doi: 10.17221/105/2015-SWR

Hindcast of Wind Driven Wave Heights in Water Reservoirs

Petr PELIKÁN and Ladislav KOUTNÝ

Faculty of Forestry and Wood Technology, Mendel University in Brno, Brno, Czech Republic

Abstract

Pelikán P., Koutný L.: (2016): Hindcast of wind driven wave heights in water reservoirs. Soil & Water Res., 11: 205–211.

The paper is focused on the problems of water level motion in water reservoirs. Dimensions of wind driven waves are closely related to the parameters of occurring wind. Due to the complexity of the physical phenomena, most methods for wave prediction are based on semi-empirical relations. The theories for approximation of waves follow two approaches. The first one, called regular waves, is based on mathematical description of water surface. The second one, called irregular waves, results from statistical processing of collected data. The methods have been modified as wind and wave data were accumulated over time, resulting in better predictions. The aim of the present research consists in verification of two selected irregular wave models for characteristic wave height estimation – the first one widely used by U.S. Army Corps of Engineers (USACE) for sea and large inland water bodies conditions and the second one related to the Czech standard specification CSN 75 0255 Calculation of wave effects on hydraulic structures. Characteristic wave height represents one of the most important wave parameters as an input for consequent computational tasks dealing with hydrodynamic events occurring on the point of interaction between water level and shore (wave breaking, wave setup, wave run-up on structures and banks, etc.). Further, the paper discusses relevant statistical techniques for proper exploration of special data of wave motion gained from *in situ* measurements.

Keywords: characteristic wave height; irregular waves; wave train analysis

The primary research of wind-water interactions and wave mechanics had been accomplished in coastal areas along the shores of world oceans and seas because a basic understanding of coastal meteorology is an important component in coastal and offshore design and planning. Consequently, the similar principles of water wave mechanics started to be considered in conditions of inland water bodies.

The problem of wind driven wave heights was investigated worldwide by many specialists as published by Miles (1957), Phillips (1957),Hsu (1988) in the sea conditions and Lukáč (1972), Lukáč and Abaffy (1980), Resio and Vincent (1977), Kratochvil (1970), Šlezingr (2004, 2007), Ozeren and Wren (2009) in the conditions of water reservoirs.

In the Czech Republic, the wave parameters and characteristics are usually considered during the design of dikes and stabilization measures around the water reservoir backwater zone according to the valid Czech standard specification CSN 75 0255:1987 Calculation of wave effects on hydraulic structures. The motivation of the submitted research lies in the fact that investigation of wave parameters in deep water conditions in Czech reservoirs remains out of scope today, the experimental *in situ* measurements are rare and state of the art calculation models are only adopted from abroad.

The simplest method describing water surface is called the first order theory (linear wave theory) – the original regular wave theory. The wave motion is represented by sinusoidal advancing wave (simple linear wave). Sinusoidal character means that the wave is steadily repeated in the form of constant smooth shape. The crests of particular waves have the same height, constant celerity, and they collaterally proceed in the same mutual distance in perpendicular direction to the wave front without change of their shape. The theory was presented by English mathematician George Biddell Airy in 1845. The theory is simple and it is possible to relatively exactly determine the number of wave

doi: 10.17221/105/2015-SWR

characteristics with its aid. However the higher order theories were developed (Chen *et al.* 2014). They have been frequently used for modelling waves along sea coastlines (Hsieh *et al.* 2015; Janno & Šeletski 2015).

The energy transferred to the water surface by wind generates a range of wave heights and periods that increase as the waves travel across the available fetch length (well describable by irregular wave theory). The process of wave generation by wind can be explained by combining the resonance model developed by Phillips (1957) and the shear flow model developed by Miles (1957) (see details in CERC 1973–1984, USACE 2002–2011). Today, the irregular wave approach leads to derivation of wave characteristics of wave spectra – spectral analysis (Kumar *et al.* 2014).

Due to the complexity of the physical phenomena, most methods for wave prediction are based on semi-empirical relationships. The methods have been modified as wind and wave data were accumulated over time, resulting in better predictions.

MATERIAL AND METHODS

Actual observed waves do not look as simple as the sinusoidal profile. With their irregular shapes, they appear as a confused and constantly changing water surface, since waves are continually being overtaken and crossed by others. As a result, waves are often short-crested. This is particularly true for waves growing under the influence of the wind.

The irregular wave theory was used for purposes of the research. An irregular wave train is constructed by linear superposition of a number of linear wave components. Wave train analysis is based on statistical processing of measured data. The data are represented by records of water level motion in a given point. The individual waves are identified by local maximums (wave crest) and local minimums (wave toe) of water level fluctuation.

Table 1. Statistical parameters of irregular waves (height)

Parameter	Description
$\bar{H}$	average wave height
H_{max}	max. wave height in a record
$\bar{H}_{1/3}$	significant wave height

A measured wave record never repeats itself exactly, due to the random appearance of the sea surface. But if the sea state is "stationary", the statistical properties of the distribution of periods and heights will be similar from one record to another. The most appropriate parameters to describe the sea state from a measured wave record are therefore statistical (WMO 1998). The selected main important parameters related to the wave height are presented in Table 1.

The theory handles with a concept of significant (characteristic) wave height ($\bar{H}_{1/3}$). That is the mean height of one third of the highest waves in wave spectra, i.e. the wave with the height coming up to the 13% probability of occurrence (Figure 1). The term is applied worldwide in wave estimation and related calculations (also in CSN 75 0255:1987).

The two approaches in estimation of characteristic wave height in deep water conditions (H_0) were tested with the aid of measured data. Both are constructed on the assumption that the wind with constant speed and direction blowing over given fetch causes the waves of certain parameters which are limited by fetch length (i.e. fetch-limited conditions).

The first one is used by U.S Army Corps of Engineers (USACE) for wave estimation in the conditions

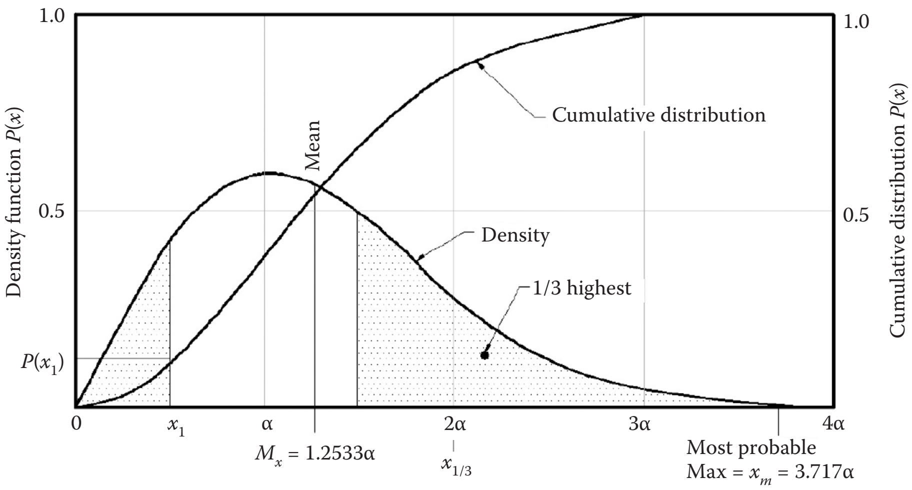


Figure 1. Characteristic wave height derived from Rayleigh distribution (USACE 2002)

doi: 10.17221/105/2015-SWR

of seas and large inland water bodies (Great Lakes of North America). The fetch length is expressed directly, conversely the wind speed is expressed indirectly via friction velocity with consideration of the drag coefficient (roughness of water level).

$$H_0 = \frac{u_*^2}{g} 0.0413 \left(\frac{gF}{u_*^2}\right)^{1/2} \quad (1)$$

where:
H_0 – characteristic wave height in deep water (m)
u_* – friction velocity (m/s)
g – gravitational constant (m/s^2)
F – fetch length (m)

The second calculation method is used in the conditions of water reservoirs in the Czech Republic and Slovakia due to the date of release of CSN 75 0255 Calculation of wave effects on hydraulic structures (valid since 1987). In the main equation, the expressions of direct wind speed and indirect fetch length are used. The fetch is derived from the several radials led under given angles through examined point to the opposite shoreline of reservoir.

$$H_0 = 0.0026 \frac{u_{10}^{1.06} F_{ef}^{0.47}}{g^{0.53}} \quad (2)$$

where:
u_{10} – wind speed in 10 m reference level above water level (m/s)
F_{ef} – effective fetch length (m)

Other used magnitudes are of the same meaning. A detailed calculation process is possible to explore in CSN 75 0255 (1987), USACE (2002–2011), Šlezingr (2004), Pelikán (2013), Pelikán *et al.* (2014).

Data collection was accomplished in fetch limited deep water conditions on water reservoir Nové Mlýny – Dolní (Dolní Věstonice, harbour). The term fetch limited conditions means the wind blows for sufficient time to develop the waving along the whole fetch from the opposite site of reservoir to the investigated point. The linear waves are characterized by their length, height, period and actual water depth. The calm water column is disturbed by wave advance, so the water level also in deeper parts of the water column gets into the motion. Water particles describe vertical circles which exponentially decrease with increasing depth (Gerstner's waves). If the actual water depth is less than one half of wave length, the influence of the bottom on the motion of water particles is negligible. The wave is not deformed and we consider it occurs in the deep water. We can distinguish three types of waves pursuant to the relative depth: short waves (deep water), transitional waves (transitional water), and long waves (shallow water).

The collected data are represented by four short wave train records with simultaneous measurement of wind speed and direction. The recording was realized with the aid of levelling staff and camera.

RESULTS

A wave train analysis determines wave properties by finding statistical quantities (i.e. heights in the research) of the individual wave components present in the wave record. The wave train analysis was essentially a manual process of identifying the heights of the individual wave components followed by a simple counting of wave crests in the wave record. The process began by dissecting the entire record into a series of subsets for which individual wave heights were then noted. The results of particular records of water level motion were transferred into the spreadsheet processor – capturing of wave crests and toes representing the individual waves. In the interest of reducing manual effort, wave height was defined as the vertical distance between the highest and lowest points (local maxima and minima). The result of the process is represented by data sets of wave heights in wave trains (4 measurements, 1200 waves identified totally). In addition, the values read on the levelling staff were fitted to the real elevation above sea level (geodetical survey of experimental site). The example of wave train is depicted in Figure 2.

The obtained datasets of wave heights were statistically processed with emphasis on exploratory data analysis (EDA). The normality of data was investigated at first (software QCExpert 3.3). Three types of tests were used because of sufficient verification of the result – moment test, D'Agostino, and Kolmogorov–Smirnov. The null hypothesis states the data follow normal distribution and it was rejected by all types of tests in all cases (Table 2), *P*-value is smaller than significance level of 5%. The first results indicated the data do not follow normal distribution, however at the moment we are not able to estimate which distribution data follow.

Hence, the data were analyzed through subsequent graphical statistical methods (STATISTICA Version 12, EasyFit). The Box-Whisker plots better showed the skewness of data distribution and quality. Histograms revealed the right skewed data distribution of all samples (Figure 3).

doi: 10.17221/105/2015-SWR

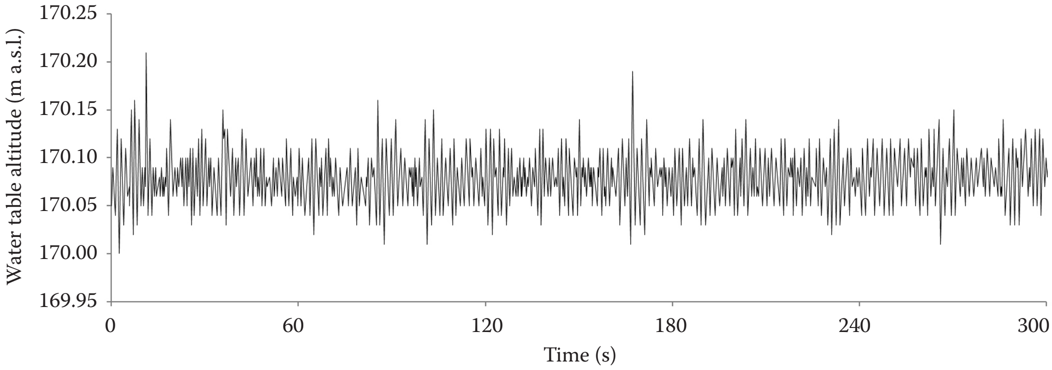


Figure 2. Record of water level motion (measurement No. 1)

Table 2. Test of normality (QCExpert)

Label of measurement		1	2	3	4
Number of observations (waves)		303	301	299	297
Moment	*P*-value	0.00296	0.01289	0.01592	0.02317
	normality	rejected	rejected	rejected	rejected
D'Agostino (N ≥ 100)	*P*-value	2.55E-06	1.38E-03	2.38E-03	3.42E-03
	normality	rejected	rejected	rejected	rejected
Kolmogorov–Smirnov	*P*-value	0.00019	0.00007	0.00001	0.00003
	normality	rejected	rejected	rejected	rejected

The statistical model (distribution) best corresponding with data was explored with the aid of software EasyFit. The program can evaluate the goodness of fit (GOF) of more than 60 distributions through Kolmogorov–Smirnov, Anderson–Darling, and Chi-Squared tests. The preview of distribution fitting is depicted in Table 3 on the example of selected continuous distributions (Kolmogorov–Smirnov test). The GOF tests measure the compatibility of a random sample with a theoretical probability distribution function. In other words, these tests show how well the distribution fits to our data. GOF tests measure the distance between the data and the tested distribution, and compare that distance to some threshold value. If the distance (called the test statistic) is less than the threshold value (the critical value), the fit is considered good. The principle of applying various GOF tests is the same, however, they differ in how the test statistics and critical values are calculated. The test statistics are usually defined as some function of sample data and the theoretical (fitted) cumulative distribution function. The critical values depend on the sample size and the significance level chosen. The significance level is the probability of rejecting

Table 3. Distribution fitting (EasyFit)

Measurement					
1			2		
Distribution	statistics	rank	distribution	statistics	rank
Rayleigh (2P)	0.080	1	Rayleigh (2P)	0.105	1
Rayleigh	0.081	2	Weibull	0.106	2
Weibull	0.086	3	Rayleigh	0.111	3
Gamma	0.104	4	Normal	0.117	4
Normal	0.109	5	Gamma	0.128	5
Logistic	0.130	6	Logistic	0.132	6
Lognormal	0.145	7	Lognormal	0.171	7
3			4		
Rayleigh (2P)	0.091	1	Weibull	0.087	1
Rayleigh	0.092	2	Rayleigh (2P)	0.089	2
Weibull	0.101	3	Rayleigh	0.090	3
Gamma	0.116	4	Gamma	0.115	4
Normal	0.127	5	Normal	0.123	5
Logistic	0.144	6	Logistic	0.140	6
Lognormal	0.166	7	Lognormal	0.145	7

doi: 10.17221/105/2015-SWR

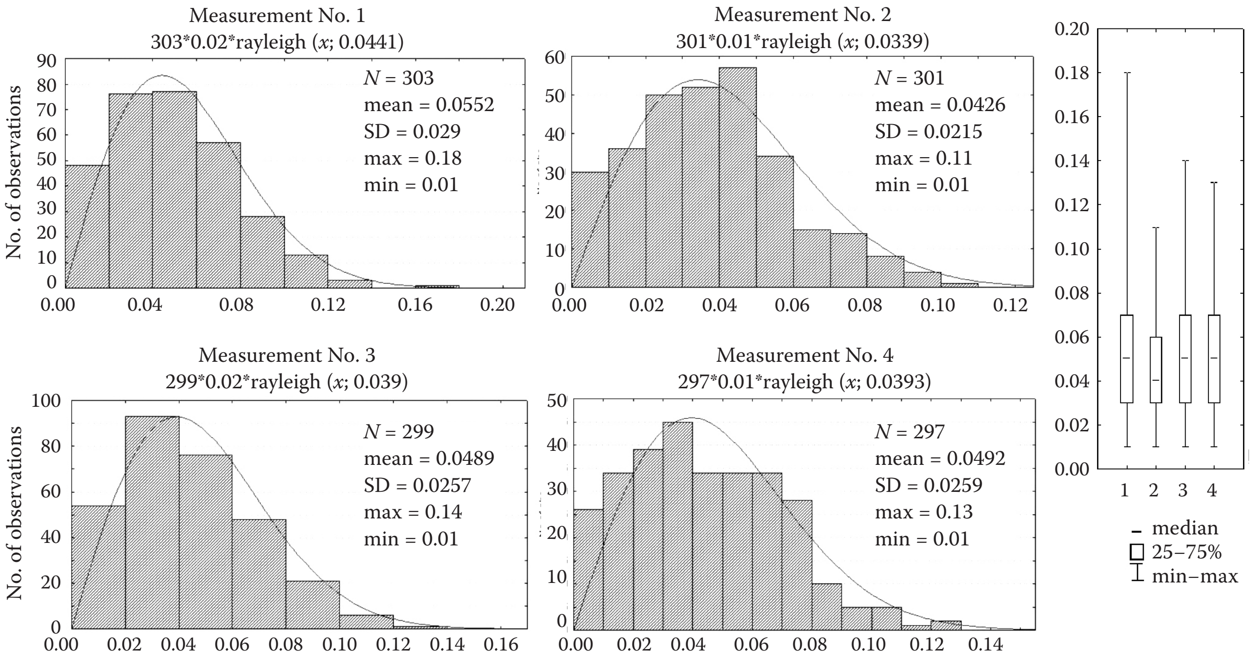


Figure 3. Graphical exploratory data analysis and distribution fitting (STATISTICA software)

a fitted distribution (as if it was a bad fit) when it is actually a good fit. The significance level was considered as the value of 0.05. The results confirmed the presumption of suitability of Rayleigh probability distribution (Figure 3), although Weibull distribution got high rank as well (measurement No. 4). Classical estimates of parameters were not assumed due the asymmetric character of data. Conversely the robust estimates were considered, especially quantiles related to the characteristic wave height probability of occurrence 13%.

The input data for the computational model of characteristic wave height are represented by measured average wind speed and direction. However the data were collected during the same experimental measurement, the input data of wind and fetch may differ in models (Table 4). The fact is caused by the different methodology of wind and fetch determination in the models that was fully respected (detailed practice available in Pelikán 2013). The different methodology to define the wind speed for the purposes of further calculations may cause variant values in the reference level 10 m above water level. The considered wind speeds reached the values of 3.0–3.6 m/s for model of USACE, 2.6–3.6 m/s for model of CSN. The fetch lengths were derived from wind directions and the same problem induced the difference in assumed fetch length: 2669 m (USACE) and 2739 m (CSN). The modelled characteristic wave heights with above-mentioned inputs gained values of 7.1–8.6 cm (USACE), or 8.9–12.4 cm (Czech model) (Table 4).

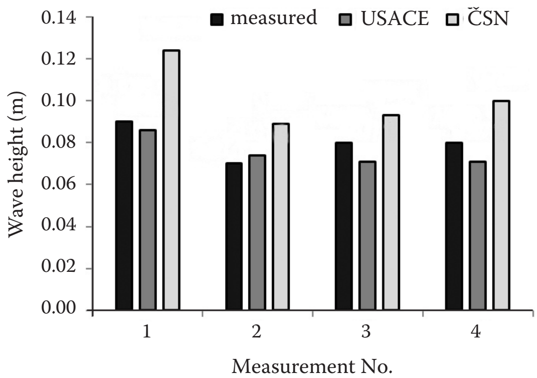
The comparison of measured and modelled values is demonstrated in Figure 4. The graph shows the differences between characteristic wave heights in real units. The right graph illustrates the deviations

Table 4. Modelled values of characteristic wave heights

Values	Magnitude			
	1	2	3	4
USACE				
u_{10} (m/s)	3.6	3.1	3.0	3.0
u_* (m/s)	0.126	0.108	0.104	0.104
F (m)	2699	2699	2699	2699
H_0 (m)	0.086	0.074	0.071	0.071
CSN				
u_{10} (m/s)	3.6	2.6	2.8	3.0
F_{ef} (m)	2739	2739	2739	2739
H_0 (m)	0.124	0.089	0.093	0.100

u_{10} – wind speed in 10 m reference level above water level (m/s); u_* – friction velocity (m/s); F – fetch length (m); H_0 – characteristic wave height in deep water (m); F_{ef} – effective fetch length (m)

doi: 10.17221/105/2015-SWR



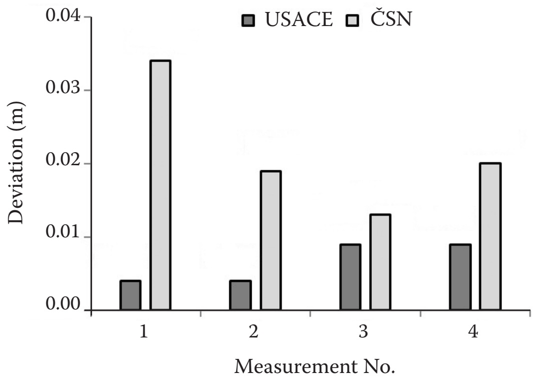


Figure 4. Characteristic wave height H_0 – measured and modelled values

from measured data. The overall results can help evaluate the exactness of the investigated models.

DISCUSSION

Although it may seem the model of USACE provides better estimates of wave height in all cases, the deviations from measured data are practically negligible in connection with water reservoir extent. The Czech model provides probably overestimated results.

The application of wave models is necessary for the solution of coastal engineering studies and long-term prognostic tasks. The characteristic wave height represents the main input for consequent calculations – for example wave breaking, wave setup and run-up on the structures and reservoir banks. The knowledge of this wave parameter allows the determination of the active part of shore due to the waving, proper altitudinal emplacement of shore-stabilization constructions, and altitudinal dimensions of dikes due to the water level.

The important decision lies in the formulation of the protection degree of shoreline – setting up of the design wave height and design wind speed, respectively. The dams and levees are protected from the waves with 1% probability of occurrence and the backwater zone is usually protected from characteristic wave with 13% probability of occurrence due to high expenses spent on stabilization measures (ČSN 75 0255:1987). Any degree of protection could be determined thanks to known statistical distribution (and estimation of statistical parameters) with the aid of the presented models.

Knowledge of the characteristic wave height is essential for the design of coastal projects because it is the major factor that determines the geometry of beaches, the planning and design shore protection measures, and hydraulic structures. Estimates of wave conditions form the basis of almost all coastal engineering studies.

The presented relationships are well applicable in combination with regular wave theory (Airy waves) leading to accurate estimates of wave parameters and characteristics on open water areas and inland water bodies.

CONCLUSIONS

The verification of models with real data is a quite complex procedure involving data collection and statistical processing with the emphasis on exploratory data analysis due to the special attributes (skewness, uncommon distribution). The overall results indicate the model used by USACE is applicable for wave estimations in inland water bodies as well however originally developed for sea conditions.

The characteristic wave height in deep water conditions represents basic input into the subsequent computations of hydrodynamic events occurring on the point of interaction between water level and shore, i.e. solution of engineering tasks generally in the field of coastline water management in transitional and shallow water conditions. For example, it is possible to compute proper altitude emplacement of particular design components of bank stabilization measures. The knowledge of probability distributions of some design parameters allows us to compute waves with any level of probable exceeding. Thus, the models could be useful for calculations when designing the dam crest and its fortification where the probability of wave run-up occurrence is considered 1%.

The gains of the research consist in verifying foreign state of the art computational methods and implementing new piece of knowledge into conditions of water reservoirs in the Czech Republic.

doi: 10.17221/105/2015-SWR

Acknowledgements. The paper was worked out as a part of research project "Active anti-abrasion structures", reg. No. LDF_VT_2015011, funded by IGA FFWT MENDELU Brno.

References

CERC (1973, 1977, 1984): Shore Protection Manual. Washington, D.C., Waterways Experiment Station, USACE.

Chen G.-Y, Wu C.-L., Wang Y.-H. (2014): Interface depth used in a two-layer model of nonlinear internal waves. Journal of Oceanography, 70: 329–342.

Hsieh C.-M., Hwang R. R.,Hsu J. R.-C., Cheng M.-H. (2015): Numerical modeling of flow evolution for an internal solitary wave propagating over a submerged ridge. Wave Motion, 55: 48–72.

Hsu S.A. (1988): Coastal Meteorology. New York, Academic Press.

Janno J., Šeletski A. (2015): Reconstruction of coefficients of higher order nonlinear wave equations by measuring solitary waves. Wave Motion, 52: 15–25.

Kratochvíl S. (1970): Determination of parameters of wind waves of gravitational waves in deep dam reservoirs and lakes. Vodní hospodářství, No. 10: 265–270. (in Czech)

Kumar V.S., Dubhashi K.K., Nair T.B.M. (2014): Spectral wave characteristics of Gangavaram, Bay of Bengal. Journal of Oceanography, 70: 307–321.

Lukáč M. (1972): Waves on the Water Reservoir and its Effects on the Reservoir Bank. Bratislava, Katedra Geotechniky SVŠT. (in Slovak)

Lukáč M., Abaffy D. (1980): Waves on the Water Reservoirs, its Effects and Anti-abrasive Measures. Bratislava, Ministerstvo lesného a vodného hospodárstva SSR. (in Slovak)

Miles J.W. (1957): On the generation of surface waves by shear flows. Journal of Fluid Mechanics, 3: 185–204.

Ozeren Y., Wren D.G. (2009): Predicting wind-driven waves in small reservoirs. American Society of Agricultural and Biological Engineers, 52: 1213–1221.

Pelikán P. (2013): The effect of shore abrasion on the transformation of the bank of water reservoirs. [Ph.D. Thesis.] Brno, Mendel University in Brno, Faculty of Forestry and Wood Technology. (in Czech)

Pelikán P., Šlezingr M., Marková J., Pilařová P. (2014): Wind parameters related to the wind driven waves on water reservoirs. In: 4th Int. Conf. INFRAEKO 2014, Krakow, May 29–30, 2014, Vol. 1: 235–241.

Phillips O. M. (1957): On the generation of waves by a turbulent wind. Journal of Fluid Mechanics, 2: 417–445.

Resio D.T., Vincent C.L. (1977): Estimation of winds over the Great Lakes. Journal of the Waterways, Harbors, and Coastal Engineering Division, 103: 265–283.

Šlezingr M. (2004): Bank Abrasion. Brno, CERM. (in Czech)

Šlezingr M. (2007): Stabilisation of reservoir banks using an "armoured earth structure". Journal of Hydrology and Hydromechanics, 55: 64–69.

USACE (2002–2011): Coastal Engineering Manual. Engineer Manual 1110-2-1100, Vol. I–VI, Washington, D.C., USACE.

WMO (1998): Guide to Wave Analysis and Forecasting. 2nd Ed. WMO Paper No. 702, Geneva, WMO.

Received for publication June 1, 2015
Accepted after corrections October 16, 2015
Published online May 18, 2016

Corresponding author:

Ing. Petr Pelikán, Ph.D., Mendelova univerzita v Brně, Lesnická a dřevařská fakulta, Zemědělská 3, 613 00 Brno, Česká republika; e-mail: pelikanp@seznam.cz