NUMERICAL AND FIELD INVESTIGATIONS
OF LOCAL BRIDGE ABUTMENT SCOUR
AND UNSTEADY DOWNSTREAM RIVER FLOW
FROM A NEARBY HYDROPOWER PLANT

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Abstract. Sudden and often fluctuations of water discharges and levels in the downstream reach of the hydropower plant make water fauna and flora existence conditions unbearable. Besides, a riverbed here is subjected to long lasting and intensive scour and deformations. The aim of our investigations is to develop measures improving the water flow regime in the downstream reach of the hydropower plants. The Nemunas River and Kaunas hydropower plants were selected as basis for our numerical and field investigations. Actual flow regime below the hydropower plant was observed and analysed on the ground of the long lasting dynamic measurement data. Numerical treatment of field measurement data and flow simulation works were arranged and carried out simultaneously. Numerical hydrodynamic model MIKE 21 made it possible to evaluate existing river flow conditions and to apply the model for simulation of many different turbine regimes and to select the optimal one. The content of our research was as follows: 1) analysis of typical turbines regimes while switching them on and off and flow regimes downstream; 2) continuous long lasting measurement of water level in many cross-sections of the river under investigation downstream the hydropower plants; 3) numerical simulation and investigation of untypical turbines regimes while switching them on and off and flow regimes downstream with application software MIKE 21 in the same unsteady flow conditions in which field measurements were made; 4) evaluation of the accuracy of hydrodynamic modeling results and possibilities for their application in predicting the local bridge abutment scour. The obtained data on water level and flow rate dynamics downstream the Kaunas hydropower plants have been analysed. Numerical model was developed to simulate the scour at bridge abutment and was calibrated for the same unsteady flow conditions. The obtained accurate results can be used for local bridge abutment scour evaluation.

Keywords: hydrodynamic model, Kaunas hydropower plant, hydropower plants (HPP), bridge abutment scour.

1. Introduction

The practice show that the hydropower plants (HPP) and the regulation of rivers runoff change their hydro-morphological characteristics. Sudden starting and stopping of HPP turbines due to uneven demand of electric power during a 24-hour time period cause stream flow fluctuations in the river which have an effect on riverbed deformations, local and general scours of the riverbed and ultimately – changes of ecosystems and hydraulic-hydrologic state (Labutina 1985; Milius, Darbutas 2011). In recent years several studies have been published about unsteady water flow in river stretches where it is affected by irregular operation of HPP (Kaganov *et al.* 2010; Šikšnys, Sabas 2011; Vaikasas, Poškus 2009; Ždankus, Sabas 2005, 2006; Ždankus *et al.* 2008). Lithuanian researchers often discuss problems related to a negative impact of HPP on ecosystems; suggest different measures to reduce this negative impact. In addition, stream flow fluctuations in the plain rivers are a scour result of erosive action around the piers and abutments of bridges (Richardson, Davis 2001; Ždankus 2000). Local scour arises around abutments, piers and embankments, which is caused by the rapidity

of the flow. Many studies have been carried out to develop the relationships for predicting scour depth rate at bridge abutments under stream flow scour condition and these relations have been used for designing purposes (Esmaeili *et al.* 2011; Moncada-M *et al.* 2009; Zarrati *et al.* 2006). The prediction of damages or failure of the bridge structures in flow, causing the scour near bridges in rivers, is one of the main tasks for engineers (Gjunsburgs *et al.* 2014; Kamaitis 2012; Skibniewski *et al.* 2014).

It is established that the largest and the most intensive fluctuations of water levels and flow velocities are observed in the downstream reach of the HPP dams and gradually weaken in the flow direction. Rather complex hydraulic problems involving unsteady water flow in many cases were approached using the simplest calculation methods where one-dimensional Saint-Venant, Bernoulli and continuity equations, Newton's second law etc. were applied, complex morphometric characteristics of natural riverbeds were calculated using simplified prismatic riverbed schemes not taking into consideration geometric shapes of a river channel and its variety of hydraulic roughness, ignoring the uneven distribution of flow velocities across the width of the river, vortex zones and other important factors. Such a schematization of hydraulic calculations can affect significantly the accuracy and the reliability of obtained results. For example, Kaganov *et al.* (2010) several parameters (water levels, water discharges, wave motion velocities, etc.) of a natural gravitational wave are compared using different methods: using "BOR", "RIVER", "MIKE 11" computer programs. It was established, that water flow wave height errors reached 20%, wave motion velocity errors – 16% and water level rising time errors – 28%.

Irregular operation of Kaunas HPP (the largest in Lithuania) turbines induces flux and reflux waves in the Nemunas river near Kaunas, therefore flow velocities, water levels and water discharges vary very intensely in a rather long river stretch. In the study (Ždankus, Sabas 2005) the modelling exercise was performed by simulating several cases of unsteady water flow observed in the downstream reach of the HPP and in a rather short (1.3 km) stretch of the Nemunas River using the two-dimensional

Fig. 1. The stretch of Nemunas River under study, water level measuring points: 1 – Kaunas HPP; 2 – Panemune Bridge; 3 – Railway Bridge; 4 – Aleksotas Bridge

numerical MIKE 21 hydrodynamic model. Results showed that: 1) flux wave motion velocities in the nearest stretch of the downstream reach of the HPP simulated using the MIKE 21 hydrodynamic model are much larger than velocities calculated using formulas given by the other authors, however, these differences gradually decrease in the flow direction moving away from the HPP; 2) flux wave motion velocity in the nearest zone to HPP does not depend on wave water discharge and its height, however, in the subsequent flow stretch wave motion velocity increases when wave water discharge and its height increase; 3) for the estimation of the reliability of hydrodynamic modeling results field measurements are necessary.

Why are the reliability and the accuracy of obtained results so important while investigating different cases of unsteady water flow in the Nemunas River below the HPP? Such approach is necessary in order that many important questions about the safety of Kaunas residents and the Nemunas River ecosystem could be answered reasonably, e.g.: 1) What is the real threat to the "Zalgiris" arena in the Nemunas River island in case of an accident on HPP? 2) How long would it take for a wave to reach the Nemunas River Island in case of an accident on HPP and what would be the height of such a wave? 3) How does the Kaunas HPP affect the navigation conditions in the Nemunas River? 4) How can a negative impact of the Kaunas HPP on the river ecosystem be reduced?

The main goal of this work is to analyze characteristic cases of unsteady flow in the downstream reach of the Kaunas HPP, investigate them using the MIKE 21 hydrodynamic model, and estimate the reliability of modeling results and the suitability of the model for solving practical problems.

2. Investigation methods

The whole investigation was split into several stages and conducted in the following order: 1) Kaunas HPP characteristic operation regimes causing unsteady flow in the downstream reach of HPP were analyzed; 2) synchronous field measurements of water level dynamics in the Nemunas River near Kaunas were performed for various Kaunas HPP operation regimes; 3) numerical hydrodynamic model MIKE 21 was used to investigate the dynamics of hydraulic characteristics of the Nemunas River under the same conditions of unsteady water flow as in field measurements; 4) the accuracy of modeling results was estimated and the possibilities of application of these modeling methods to solve practical problems was evaluated.

In this work the authors investigated the unsteady water flow in the stretch of the Nemunas River from Kaunas HPP to the mouth of the Neris River (Fig. 1). The length of this stretch is 15.3 km. Analysis of water level fluctuations in this stretch was performed in four places of the river cross-section (measuring in distance from the river mouth): 225.82 km (near the Kaunas HPP below dam); 218.48 km (at the Panemune Bridge); 214.46 km (at the Railway Bridge); 211.54 km (at the Aleksotas Bridge). At these control points accurate synchronous measurements

of water level dynamics were carried out: from 2004.09.20 until 2004.10.11 and from 2005.07.04 until 2005.07.22 (Fig. 2). For this purpose autonomous pressure sensors with programmable data drives were used. The first series of measurements were performed with the period of 1 minute and the second – with a 5-minute period. The accuracy of measurements was ±1 mm in both cases. Pressure sensors were submerged under water and placed on the bottom of the river. During this operation river water levels were measured at all four control points. Measurement results were transferred to a computer where barometric corrections were made and altitudes of water surface were calculated.

Data collected by the Kaunas HPP on turbine water discharges and water levels is sufficiently accurate (accuracy of water levels ±1 cm, water discharges – ±1.0 m³/s) though the period of measurements is quite long – 1 hour.

Using the results of water level measurements the heights of flux waves caused by HPP as well as their motion velocities and the intensity of water level fluctuations were calculated and analyzed. The principles and methods of numerical modeling of channel flows with MIKE 21 programs are discussed by Ždankus *et al.* (2008), Šiksnys and Sabas (2011). It was tested an advanced method of unsteady water flow modeling using MIKE 21 program. The basis of the two-dimensional numerical hydrodynamic model is the digital terrain model (DTM) of the Nemunas riverbed which was prepared using GIS software and the bathymetric measurement data of the Nemunas River provided by the Lithuanian Inland Waterways Authority. These measurements were carried out in 2005 when the possibilities of cargo and passenger shipping on this water route were studied. The above mentioned model consists of a rectangular grid (dimensions of an element 5´5 m, the total number of elements exceeds 230 thousands. The DTM of the riverbed was converted from GIS format to MIKE 21 format. The accuracy of the DTM of the riverbed was tested visually with programs designed for the visualization of MIKE 21 data and also using hydraulic characteristics (riverbed roughness coefficient, flow widths and velocities) simulated during the model calibration process.

The analysis of the graphs representing the operation regimes of the HPP showed that on weekends, when the consumption of electrical energy decreases, Kaunas HPP often operates in the stable 1 turbine mode with an exception of flux periods and special circumstances in the power grid. For this reason on weekends the water flow in the Nemunas River below the Kaunas HPP usually stabilizes and when on Mondays other turbines of the HPP are started, the river water discharges and water levels begin to increase (Fig. 2).

For these reasons the modeling of unsteady water flow was conducted in two stages: 1) modeling of steady water flow under HPP operation regime typical for weekends, which allows the calculation of steady flow velocities and water levels in the whole river area under study; 2) modeling of unsteady water flow under HPP operation

regime typical for Mondays, which allows the calculation of flow velocity and water level changes in each time step.

The first stage of modeling was matched to the model calibration procedure. The final results from the fully calibrated and tested hydrodynamic model obtained in the last time step of the first stage were used in the second stage model for the calculations of unsteady flow during the first time step.

The HPP operation regime selected for investigations from several discussed weekend versions was the one which allows to reveal and to measure more accurately the characteristics of flux waves – the height and velocity of the wave, flow velocity and water level changes, their intensity, etc. In this respect the most appropriate HPP operation regime was observed on 2005.07.17 which was characterized by a rather long period of steady flow with one active turbine until 07:20 hours. After that followed a sudden start and short operation for several hours of additional 3 turbines and from 10:30 hours 3 turbines were stopped and again a single turbine of the HPP was operational for a rather long time (Fig. 3).

At the open lower boundary of the model under steady flow conditions a varying water level at the mouth of the Neris River was observed (WL – Neris Fig. 3). It was calculated using actual heights of the water surface measured on 2005.07.18 at the nearest measurement point at the Aleksotas Bridge. The slope of the water surface typical for this stretch of the river, the distance between the measurement point at the Aleksotas Bridge and the mouth of
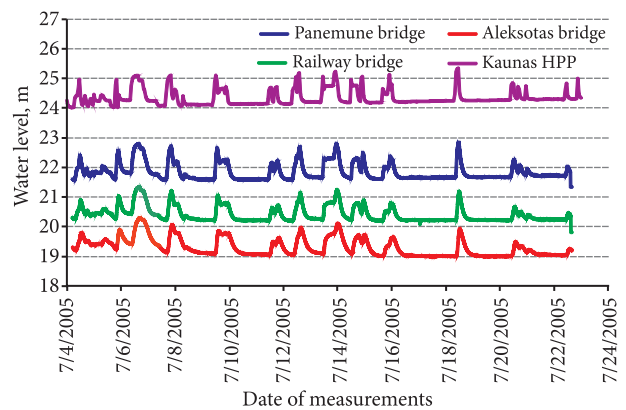

Fig. 2. Dynamics of water levels measured in the Nemunas River near Kaunas 2005.07.04 – 2005.07.23
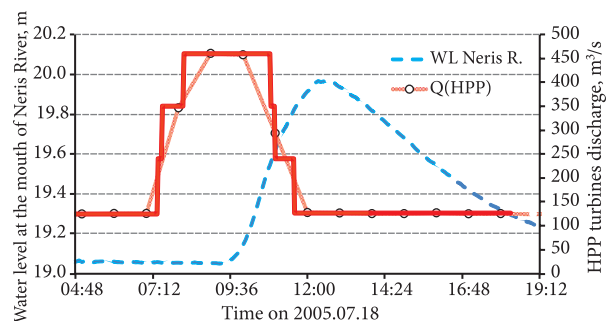

Fig. 3. Hydrodynamic model background conditions

the Neris River and the duration of motion of flux waves in this stretch were used in these calculations.

At the open upper boundary a varying water discharge of the HPP was obtained. As it was already mentioned, water discharges of the HPP are measured rather infrequently, with the period of 1 hour. A turbine launch takes only about 2 minutes so the water discharge in the downstream reach of the HPP increases rather rapidly. The exact time of turbine start in an 1 hour interval is not known, therefore at the upper boundary of the model 2 graphs of water discharge variation were used: 1) Q (HPP) consists of actual HPP water discharges measured each hour; 2) Q (mod) consists of actual HPP water discharges but in this case turbine start and stop times were specified so that modeling results would match water level dynamics measured at the measurement points in the river.

The discussed hydrodynamic model was used to calculate varying water levels and flow velocities in the whole stretch of the Nemunas River under study over a period of 24 hours on 2005.07.18. Modeling results (water level dynamics) were analyzed and compared with water levels observed at the measurement points in the Nemunas River. These results were also used to make conclusions about the reliability of the hydrodynamic model and its practical application possibilities.

Many studies have been carried out to value a scour at bridge abutments (Salaheldin *et al.* 2004; Voskoboinik *et al.* 2004). The authors analyzed local bridge abutment scour steady and unsteady downstream river flow from a nearby hydropower plant. The guide was used to count the local scour depth at the Railway Bridge abutment and other hydraulic characteristics with different HPP work regimes. Flow velocity before bridge abutment has the form:

$$V_N = \frac{V_0}{K_v}, \qquad (1)$$

where V_0 – a non-scouring mean flow velocity in vertical (when the open channel bottom is horizontal), m/s; K_v – velocity coefficient for recalculating mean flow velocity to the maximum flow velocity before abutment:

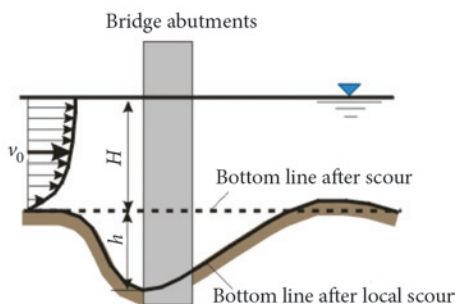$$K_v = \exp\left(\frac{1.05H}{H + \frac{b}{2}}\right), \qquad (2)$$

Fig. 4. Principal scheme of local bridge abutment scour

where b – abutment width, m; H – water depth except scour depth, m (Fig. 4).

The maximum depth of the local bridge abutment scour hole in a moment of stabilization is calculated by the formula:

$$h = \left(\frac{V}{V_N}\right)^{\frac{V_N}{V}-1}\left(\frac{V}{V_N}-1\right) \cdot H^{1-m} \cdot b^m \cdot M \cdot K, \qquad (3)$$

where V – mean flow velocity before bridge abutment, m/s; M – coefficient of bridge abutment depending on shape; K – coefficient of geometry depending on flow and abutment situation conditions; m – exponent depending on the bottom sediments carried by the flow and forming the riverbed; when the bottom sediments get into the scour hole $m = 0.5$; otherwise $m = 0.6$.

The depth of local bridge abutment scour during flooding is estimated by dividing a flood hydrograph into elementary time steps having made an assumption that in every time step the water flow is steady (*Technical Directions for the Calculation of Local Bridge Abutment Scour*). In this case the depth of local scour is estimated separately for each time step from the initial scour depth hpr (i.e., in the beginning of the estimated time step) until the end of the estimated time step h_g by the formula:

$$F(y_g) = F(y_{pr}) + \frac{\Delta t}{\tau}, \qquad (4)$$

where $y_{pr} = \dfrac{h_{pr}}{h}$ – relative scour depth in the beginning of the estimated time step; $y_g = \dfrac{h_g}{h}$ – relative scour depth at the end of the estimated time step; Δt – duration of time step, h; τ – timescale of the unsteady process of the local bottom soil scour, h.

The timescale of the local bottom soil scour under the stable water flow conditions is estimated in hours by the following formula:

$$\tau = 0.008\left(\frac{V}{V_N}\right)^{2\frac{V_N}{V}-7}\left(\frac{V}{V_N}-1\right)^2\frac{\left(H^{1-m}b^m MK\right)^2}{V_N d}, \qquad (5)$$

where d – average diameter of a soil particle, m.

To facilitate estimations a methodical material was used (*Technical Directions for the Calculation of Local Bridge Abutment Scour*) also the tables and graphs of local scour of bridge abutments of various shapes, dimensions and structures. The flow velocities V and depths H were selected according to the hydrodynamic modeling results. With the help of field investigations the estimated width of the abutment was determined $b = 6$ m, also the prevailing soil of the bottom sediments in the Nemunas River – fine sand, average diameter of sand particles $d = 0.2$ mm, the bottom of the riverbed at the bridge abutment – not strengthened. The underwater part of bridge abutment is strengthened by a vertical wall of steel dowels, the front part of the wall is rounded, the coefficient of abutment

shape $M = 0.85$ was used (*Metodical Recommendations for the Calculation of Local Bridge Abutment Scour*).

The bridge abutment scour estimations were carried out under various conditions of steady water flow: 1) under the operation of 1–4 turbines of Kaunas HPP; 2) under the maximum water discharge of spring flood of 10% reliability. Intensity of the non-stationery abutment scour was estimated under conditions of unsteady water flow when all four turbines of Kaunas HPP are put into operation and when water discharge increases from 120 to 480 m³/s.

Based on the obtained estimation results the possible scour of bridge abutments was analyzed and evaluated due to unsteady water flow in the downstream reach of Kaunas HPP.

## 3. Analysis of water level measurement results

Analysis of the dynamics of water levels measured in the Nemunas River showed that the increase of operational turbines from 1 to 4 induces a flux wave which is 1.12 m height at the Panemune Bridge and while travelling downstream it shrinks to 0.88 m at the Aleksotas Bridge (Table 1). The increase of operational turbines from 1 to 2 induces flux waves of much smaller height, e.g. on 2005. 07.11 in addition to one operational turbine another was started and then stopped after 2–3 hours and after an hour the start-stop process was repeated as shown in Fig. 5. In this case a double flux wave emerged and had a

height of approximately 0.56 m in the downstream reach of the HPP and 0.42–0.56 m at the Aleksotas Bridge (here the first number is the height of the first wave, the second – the biggest measured height of the wave). In both mentioned cases the heights of flux waves simulated using the hydrodynamic model at every measurement point using data about water level dynamics measured at the same points is presented in Table 1. Results show that flux waves on 2005.07.18 are approximately twice as high as those on 2005.07.11 although the number of operational HPP turbines and water discharges were increased 2 and 4 times, respectively. It is obvious that the change in water discharge and the wave height depend non-linearly on each other. This effect can be explained considering the shape of water discharge graphs of lowland rivers. According to these dependencies, when water level raises the ratio of the change in water level to the corresponding change in water discharge decreases, as in the above mentioned cases. Thus the higher water level is the smaller tidal wave is formed as a consequence of irregular operation of HPP turbines.

Measurement results show that during the flux wave rising phase the duration of it and the intensity of water level variation are greater in comparison with the flux wave falling phase. Both phases of the wave lengthen significantly in the direction of the flow and the wave height decreases (Fig. 5). This accumulative effect can be observed only in rather long

Table 1. Flux wave parameters calculated using the measured water levels

Position of measurement point and the river stretch No.	Height of wave, m		Water level rising intensity, m/h		Wave motion velocity, km/h	
	2005.07.11	2005.07.18	2005.07.11	2005.07.18	2005.07.11	2005.07.18
Below the Kaunas HPP	0.55–0.57	1.08	0.18–0.28	0.28–0.37		
Stretch I					8.6	5.30
Panemune Bridge	0.55–0.62	1.12	0.19	0.41		
Stretch II					7.6	7.02
Railway Bridge	0.42–0.50	0.91	0.15	0.36		
Stretch III					6.2	7.20
Aleksotas Bridge	0.42–0.56	0.88	0.15	0.34		
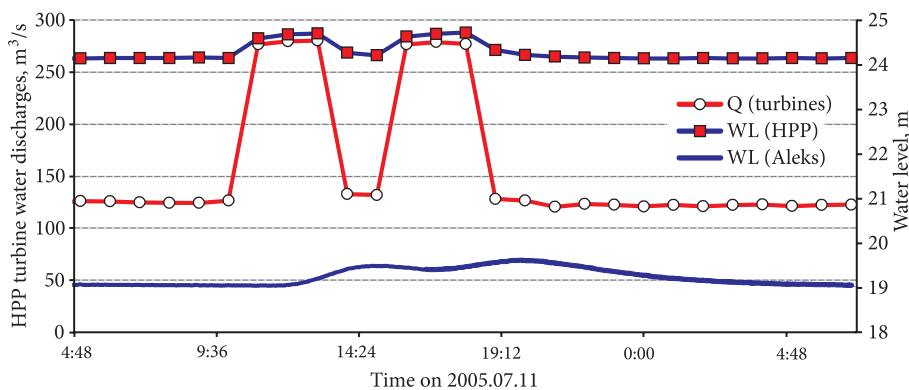
Fig. 5. Dynamics of the Kaunas HPP turbine discharges Q (HE), water levels in the downstream reach of the HPP WL (HPP), water levels at the Aleksotas Bridge WL (Aleks)

flow stretches and it depends on hydro-morphologic and hydraulic characteristics of the river.

It can be seen from the Table 1 that the water level rising intensity undoubtedly depends on the change in HPP turbine water discharges hence on the wave height (Fig. 6). It can be noticed that there exists a close relationship between the flux wave height and the water level rising intensity (coefficient of determination $R^2 = 0.93$). The assertion can be made that under normal irregular HPP turbine operation conditions the water level rising intensity (m/h) amounts to 36% of wave height (m).

Analysis of the Nemunas River water levels obtained on 2005.07.18 at the measurement points of the river showed that a flux wave travelling from the HPP reaches the Aleksotas Bridge after approx 2 hours and 30 minutes. It was established that the wave motion velocity weakly depends or does not depend at all on the change in turbine water discharges. For example, when HPP turbine water discharge increased from 127 to 277 m³/s on 2005.07.11, the flux wave reached the Aleksotas Bridge after 2 hours and 20 minutes (Fig. 7), therefore this result falls within the measurement accuracy limits.

Measurement results were used to calculate the average motion velocity of a flux wave between the Kaunas HPP and the Aleksotas Bridge. It was found to be equal to 5.7 and 6.1 km/h on 2005.07.18 and 2005.07.11 respectively. However, wave motion velocities calculated using the values of water discharges and water levels recorded at the HPP are not sufficiently accurate due to the reasons that were already mentioned. Wave motion velocity was calculated with a much greater accuracy in the second and third river stretches between the Panemune Bridge and Aleksotas Bridge where its values were in the range between 6.2 and 7.6 km/h (Table 1).

Summarizing the results of measurements of the Nemunas River water level dynamics it can be concluded that the results of such measurements can be used for an approximate establishment of the general regularities of unsteady water flow in the downstream reach of the HPP under normal operation conditions. However, they cannot be used to predict the outcome of extreme situations such as HPP accidents or spring floods. In these cases more elaborate methods of hydrodynamic investigation are required.
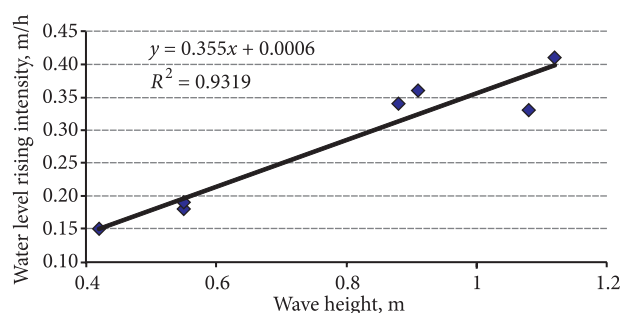
Fig. 6. Relationship between the height of the flux wave induced by the HPP and the water level rising intensity

4. Analysis of hydrodynamic modeling results

All bridge supports and the relatively small Jiesia River on the left bank of the Nemunas River were ignored in this DTM. Furthermore, several other inaccuracies were noticed at the Sanciai ramp and other navigation structures which do not affect the parameters of unsteady flow studied in this work significantly. The accuracy of the DTM was controlled using the flow widths estimated from river orthophotographs and hydrodynamic steady flow model. It was established that the errors of flow widths are acceptable – they do not exceed 5 m, i.e., the dimensions of the elementary grid point. It constitutes 3–5% of the flow width.

Roughness coefficients for different stretches of the river between measurement points were calculated during the calibration procedure of the hydrodynamic model. Their values ranged from 0.022 to 0.028, considering the guides and their recommended values for typical rivers without vegetation. According to our previous studies, roughness coefficients in the lower stretches of the Nemunas River were larger – 0.028 to 0.032. Flow velocities in the waterway of the river stretch under study do not exceed 1 m/s under one operational HPP turbine conditions. This value is close to actual flow velocity values measured under similar conditions in our previous works with an exception of a few short river stretches near the measurement point below the Kaunas HPP and other at the Panemune Bridge. It can be presumed that in the given hydrodynamic model smaller values of roughness coefficients and greater values of calculated flow velocities in the above mentioned river stretches are observed due to inaccuracies in the DTM of the river bed. Bathymetry inaccuracies revealed during this testing stage, in the authors' opinion, do not affect the modeling results of unsteady flow in comparatively short river stretches, however, in the future bathymetrical measurements of distinct river stretches should be carried out with greater accuracy. During the model calibrating procedure the authors modeled an unsteady flow with one operational HPP turbine until 2005.07.18, 7:00 hrs. Starting from this moment the authors calculated the 1st stage of unsteady water flow using the schedule of Monday HPP turbine operation. In this stage the water discharge hydrograph at the upper open boundary of the model was set using the actual HPP water discharges of that day. Calculations were performed with the hydrodynamic model for each time step and the values of unsteady flow velocities and water levels in the whole river reach area were obtained. These results were used in graphs representing the dynamics of water levels at the river measurement points (Fig. 1). Figs 7, 8 uses the following abbreviations: "(meas)" – measured water levels; "(sim)" – simulated water levels; "WL below HPP" – water level in the downstream reach of the HPP; "Panem. t." – water level at the Panemune Bridge; "R-way. b." – water level at the Railway Bridge; "Aleks. b." – water level at the Aleksotas Bridge; "Q(HPP)" – HPP water discharges.

In the graphs the water level dynamics simulated using the hydrodynamic model match the actual

measurement results during the flux wave rising phase at the measurement points, errors do not exceed 10 cm. However, the flux wave phase calculated in the model begins much earlier (1.05–2 hours) than it was actually obtained in the river at all measurement points, except at the Aleksotas Bridge where the occurrence of water level dynamics established at the lower open boundary using measurement results can be observed. After several successive trials of the model the authors managed to obtain the tidal wave water level results that matched the actual measurement data rather well (Fig. 7).

The operational period of all 4 HPP turbines was extended in the model, however, the values of HPP water discharges in the water discharge graph Q (mod) remained unchanged. The calculated water level dynamics fit the results of water level measurements during the wave rising phase rather well, errors are less than 10 cm. However, larger errors of the hydrodynamic model were still observed during the reflux: water level errors reached 30 cm and time

shift errors – up to 35 minutes. At the other measurement points the modeling results were rather accurate (Fig. 8).

More accurate actual graphs of water discharge dynamics would fit the purpose of model calibrating much better. However, in this work the modeling of unsteady flow phenomena with MIKE 21 programs was conducted for the first time and this method proved itself useful. It has many important advantages over other investigation methods and allows to establish and analyze various constantly changing hydraulic characteristics of water flow at any point of river reach area (within the boundaries of the grid) and at any moment of time, therefore it allows to better understand rather complex unsteady flow phenomena.

5. The local scour of Railway Bridge abutment

Operation of the Kaunas HPP turbines and increase in their water discharge cause a flux wave (Fig. 9) which after 1.5–2 hours reaches the Railway Bridge. From this
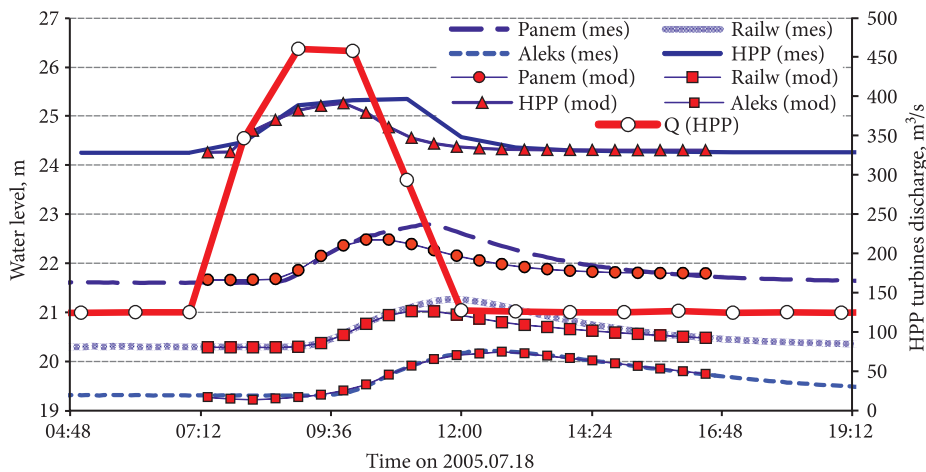

Fig. 7. Comparison of actual measurements of Kaunas HPP impact on the Nemunas river water levels and results obtained from the 1st stage of the model
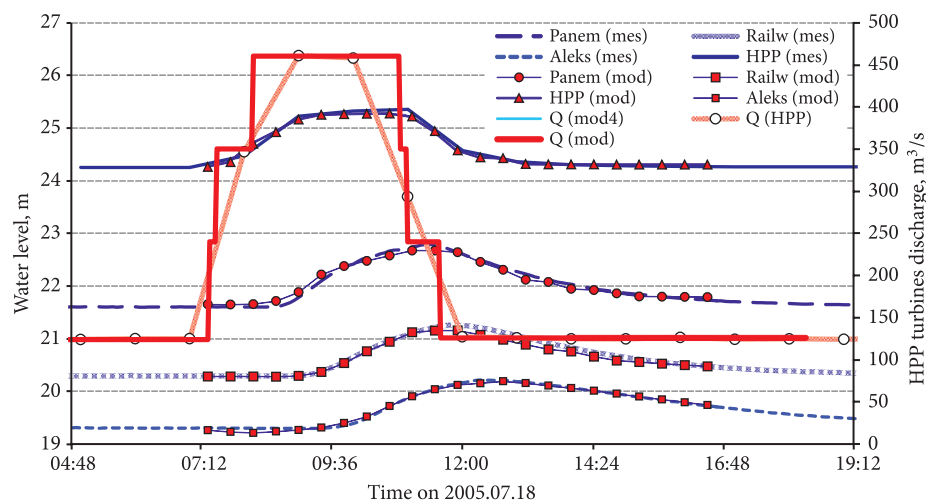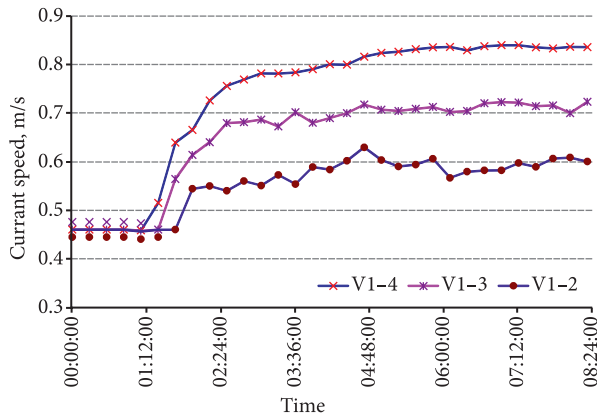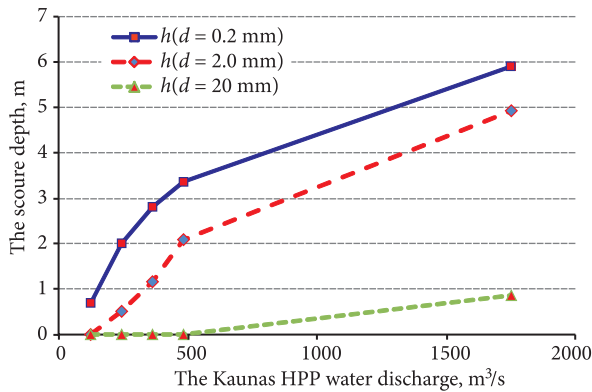

Fig. 8. Comparison of actual measurements of Kaunas HPP impact on the Nemunas River water levels and results obtained from the 2nd stage of the model

Fig. 9. Changes in the flow velocities V_0 at the second abutment of Railway Bridge due to the increase in the number of operating turbines from 1 to 2 (V1–2), from 1 to 3 (V1–3) and from 1 to 4 (V1–4)

Fig. 10. The depth and duration of the local scour at the second abutment of Railway Bridge before stabilization

moment the flow velocities start to rapidly increase and stabilize after 4–6 hours (Fig. 10).

At the same time the water levels have been correspondingly increasing (Fig. 9) as well as the flow depths before bridge abutment until reaching their maximum values (Table 2). To compare the changes in water levels (i.e., the heights of flux wave) measured at the Railway Bridge and reaching almost 1.0 m (Fig. 10) with the changes in flow depth at the bridge abutment H (2.88–1.02 = 1.86 m) determined in the model, it is obvious that the heights of flux wave in both cases differ. Hydrodynamic modeling

results showed that this difference is related to the duration of HPP turbines operation: if all the 4 turbines operated non-stop not for 2 hours but for more than 6–7 hours, the height of flux wave, illustrated in Fig. 9, would be significantly larger and would reach the height of 1.8 m.

The abutment scour duration was estimated from the initial scour depth $h = 0.69$ m which stabilizes at the minimum river water discharge, i.e., under the operation of 1 Kaunas HPP turbine for a sufficiently long time.

Relationship between the maximum depth of abutment scour (h), the steady Nemunas River flow discharge (Q) and the average diameter of bottom soil particles (d) is given in Fig. 10. The graph shows that under the operation of all HPP turbines for a sufficiently long time no bridge scour would occur if the average diameter of the bottom soil particles was > 20 mm. Even when the maximum water discharges of spring floods are passed by the HPP dam, the abutment scour in this case would not reach 1 m. However, when soils at the bridge abutment are sensitive to scouring the depth of scour under the extreme conditions of spring floods may cause danger to the stability of bridge abutments.

Time factor is not of less importance to the bridge abutment scour. The mean multi-year water discharge in the Nemunas River (260 m^3/s) is about two times less than the total discharge of all 4 turbines of the Kaunas HPP, therefore the number of the most frequently operating turbines is increased only during the "peak" electricity consumption hours, and under conditions of dry period this lasts for several hours per day only (Fig. 2).

Due to irregular operation of the Kaunas HPP turbines, the depth of bridge abutment scour has been continuously changing: with the increasing water discharge, the flow velocities and water level the bridge abutment is scoured out, and with the decreasing water discharge and velocities the scour hole is once again sanded-up. The largest scour depths could be achieved in case of the continuous operation of 4 HPP turbines for a longer period than 3 days. To determine the time factor, based on formulas (4) and (5) the dependence of the bridge abutment scour depth on time was estimated and represented graphically when during the operation of 1 turbine all the 4 turbines are switched on at the same time (Fig. 11).

Comparison of the measurement results of water levels (Fig. 2), durations (Table 2) of the maximum bridge

Table 2. Estimation results of the maximum depth and duration of bridge abutment scour ($d = 0.2$ mm)

Number of operating HPP turbines and flood probability	Water discharge of the Kaunas HPP Q, m^3/s	Flow depth H, m	Non-scouring flow velocity V_0, m/s	Flow velocity V, m/s	Scour depth after stabilization h, m	Scour duration τ, h
1	120	1.02	0.44	0.46	0.69	–
2	240	1.84	0.45	0.61	2.01	162.4
3	360	2.4	0.46	0.72	2.81	109.7
4	480	2.88	0.48	0.84	3.36	72.1
10%	1752	6.74	0.51	1.14	5.90	27.3

abutment scour (before stabilization) and dependence of scour depth on time (Fig. 11) showed that the depths of bridge abutment scour usually do not reach their maximum values estimated in Table 2. A more threatening situation from this point of view can occur under flooding conditions. When water discharges, velocities and depths of the flow increase several times, the flood lasts for several days then the processes of local scour of the riverbed become very intensive and the abutment scour can reach maximum values (Fig. 10).

The scour of one bridge abutment under the conditions of unsteady water flow studied in this paper in a simplified way does not reflect a variety of this complicated process and all its influencing factors, the obtained modeling and estimation results only illustrate the main scour regularities and the related threats. For example, in summer 1987 the measured depth of the local scour of the second abutment of the Railway Bridge was about 2.5 m, thus, it is probable that during floods the scour can be significantly larger. When monitoring and assessing technical condition of bridge abutments it is essential for the bridge maintenance specialists to take into consideration the foundation structure of each abutment, variety of riverbed soils, riverbed strengthening measures, self-formation phenomena, ice-drift capacity and other circumstances. This requires special systematic investigations of bridge abutment scour.

6. Conclusions

1. Water flow in the Nemunas River near Kaunas is unsteady in most cases (70–80% of measurement time) due to irregular operation of the Kaunas hydropower plants (HPP).

2. A flux wave induced by the start of HPP turbines rises in the downstream reach of HPP and travels at the speed of 6–9 km/h in the direction of river flow and reaches the Aleksotas Bridge after 2.5 hours.

3. Flux wave height depends on wave flow velocities and water level in the river: the higher the water level the smaller the flux wave.

4. Under normal HPP operation conditions the height of a flux wave at the Aleksotas Bridge does not exceed 1.2–1.5 m and the water level rising rate – 0.45 m/h.

5. Due to irregular operation of the Kaunas HPP turbines the depths of bridge abutment scour has been continuously changing: under the maximum water discharge they increase, the water discharge having decreased the bridge abutments are once again covered with bottom sediments.

6. A threat of bridge abutment scour is the smallest when Kaunas HPP operates with the minimum water discharge or when the increase in HPP power and water discharge is short-term. Due to irregular operation of the Kaunas HPP turbines and a changing water discharge the local bridge abutment scour is several times smaller than the scour caused by extreme spring floods. The largest and the most dangerous bridge abutment scour is expected under conditions of extreme spring floods or in case of dam collapse when the Kaunas HPP dam passes maximum water discharges for more than 24 hours.
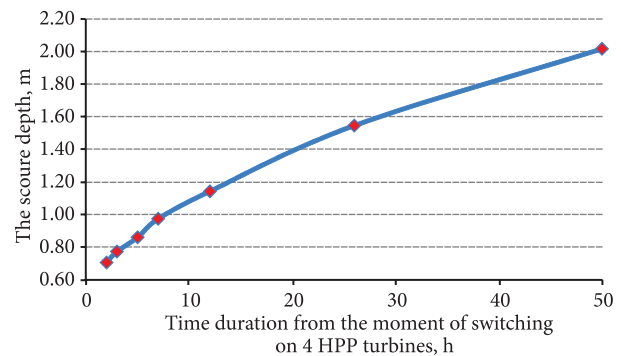

Fig. 11. Dynamics of the bridge abutment scour depth in Kaunas HPP after switching on 4 turbines ($d = 0.2$ mm)

7. A technical condition of bridge abutments should be controlled systematically and after each larger flood.

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