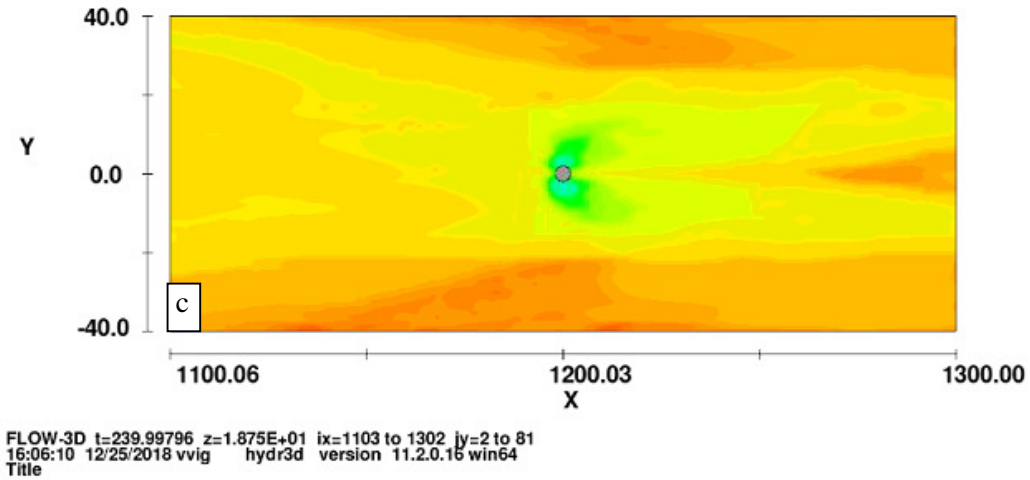


Figure 6.10 continues

6.4 Comparison of Numerical and Experimental Scour Depths

The numerical and experimental time dependent scour depths at flank (T1) and front (T2) are given in Figure 6.11.

Scouring rates were found smaller in simulations. For example, for T1, scour depth of 30 mm was reached in experiment at 30 secs. This value was reached at 180 seconds and 210 seconds in LES and RNG turbulence models, respectively.

Scour depths at front was found smaller compared to those calculated at flank. This fact is also inconsistent with experimental results.

RNG turbulence model seems to reflect better the sediment transport, since final scour depth was calculated smaller than maximum scour depth. On the other hand, The scour depths calculated by using LES turbulence model are closer to those obtained from the experiments.

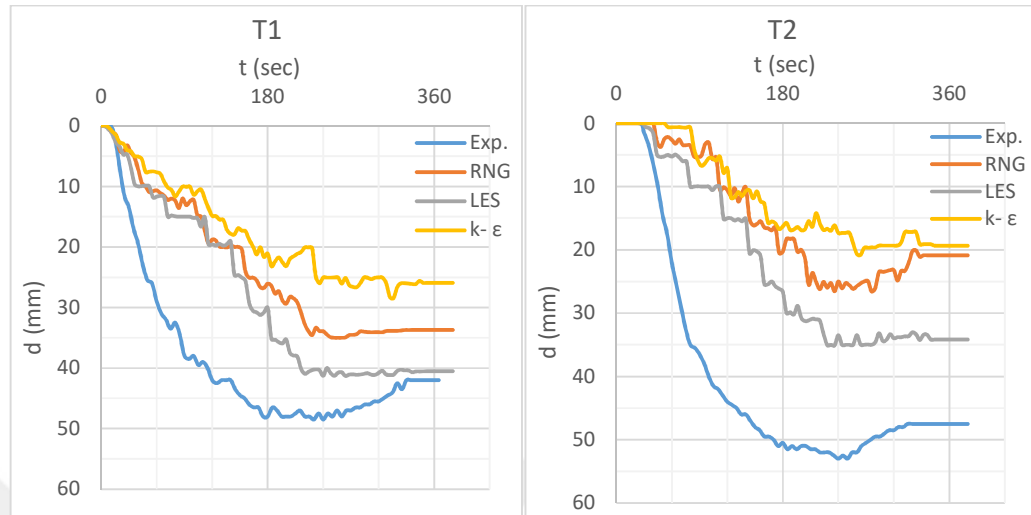


Figure 6.11 Scour depths of each turbulence model and experimental result

CHAPTER SEVEN

CONCLUSIONS AND SUGGESTIONS

16 different triangular hydrographs were generated with total durations ranging from 6 mins to 40 mins and peak discharges ranging from 20 L/s to 44 L/s. Circular piers with diameters of 4 cm, 8 cm and 11 cm were tested under these hydrographs. Scour depths were measured at three different points around the pier by means of UVP. Flow depth was measured at 1 m upstream of the pier at by means of ULS. The amount of transported sediment was measured and sediment graphs were formed for each hydrograph. Flume was fed manually according to related sediment graph.

During the experiments, it was observed that scour started initially at the flanks of the pier as two small holes. Later, these small holes met in front of the pier and scour hole began to develop with increasing discharge. Initially, scour depth at flanks was higher than scour depth at front. Scour depth at front became higher some time later according to acceleration rate of the tested hydrograph and maximum scour depth was measured in front of the pier at the end of the experiments.

During the rising limb, flow rate was dominant element of the scour. Scour depth began to increase quickly and sediment movement insignificantly affected scour depth and fluctuation heights. During the falling limb, sediment transport rate became dominant element with decreasing flow rate. Fluctuation heights became greater and supplied sediment into the scour hole provided decrease in scour depth. Thus, a lower final scour depth occurred compared to maximum scour depth.

Hydrographs with the same peak discharges but different durations were classified separately to investigate the effect of the duration of rising and falling limbs. Generally, the following facts were observed:

- Maximum scour depths increased with increase of the duration of rising limb.
- Longer duration of floods caused higher fluctuations.
- As the duration of falling limb became longer, amount of sediment supplied into the scour hole increased, so a lower final scour depth was observed.

- Difference between maximum and final scour depth ($d_m - d_f$) increased with the increase of the duration of falling limb.

Hydrographs with same durations but different peak discharges were classified separately to investigate the effect of the peak discharge. The following observations were noted:

- Scouring developed more rapidly with high velocities and higher velocities caused greater maximum scour depths.
- Magnitudes of scour depth fluctuations increased with increase in flow velocity.
- Greater velocities caused greater rate of sediment transport, so difference between maximum scour depth and final scour depth increased with increasing flow velocity. Thus, high peak discharges provided lower final scour depths at piers with relatively small diameters.

Pier diameter directly affected the influence of supplied sediment on scour depth. As known, size of scour hole increases when the pier width increases. Although the amount of transported sediment was the same for a given hydrograph, it was the size of scour hole which determined the effect of supplied sediment on scour depth. Scour holes with smaller sizes were affected from sediment transport more. So highest fluctuations were observed with the smallest pier that has a diameter of 4 cm. Also ratio between maximum and final scour depth increased with decrease in pier diameter.

It was found that when $y/b < 0.72$, scour depths at flanks of the all piers shared a common line. Scour depth curve of the pier with 4 cm diameter left the common line after $y/b = 0.97$ and scour depth curve of the pier with 8 cm diameter left the common line after $y/b = 0.61$. Further studies with piers with different diameters and flow depths will be useful to clarify this fact.

Scour hole took shape of a frustum with the development of horseshoe vortices. The final longitudinal profile of the scour hole was determined by means of a laser meter. Slope of the upstream side of the scour hole was calculated approximately as

0.76, corresponding to an angle of 37° . A mound formation was observed at downstream side of the pier with a crest 3-4 diameters away from the pier.

Three empirical equations were derived in order to predict temporal scour depth, time scale of maximum scour depth and maximum scour depth by means of the least squares method (RMSE). Significant correlations were observed between predicted and measured values. Flow intensity, relative flow depth and dimensionless time were found to be dominant parameters to translate temporal scour depth under unsteady flow-live bed conditions.

Experiments were carried out under flows with relatively low velocities and short duration of hydrographs because of limited technical opportunities. Instead of manual feeding, a sediment pump mounted into the circulation system would provide longer duration of floods and higher flow velocities with lower workforce. The effect of different sized uniform material and non-uniform materials on the scouring in the case of unsteady flow and in live bed conditions worth to be investigated.

In numerical analyses, LES turbulence model gave the most similar shape of the scour hole compared to experiment. LES turbulence model simulated scouring in front of the pier much better. On the other hand RNG turbulence model simulated scouring in wake region better than LES model. Scouring rates were found smaller in simulations. Scour depths at front was found smaller compared to those calculated at flank.

RNG turbulence model seems to reflect better the sediment transport, since final scour depth was calculated smaller than maximum scour depth. On the other hand, The scour depths calculated by using LES turbulence model are closer to those obtained from the experiments.

REFERENCES

- Breusers, H. N. C., Nicollet, G., & Shen, H. W. (1977). Local scour around cylindrical piers. *Journal of Hydraulic Research*, 15(3), 211-252.
- Chabert, J., & Engeldinger, P. (1956). Study of scour around bridge piers. *Report Prepared for the Laboratoire National d'Hydraulique*.
- Chang, W. Y., Lai, J. S., & Yen, C. L. (2004). Evolution of scour depth at circular bridge piers. *Journal of Hydraulic Engineering*, 130(9), 905-913.
- Chee, R. K. W. (1982). Live-bed scour at bridge piers. *Publication of: Auckland University, New Zealand*, (290).
- Chiew, Y. M. (1984). *Local Scour at Bridge Piers*. Ph.D. Thesis, The University of Auckland, Auckland.
- Dietz, J. W. (1972). Systematische Modellversuche über die Pfeilerkolkbildung. *Das Tideregime der Elbe-Hydraulisches Modell mit beweglicher Sohle*, (31), 95-110.
- Dogan, M., & Arisoy, Y. (2014). Time development of local scour depth below pipelines exposed to waves. *Coastal Engineering Proceedings*, 1(34), 9.
- Ettema, R. (1980). *Scour at bridge piers*. Ph.D. Thesis, The University of Auckland, Auckland.
- Froehlich, D. C. (1988). Analysis of onsite measurements of scour at piers. In *Hydraulic engineering: proceedings of the 1988 national conference on hydraulic engineering* (534-539).
- Gao, D., Posada, G., & Nordin, C. F. (1993). Pier scour equations used in China. *Hydraulic Engineering* (1031-1036).
- Guney, M. S., Bombar, G., Aksoy, A. O., & Dogan, M. (2013). Use of UVP to investigate the evolution of bed configuration. *KSCE Journal of Civil Engineering*, 17(5), 1188-1197.
- Hager, W. H., & Unger, J. (2010). Bridge pier scour under flood waves. *Journal of Hydraulic Engineering*, 136(10), 842-847.

- Hancu, S. (1971). Sur le calcul des affouillements locaux dans la zone des piles des ponts. In *Proceedings of the 14th IAHR Congress, Paris, France* 3(1), 299-313.
- Jain, S. C., & Fischer, E. E. (1979). *Scour around circular bridge piers at high Froude numbers* (No. FHWA-RD-79-104 Final Report).
- Jain, S. C., & Fischer, E. E. (1980). Scour around bridge piers at high flow velocities. *Journal of the Hydraulics Division*, 106(11), 1827-1842.
- Jia, Y., Altinakar, M., & Guney, M. S. (2017). Three-dimensional numerical simulations of local scouring around bridge piers. *Journal of Hydraulic Research*, 1-16.
- Kothyari, U. C., Ranga Raju, K. G., & Garde, R. J. (1992a). Live-bed scour around cylindrical bridge piers. *Journal of Hydraulic Research*, 30(5), 701-715.
- Kothyari, U. C., Garde, R. C. J., & Ranga Raju, K. G. (1992b). Temporal variation of scour around circular bridge piers. *Journal of Hydraulic Engineering*, 118(8), 1091-1106.
- Kothyari, U. C., Hager, W. H., & Oliveto, G. (2007). Generalized approach for clear-water scour at bridge foundation elements. *Journal of hydraulic Engineering*, 133(11), 1229-1240.
- Lai, J. S., Chang, W. Y., & Yen, C. L. (2009). Maximum local scour depth at bridge piers under unsteady flow. *Journal of Hydraulic Engineering*, 135(7), 609-614.
- Laursen, E. M. (1963). An analysis of relief bridge scour. *Journal of the Hydraulics Division*, 89(3), 93-118.
- Laursen, E. M., & Toch, A. (1956). *Scour around bridge piers and abutments* (Volume 4). Ames, IA: Iowa Highway Research Board.
- Liu, Z. (2001). *Sediment transport* (3rd ed.). Aalborg: Aalborg Universitet.
- Mastbergen, D. R., & Van Den Berg, J. H. (2003). Breaching in fine sands and the generation of sustained turbidity currents in submarine canyons. *Sedimentology*, 50(4), 625-637.
- Melville, B. W. (1975). *Local scour at bridge sites*. Ph.D. Thesis, The University of Auckland, Auckland.

- Melville, B. W. (1984). Live-bed scour at bridge piers. *Journal of Hydraulic Engineering*, 110(9), 1234-1247.
- Melville, B. W. (1997). Pier and abutment scour: integrated approach. *Journal of Hydraulic Engineering*, 123(2), 125-136.
- Melville, B. W., & Chiew, Y. M. (1999). Time scale for local scour at bridge piers. *Journal of Hydraulic Engineering*, 125(1), 59-65.
- Meyer-Peter, E., & Müller, R. (1948). Formulas for bed-load transport. *IAHSR 2nd meeting, Stockholm, appendix 2*. IAHR.
- Mia, M. F., & Nago, H. (2003). Design method of time-dependent local scour at circular bridge pier. *Journal of Hydraulic Engineering*, 129(6), 420-427.
- Nielsen, P. (1992). *Coastal bottom boundary layers and sediment transport 4*. World Scientific Publishing Company.
- Oliveto, G., & Hager, W. H. (2002). Temporal evolution of clear-water pier and abutment scour. *Journal of Hydraulic Engineering*, 128(9), 811-820.
- Raudkivi, A. J. (1986). Functional trends of scour at bridge piers. *Journal of Hydraulic Engineering*, 112(1), 1-13.
- Richardson, E.V. & Davis, S.R. (2001). *Evaluating scour at bridges* (4th Edition) Federal Highway Administration Hydraulic Engineering Circular No. 18, FHWA NHI 01-001.
- Rijn, L. C. V. (1984). Sediment transport, part I: bed load transport. *Journal of Hydraulic Engineering*, 110(10), 1431-1456.
- Rouse, H. (1959). *Advanced Hydromechanics of Fluids*. New York: Wiley.
- Shen, H. W., Schneider, V. R., & Karaki, S. S. (1966). Mechanics of Local Scour. US Department of Commerce, National Bureau of Standards. *Institute for Applied Technology*.
- Shen, H. W., Schneider, V. R., & Karaki, S. S. (1969). Local scour around bridge piers. *Journal of the Hydraulics Division*, 95(6), 1919-1940.
- Sheppard, D. M. (2003). *Large scale and live bed local pier scour experiments. Phase 2: live bed experiments* (No. Final Report).

- Sheppard, D. M., & Miller Jr, W. (2006). Live-bed local pier scour experiments. *Journal of Hydraulic Engineering*, 132(7), 635-642.
- Sheppard, D. M., Odeh, M., & Glasser, T. (2004). Large scale clear-water local pier scour experiments. *Journal of Hydraulic Engineering*, 130(10), 957-963.
- Soulsby, R. (1997). *Dynamics of marine sands: a manual for practical applications*. Thomas Telford.
- Wardhana K., & Hadipriono F. C. (2003). Analysis of Recent Bridge Failures in the United States. *Journal Of Performance Of Constructed Facilities*, 17(3), 144–150.
- Yanmaz, A. M., & Altinbilek, H. D. (1991). Study of time-dependent local scour around bridge piers. *Journal of Hydraulic Engineering*, 117(10), 1247-1268.
- Yanmaz, A.M. (2001). Uncertainty of local scouring parameters around bridge piers. *Turkish Journal of Engineering and Environmental Sciences*, 25(2), 127-137.
- Yanmaz, A.M. (2002). *Köprü Hidroliği*. Ankara: METU Press

APPENDICES

APPENDIX 1

TIME DEPENDENT SCOUR DEPTHS

Time dependent scour depth according to transducers were given in Figure A.1 for each hydrograph and pier.

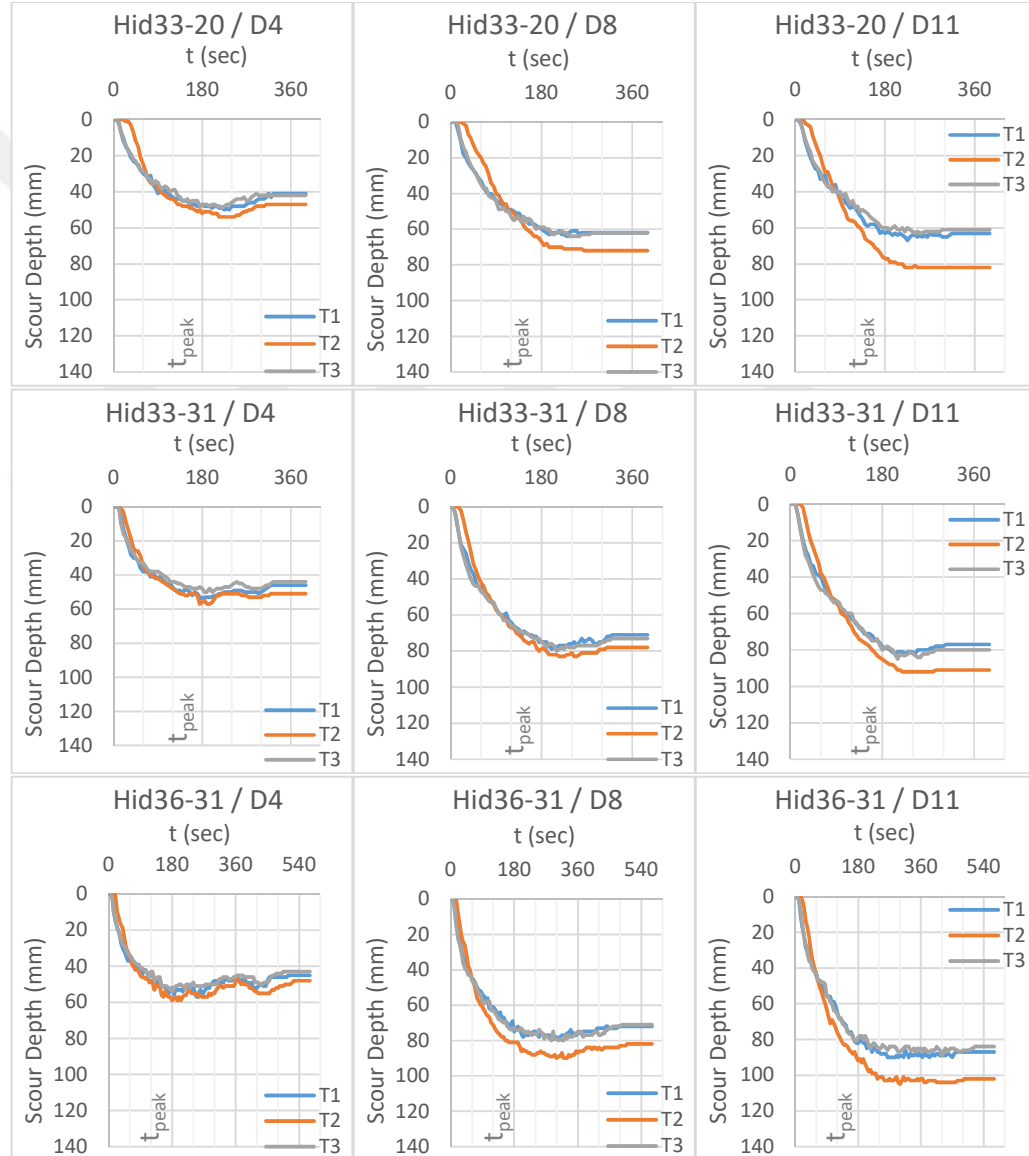


Figure A.1 Time dependent scour depths according to transducers

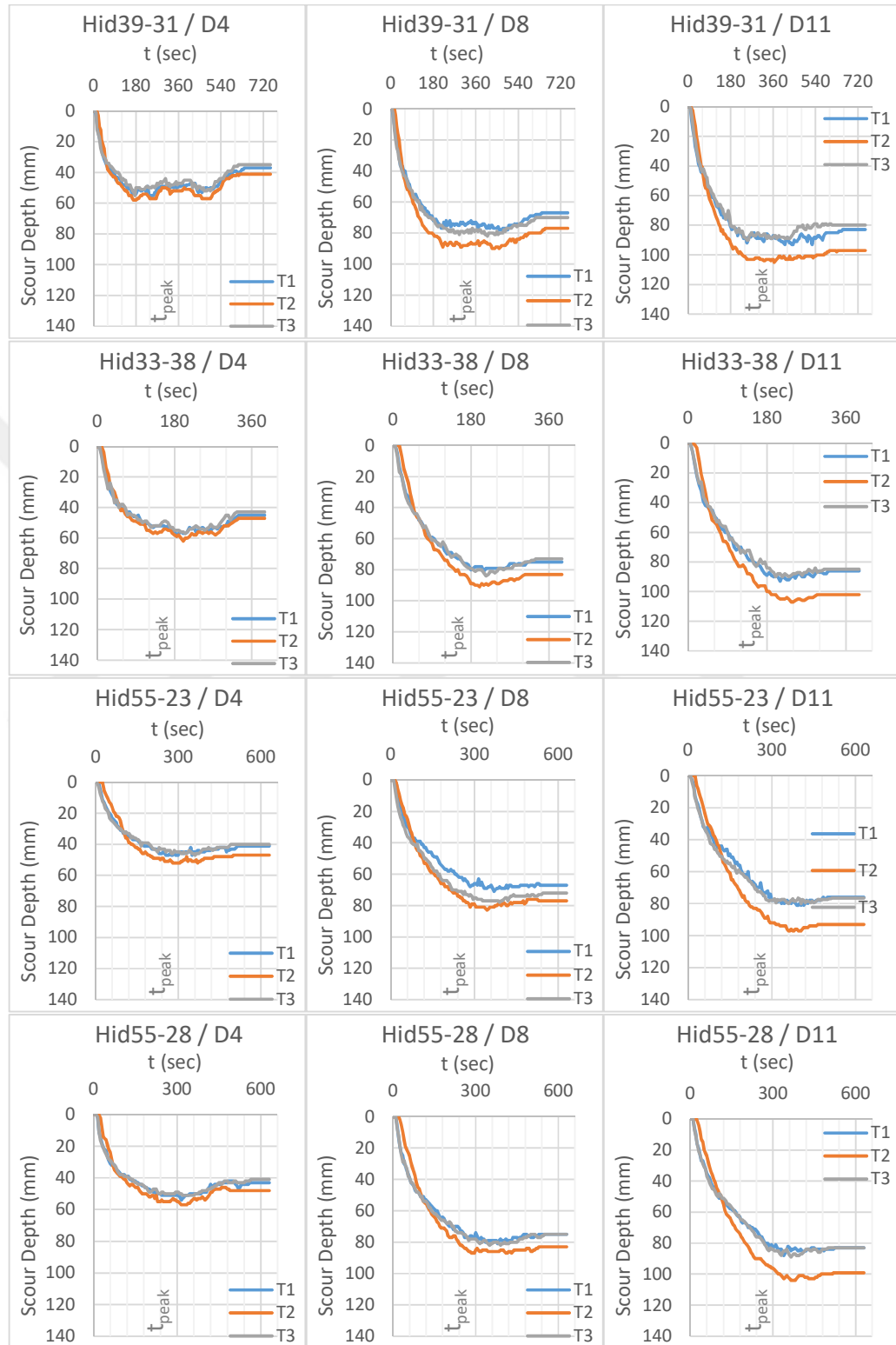


Figure A.1 continues

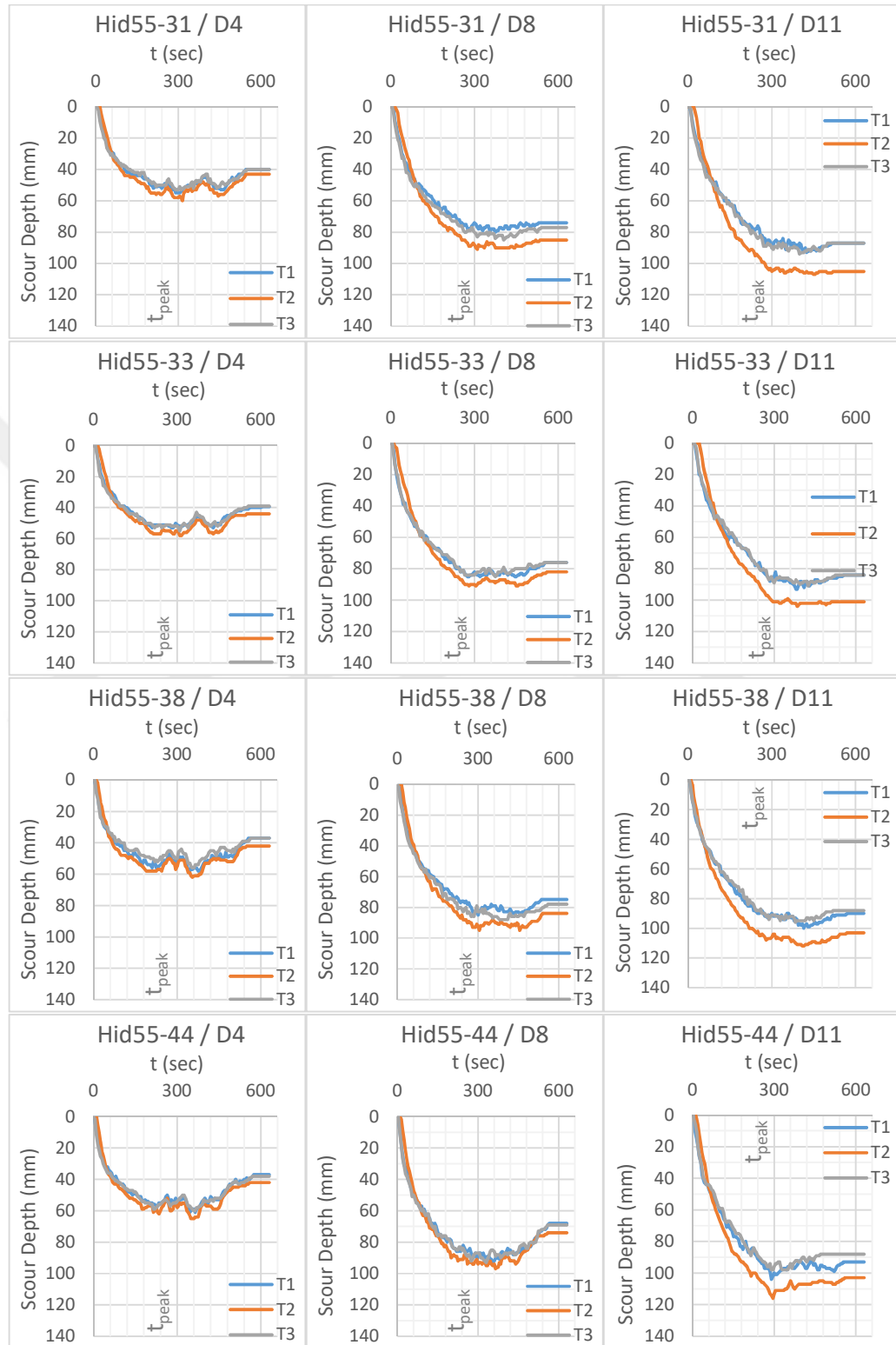


Figure A.1 continues

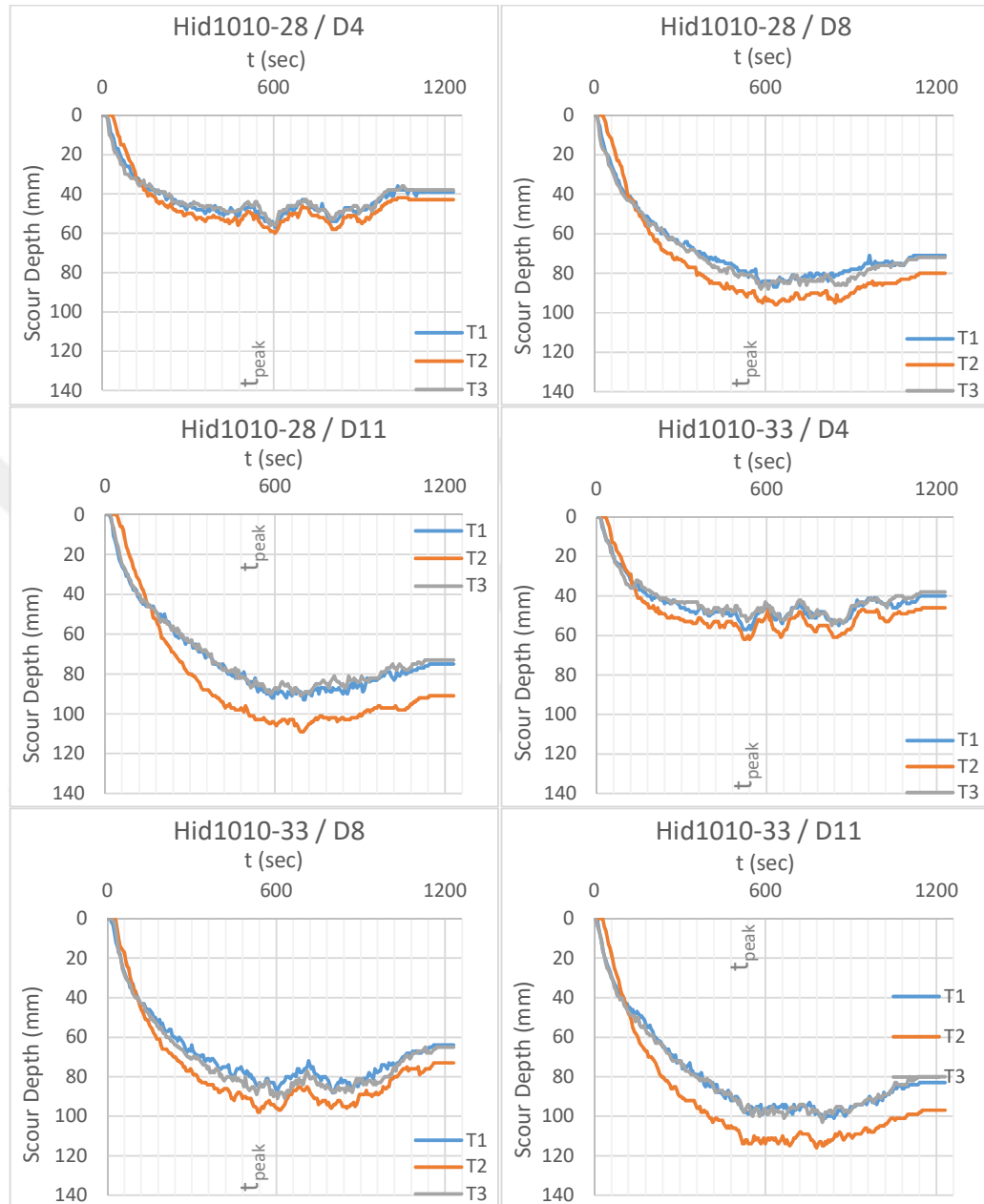


Figure A.1 continues

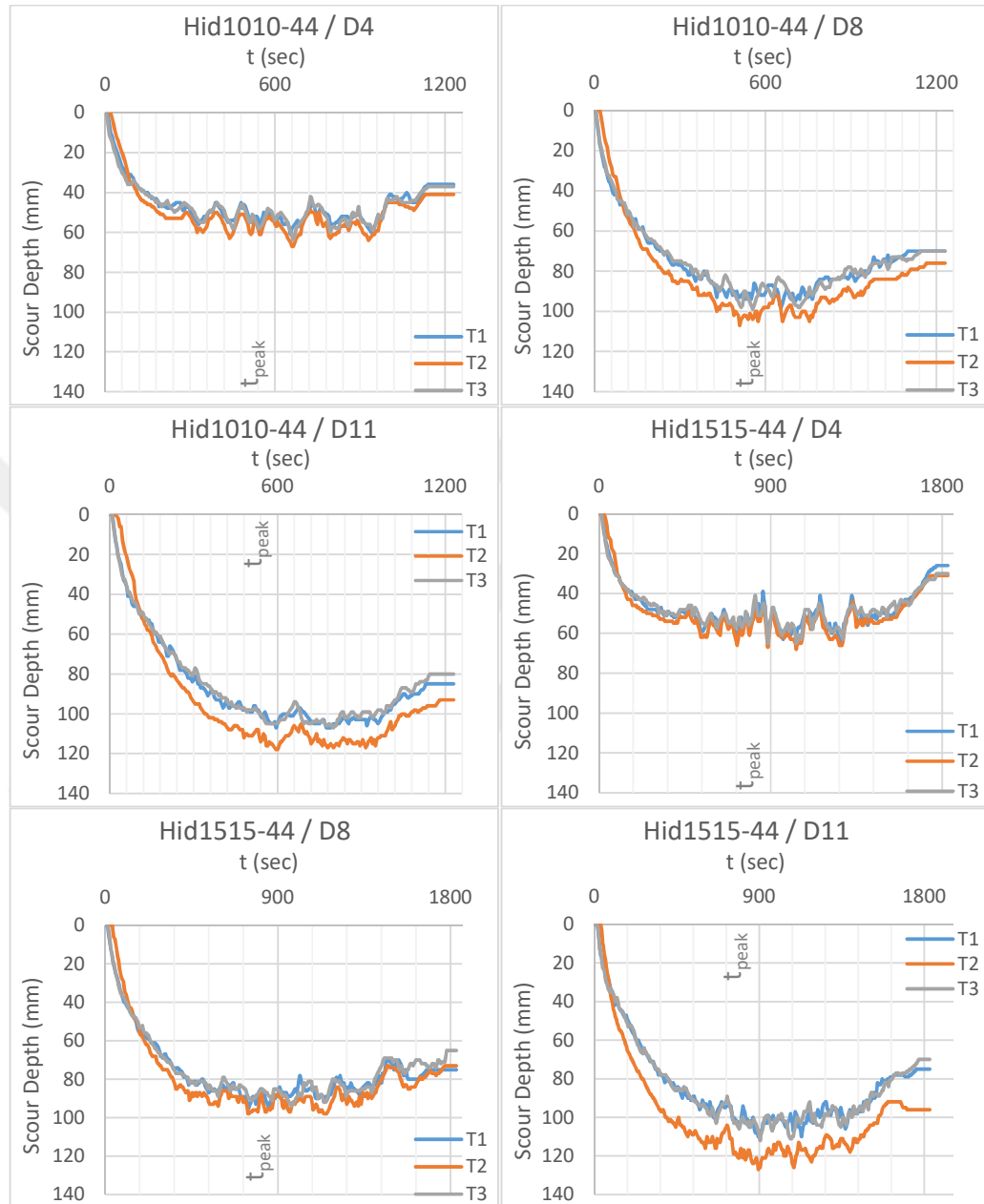


Figure A.1 continues

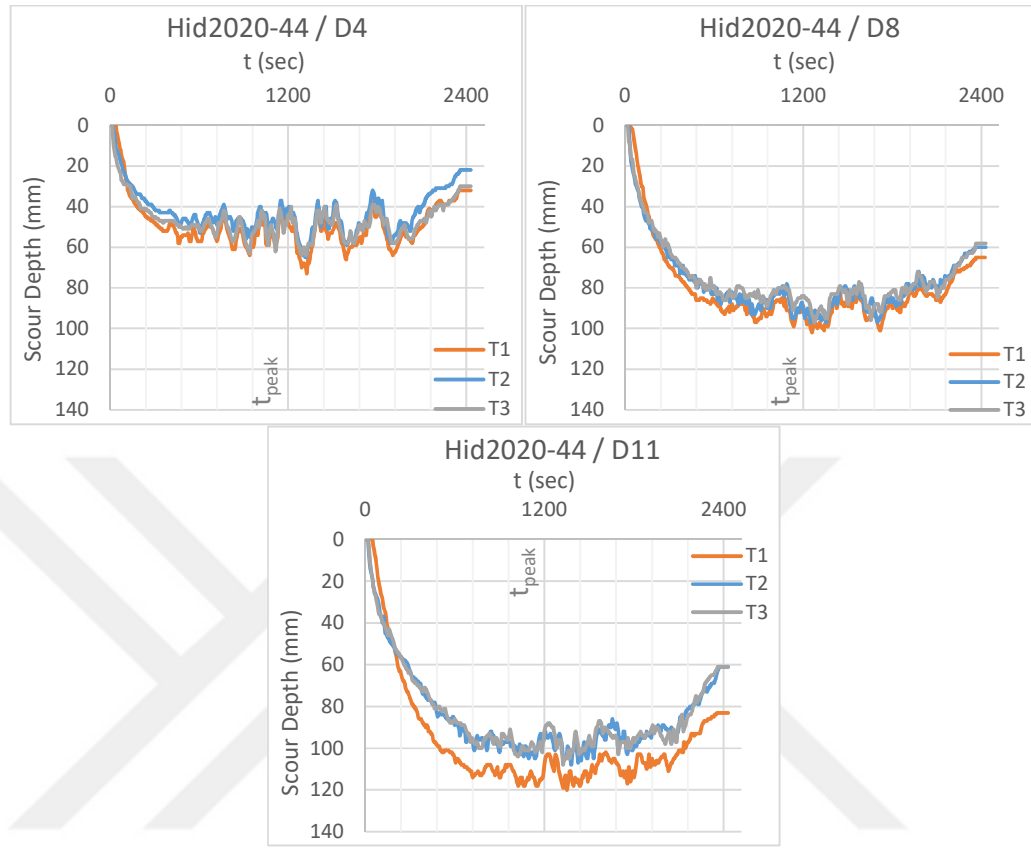


Figure A.1 continues

APPENDIX 2

SCOUR HOLE PROFILES

Scour depth profiles along center of the flow direction are given according to hydrographs in Figure B.1, B.2 and B.3, for the piers with diameters of 4 cm, 8 cm and 11 cm respectively.

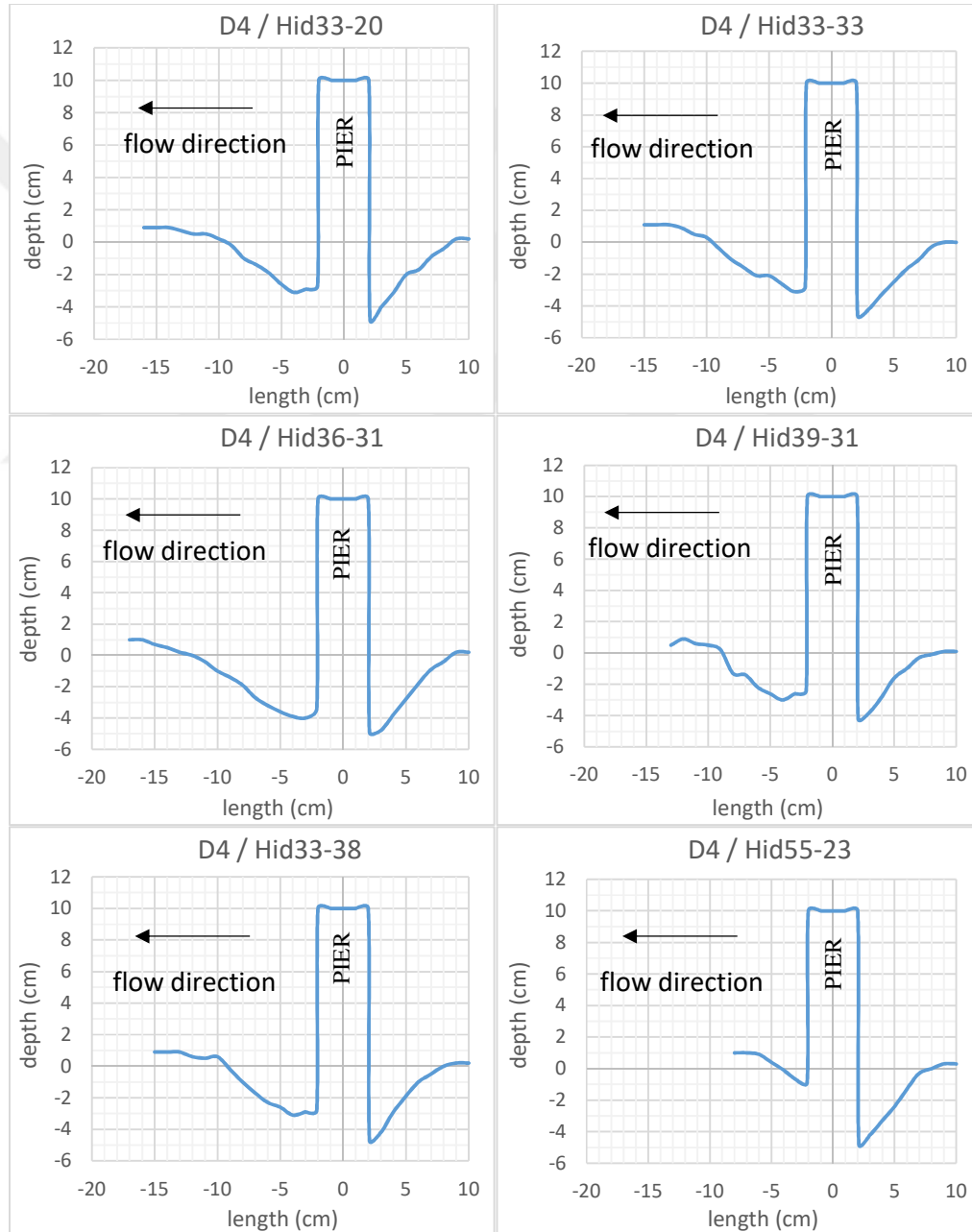


Figure B.1 Scour profiles according to hydrographs for the pier with 4 cm diameter

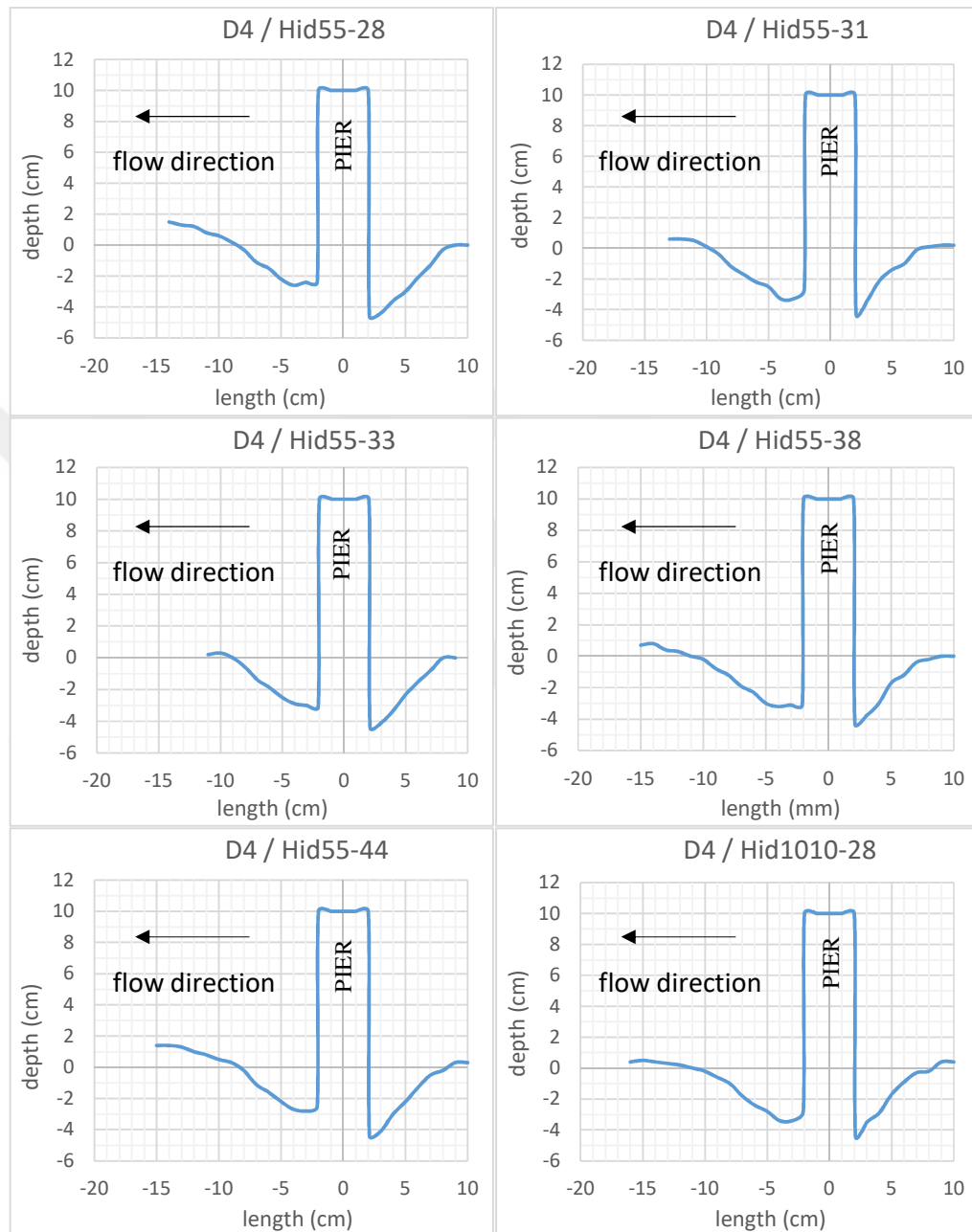


Figure B.1 continues

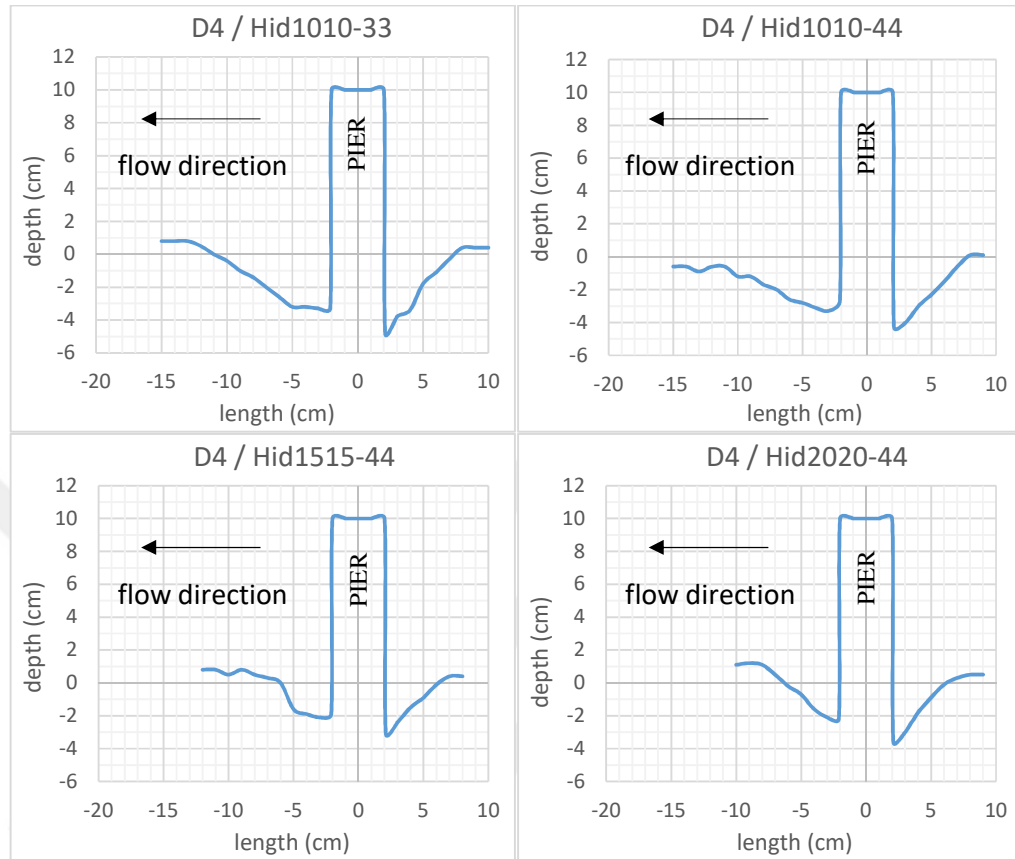


Figure B.1 Continues

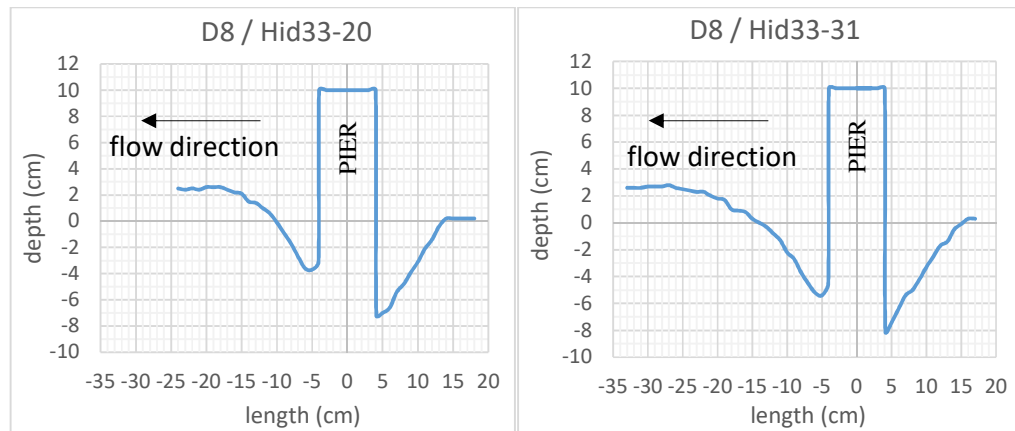


Figure B.2 Scour profiles according to hydrographs for the pier with 8 cm diameter

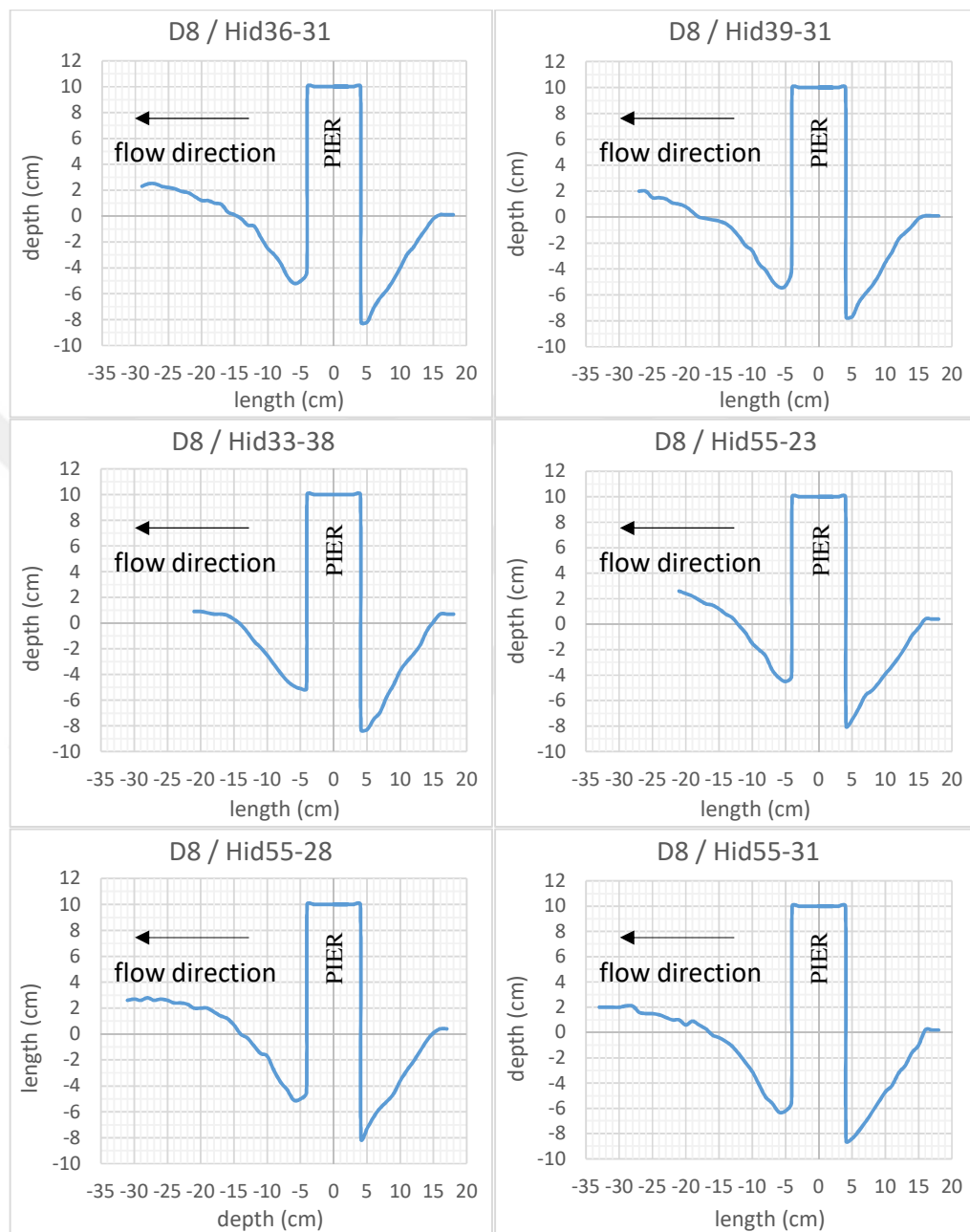


Figure B.2 continues

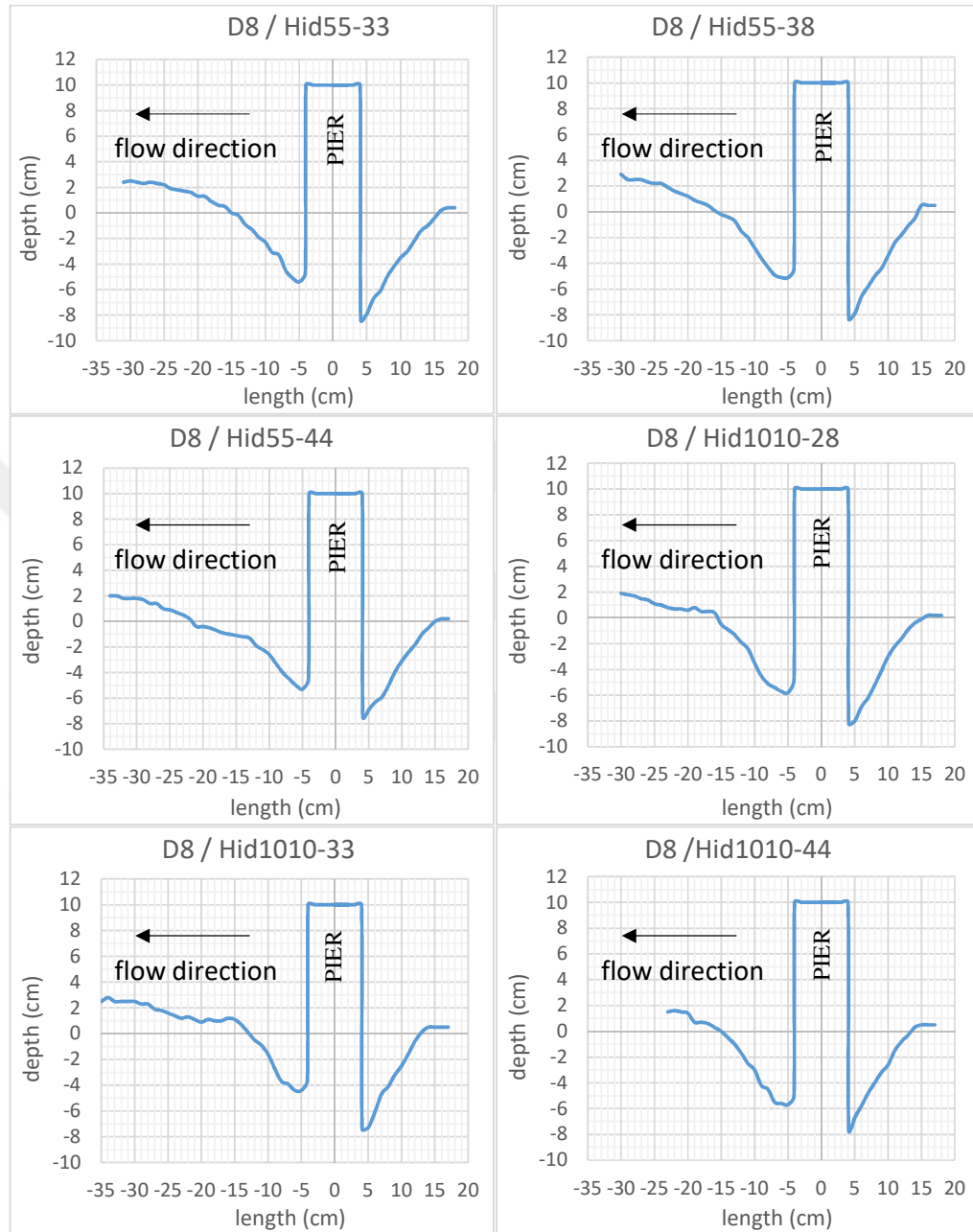


Figure B.2 continues

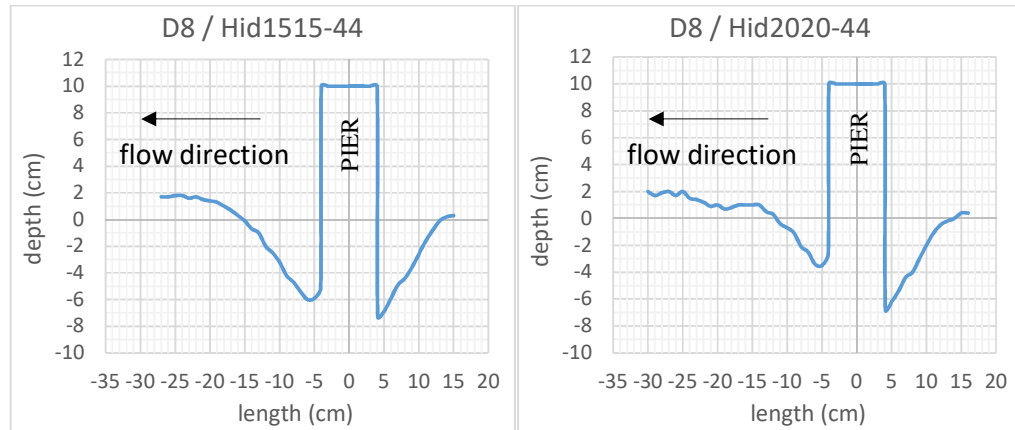


Figure B.2 continues

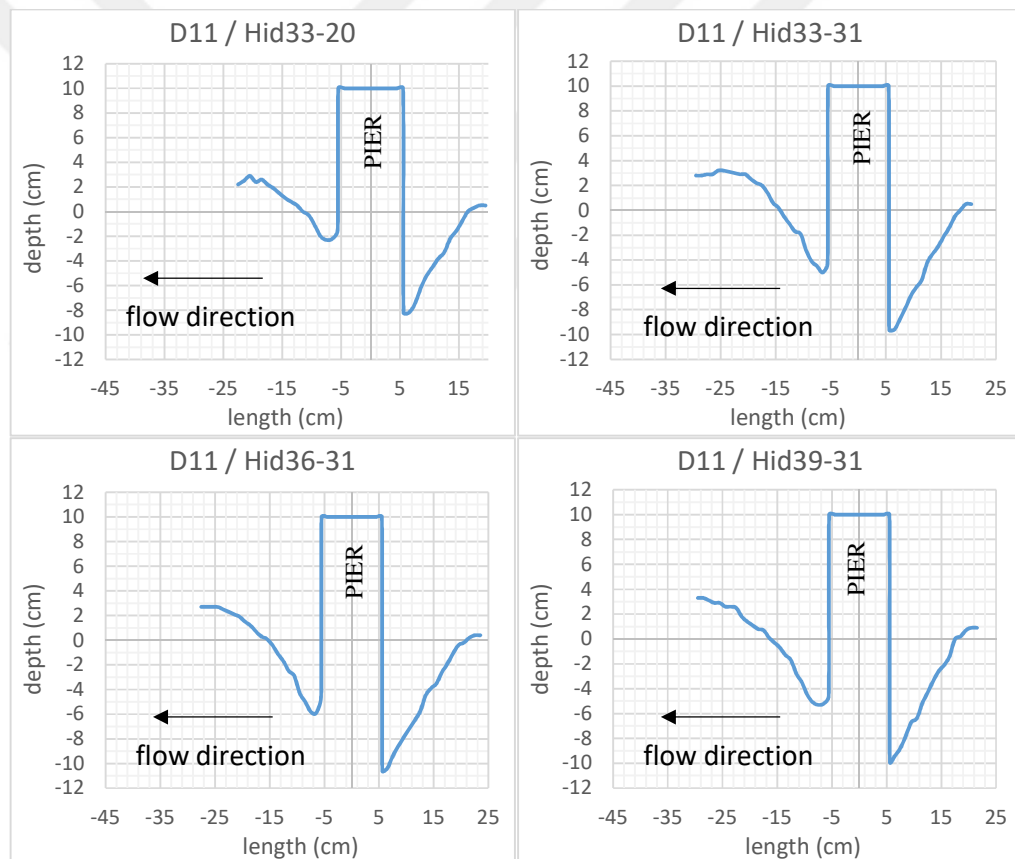


Figure B.3 Scour profiles according to hydrographs for the pier with 11 cm diameter

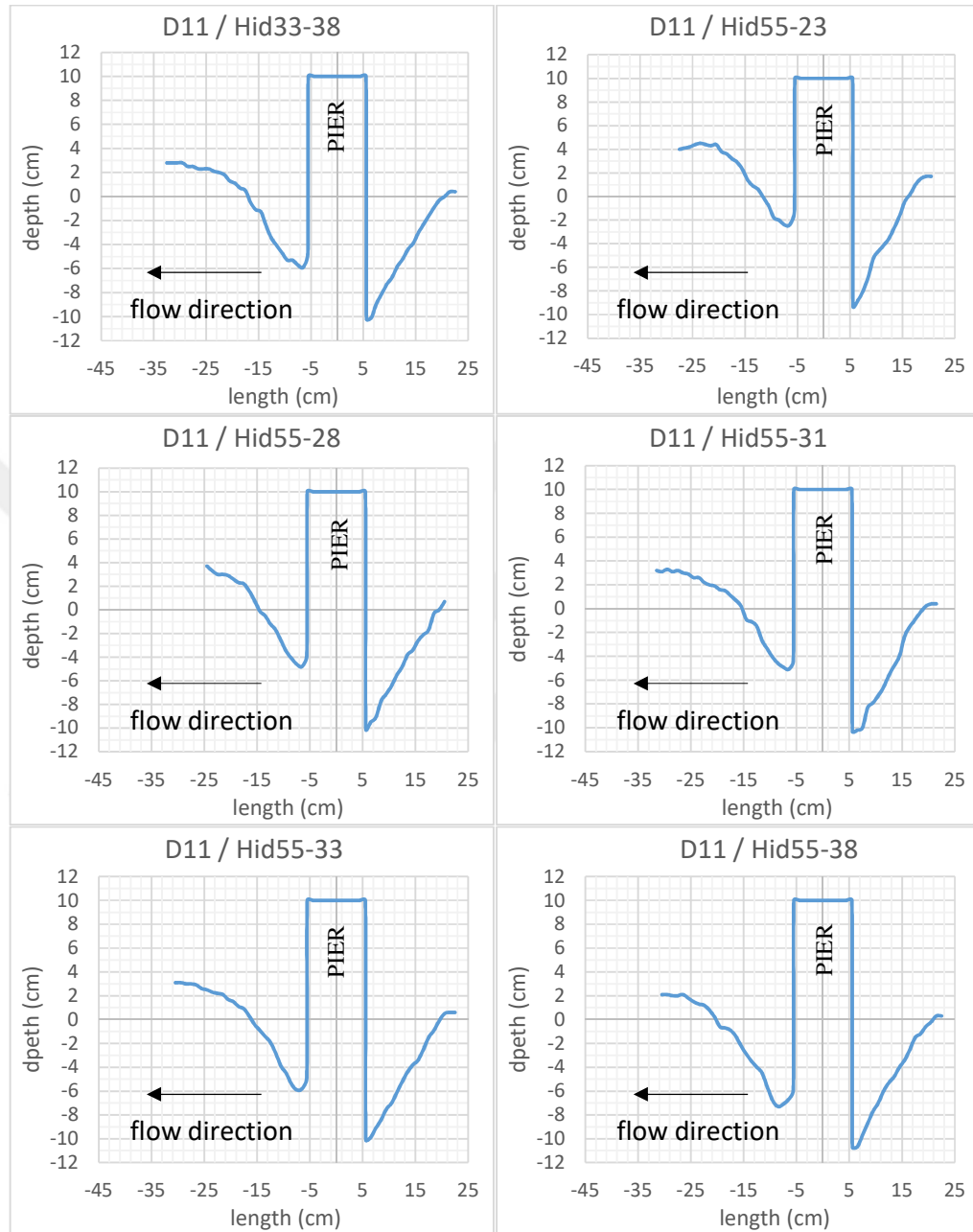


Figure B.3 continues

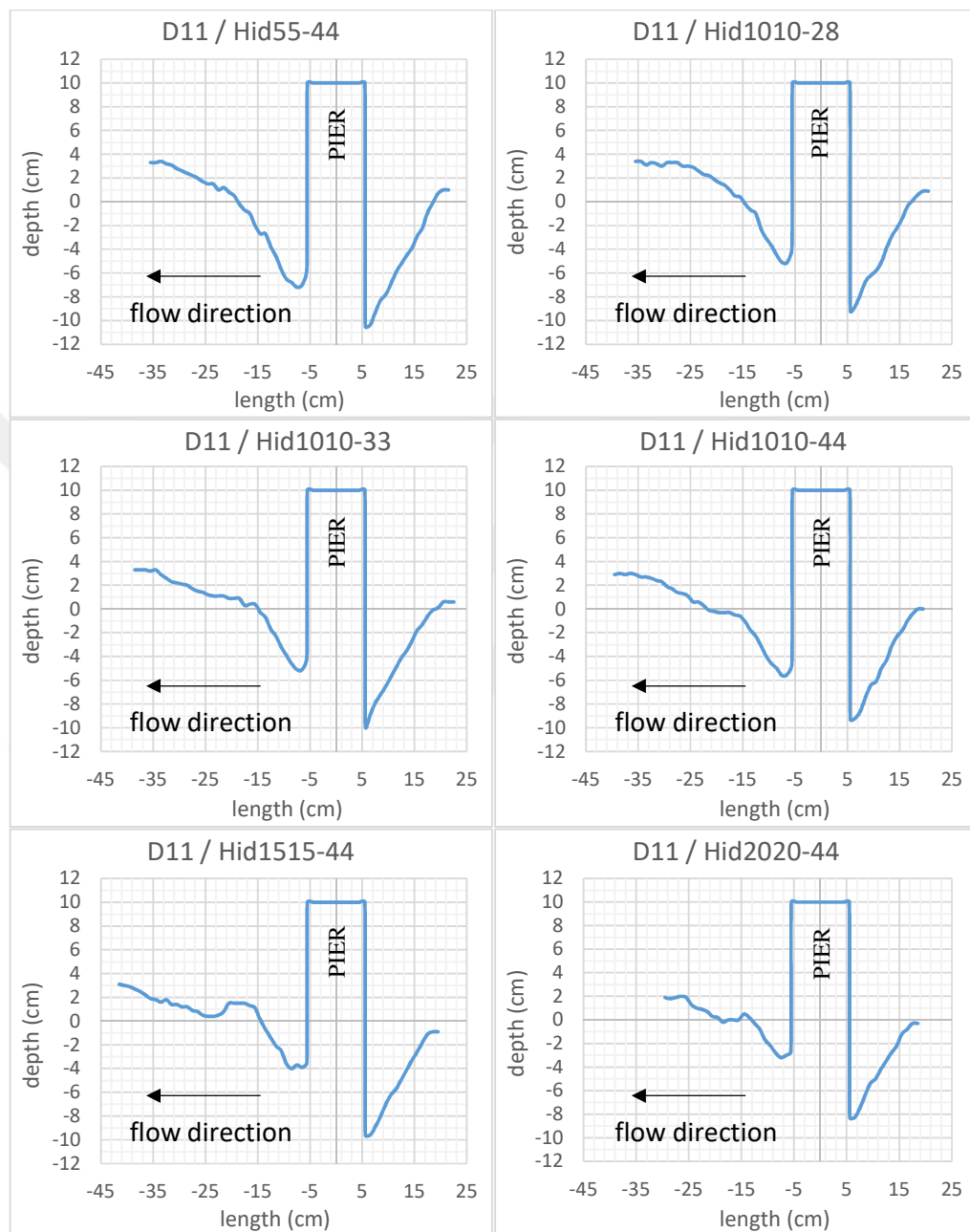


Figure B.3 continues

APPENDIX 3

COMPARISON OF EXPERIMENTAL TIME VARIED SCOUR DEPTHS

WITH THOSE OBTAINED FROM EQS. (5.6) AND (5.7)

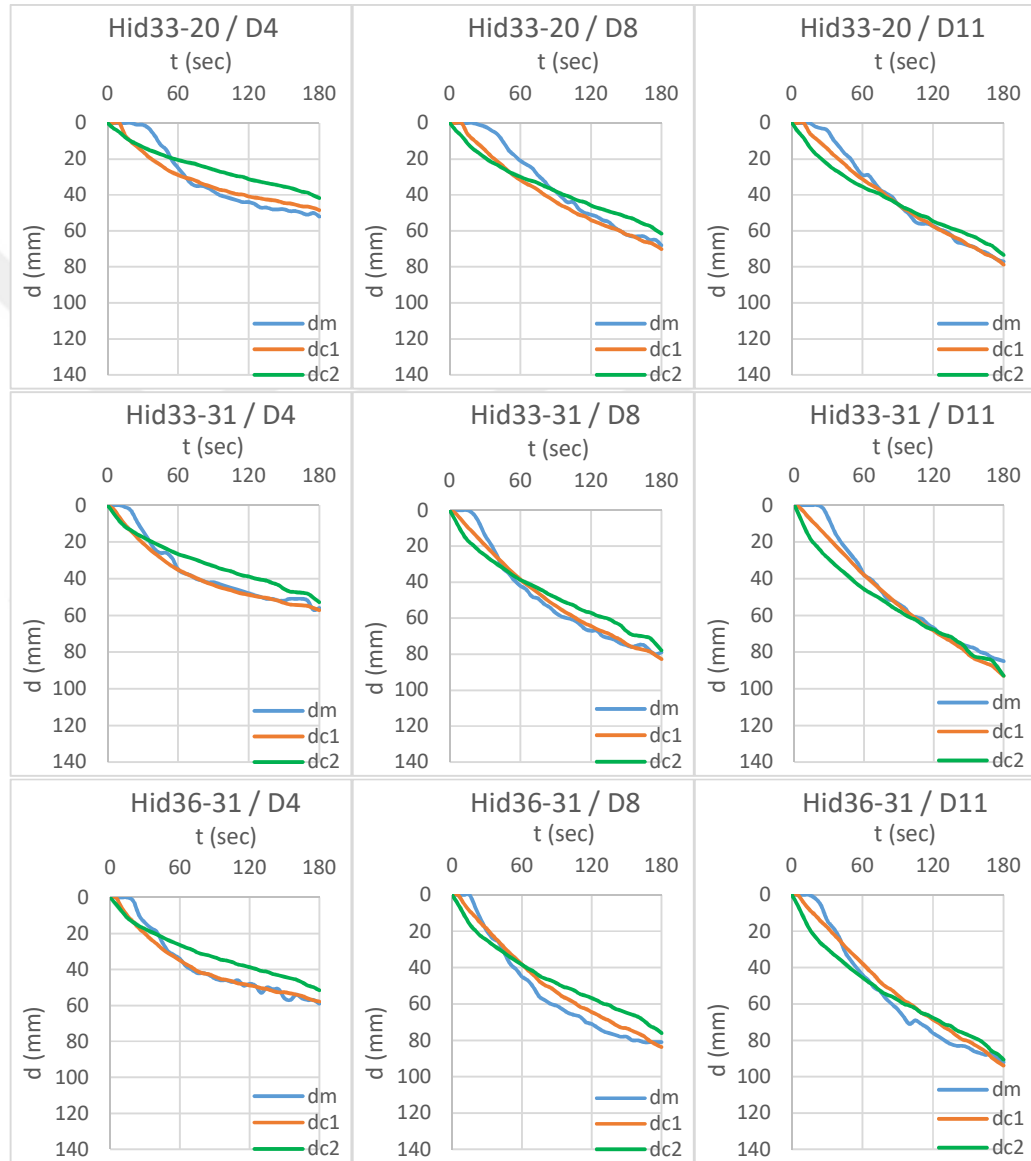


Figure C.1 – Comparison of experimental time varied scour depths with those obtained from the Eqs. (5.6) and (5.7)

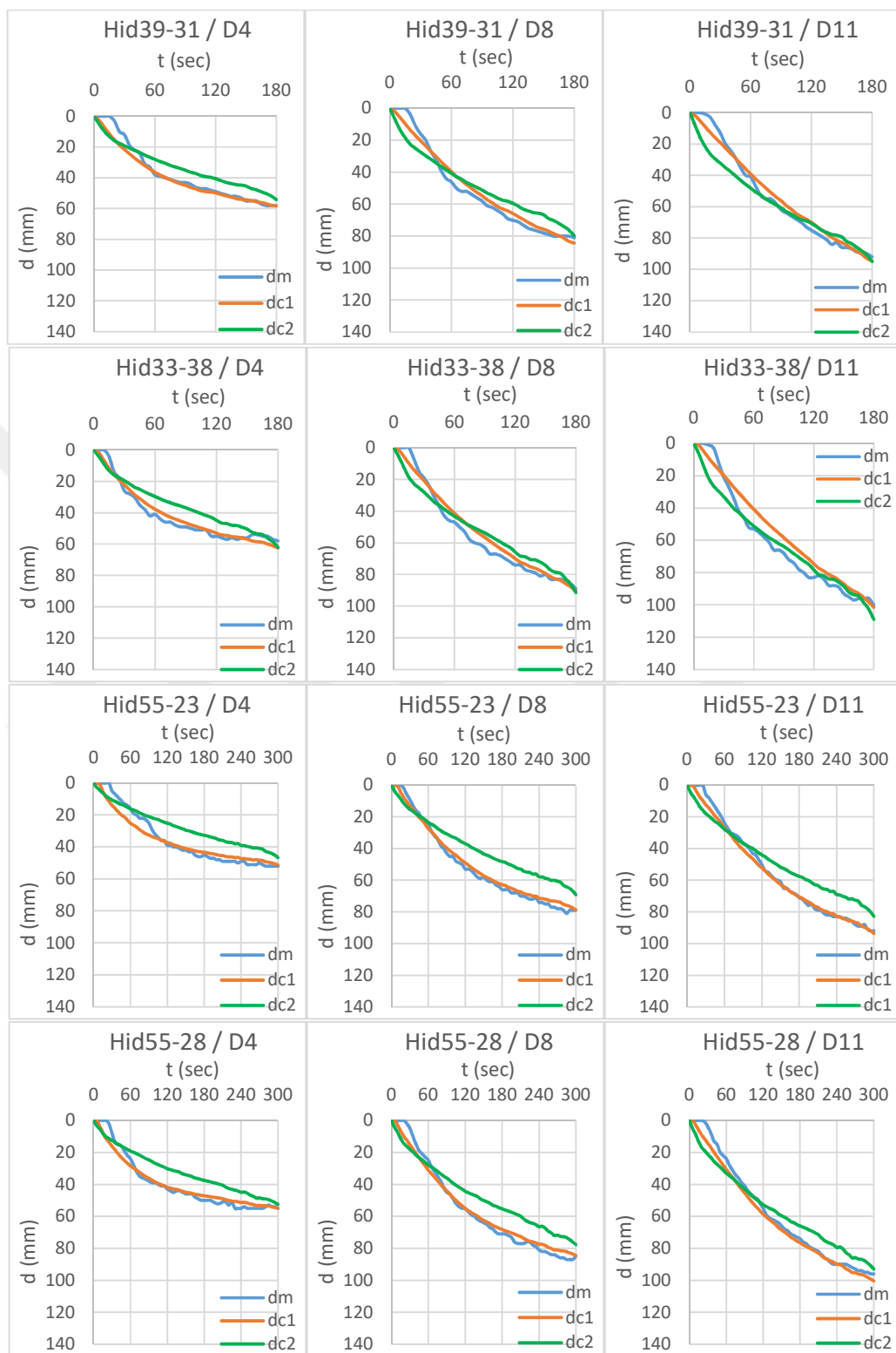


Figure C.1 continues

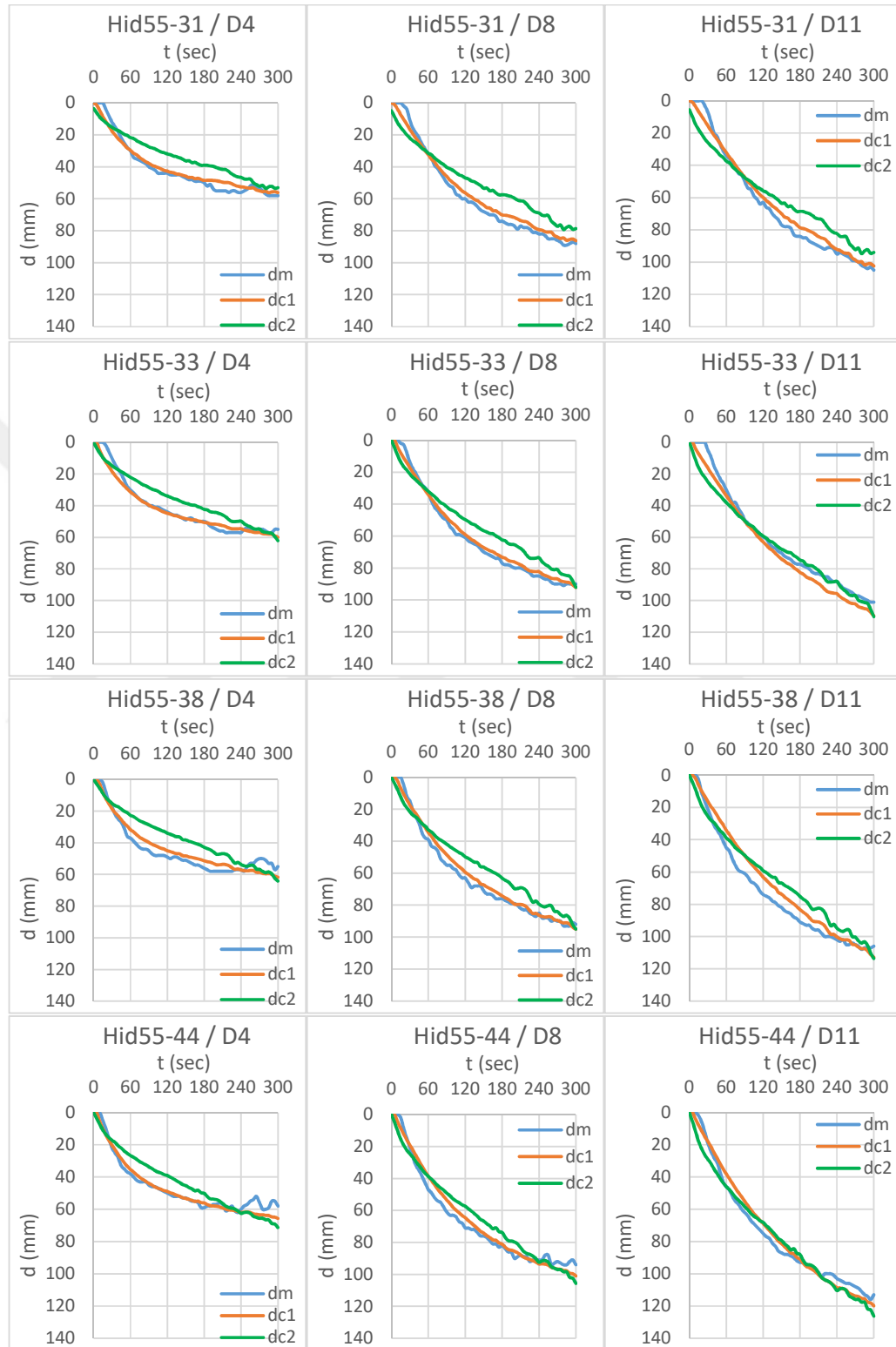


Figure C.1 continues

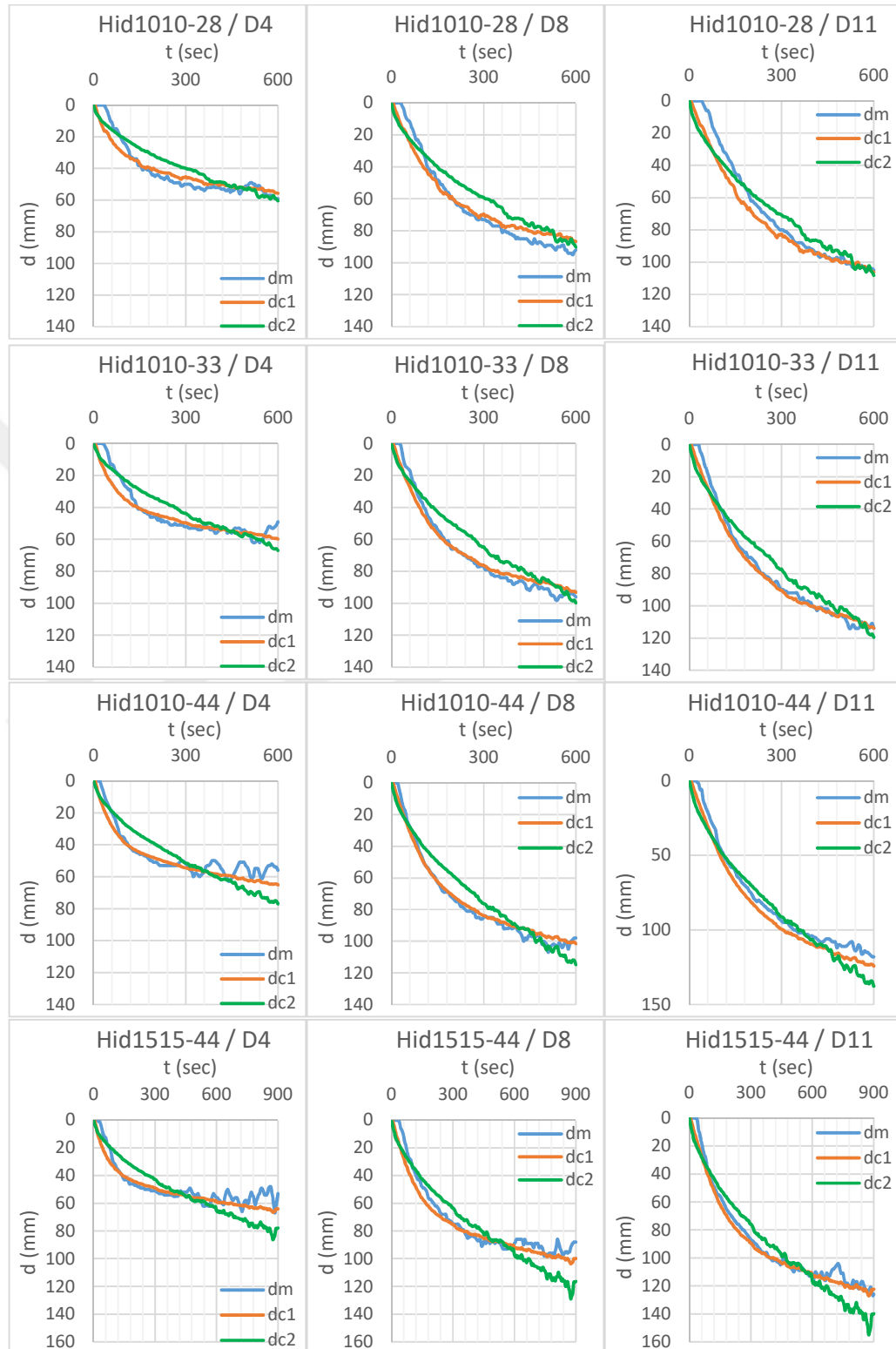


Figure C.1 continues

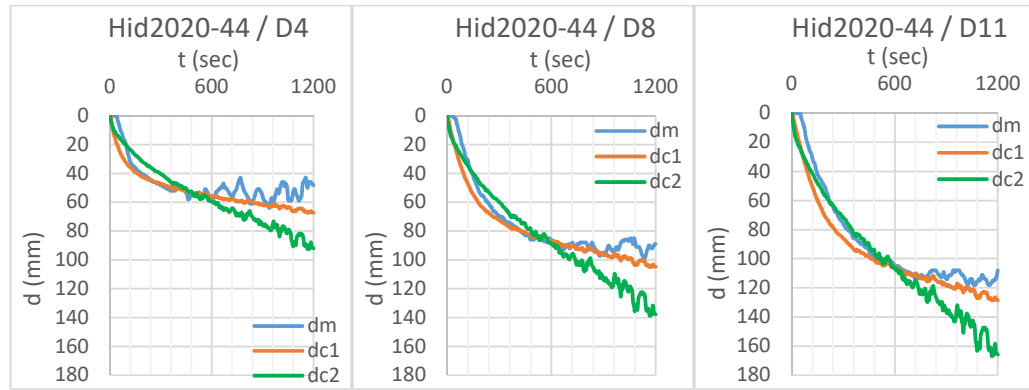


Figure C.1 continues