# EFFECT OF THE TURBINE CLOSING TIME ON THE WATER FLOW IN THE SUPPLY SYSTEM OF A DIVERSION HYDROPOWER PLANT WITH A SURGE TANK

# EFEKT VREMENA ZATVARANJA TURBINA NA TOK VODE U DOVODNOM SISTEMU DIVERZIONE HIDROELEKTRANE S VODOSTANOM

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*Preliminary Notes*

**SUMMARY**
*In this paper, a model based on numerical solution of two ordinary differential equations is used to obtain the water level in the surge tank and the static pressure in the headrace tunnel – the properties of essential importance for the functioning of the water supply system during the turbine shut-off. The model allows a fast and reliable simulation of the hydraulic processes in the headrace tunnel and the surge tank. It was validated by comparing the numerical results with the data available from the experiments conducted under real conditions in a surge tank of the HPP Jablanica. This model is used to analyze of influence of different parameters on variations of water level oscillation in the surge tank and the static pressure in the headrace tunnel.*

*Preliminarni rad*

**SUMMARY**
*U ovom radu je korišten model baziran na numeričkom rješavanju sistema dvije diferencijalne jednačine da bi se dobio nivo vode u vodostanu i pritisak u dovodnom tunelu – osobine od krucijalne važnosti za funkcionisanje dovodnog sistema vode za vrijeme ispada agregata. Metod omogućava brzu i pouzdanu simulaciju hidrauličkih procesa u dovodnom tunelu i vodostanu. Validacija je izvršena upoređujući rezultate numeričkog proračuna sa dostupnim rezultatima eksperimenta izvedenim u realnim uslovima na HE Jablanica. Model je korišten za analizu uticaja različitih parametara an varijacije oscilacija nivoa vode u vodostanu i pritiska u dovodnom tunelu.*

## 1. INTRODUCTION

Surge tanks are auxiliary water storage units constructed in hydropower plants between the main reservoir and the power house [1, 2]. Surge tanks play a very important role in diversion hydropower plants. They protect water supply components from negative impact of water hammer, neutralizing strong pressure rises and drops appearing in penstock during the turbine start or closure. Appropriate shapes and dimensions of surge tanks are decisive for their proper operation, so that a reasonable attention is paid to these during design of a new power plant or in refurbishment of existing power plants.

Beside the surge tank operation, the turbine opening or closing time may also influence the flow development in the water supply system, where the headrace tunnel and the penstock are of primary interest. The opening/closure time cannot be varied arbitrarily, since it is related to the operation of a number of other mechanical and electrical components in the power plant, and it is an integrated element of the plant automation, so that it has to be carefully chosen in order to provide proper functioning of the power plant.

In this paper, a model based on numerical solution of two ordinary differential equations [3] is used to obtain the water level in the surge tank and the static pressure in the headrace tunnel – the properties of essential importance for the functioning of the water supply system during the turbine shut-off. The method allows a fast and reliable simulation of the hydraulic processes in the headrace tunnel and the surge tank. It was validated by comparing the

numerical results with the data available from the experiments conducted under real conditions in a surge tank of the hydropower plant Jablanica, Bosnia-Herzegovina. Consequently, here it is used to analyze the effect of turbine closure time onto the pressure development in the headrace tunnel in the case of a generic cylindrical surge tank.

## 2. METHOD

A typical diversion hydropower plant system with the reservoir, the intake tunnel, the surge tank with orifice, and the penstock is shown in Figure 1. The momentum equation for the water flow in the headrace tunnel of this system can be written in the form [4,7]:

$$\frac{L}{g}\frac{dQ_t}{dt} = A_t(-z - h_f - h_j) \qquad (1)$$

where $L$ is the length of the headrace tunnel, $g$ is the gravitational acceleration, $Q_t$ is the water flow rate in the tunnel, $t$ is the time, $A_t$ is the cross-sectional area of the tunnel, $z$ is the vertical position of the water free surface in the surge tank measured from the free surface level in the reservoir, $h_f$ is the head loss caused by friction in the tunnel, and $h_j$ is the minor head loss in the junction of the tunnel and the surge tank.
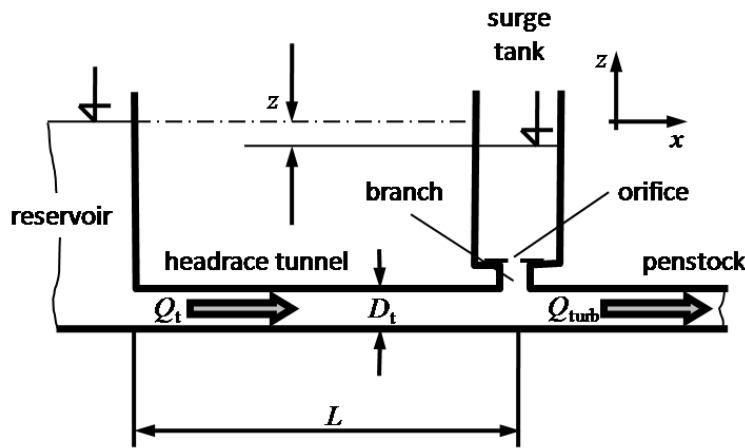


**Figure 1.** *A typical water supply system with a surge tank in a diversion hydropower plant.*

The friction loss in the tunnel is calculated using Darcy-Weisbach equation:

$$h_f = \lambda \frac{L}{D_t}\frac{v_t^2}{2g} \qquad (2)$$

where $\lambda$ is the friction coefficient, $v_t$ is the flow velocity in the tunnel (proportional to the flow rate $Q_t$), and $D_t$ is the tunnel diameter.

The minor head loss in the junction is given by the formula:

$$h_j = \xi_j \frac{v_t^2}{2g} \qquad (3)$$

where $\xi_j$ is the minor loss coefficient in the junction (like a T-branch).

The continuity equation for the surge tank reads:

$$\frac{dz}{dt} = \frac{1}{A_v}(Q_t - Q_{turb}) \qquad (4)$$

where $A_v$ is the area of the water free surface in the surge tank, and $Q_{turb}$ is the water flow rate through the penstock toward the turbines. All quantities on the right-hand side of eq. (4) are regarded as functions of time $t$. Obviously, eq. (4) is coupled with eq. (1) through the quantities $Q_t$ and $z$.

In the algorithm employed here, which is described in the previous publication [3], the geometric data ($L$, $A_t$, $A_v$) are assumed to be known, as well as the flow rate toward the turbines $Q_{turb}$. The considered time interval is divided into a number of finite time steps. The mutually coupled eqs. (1) and (4) are solved at each time step, starting from the initial one and proceeding the calculation to the next step until the complete time interval is processed. In doing so, the time derivatives are replaced by finite differences (implicit Euler method). First, eq. (1) is solved for $Q_t$, then eq. (4) is solved for $z$, delivering a temporary value of the water level in the surge tank. In order to resolve the inter-equation coupling, the process is repeated

several times within each time step until the convergence is reached. If the surge tank has a variable cross section (such as in the case of tanks with side chambers), the free surface area $A_v$ depends on its current displacement $z$ and has to be recalculated based on the known tank geometry and assuming that the water free surface is flat and horizontal during the entire process.

The described method depends considerably on reliable assessment of the friction and the minor loss. While the former is relatively easy to estimate (e.g. using Colebrook equation), the latter depends on topology and geometric features, as well as on flow conditions of the specific case, and can be found in various engineering tables and diagrams or using empirical formulae, such as those given in [5]. An approximate value of the minor loss coefficient $\xi_j$ of 20, defined with respect to the area-averaged water velocity in the tunnel $v_t$ as indicated by eq. (3), is adopted in this work based on the computational fluid dynamics (CFD) simulations done in [3] for both flow directions (flow from the tunnel to the surge tank, and reverse flow from the tank to the tunnel).

## 3. RESULTS AND DISCUSSION

### 3.1 Validation case
The results described here show the calculations done for the test case under real conditions in HPP Jablanica [8]. The diameter of the headrace tunnel is 6,3 m, its length is 1950 m, and the diameter of the surge shaft is 13 m. The tank contains an 80 m long side chamber at the position between 51.5 m and 59 m above the tunnel axis. The initial conditions are: water flow rate through the headrace tunnel of 72 m$^3$/s and the water level in the surge tank is 44,41 m. In the simulation, 10 s after the initial instant of time, the flow rate through the penstock is linearly decreased to zero over the closure period of 8 s.

Fig. 2 shows the history of the free surface level in the tank, compared to the experimental values [6, 3]. The agreement of the results is acceptable. For the operation of the plant, the maximum level during the first water rise is the most important. Obviously, the simulation predicted this value reasonably well, although the test in real conditions shows a certain damping in the subsequent cycles (especially, during the water sinking), which is not captured by the simulation. The reason for this discrepancy might be in the adopted value of the minor loss coefficient.
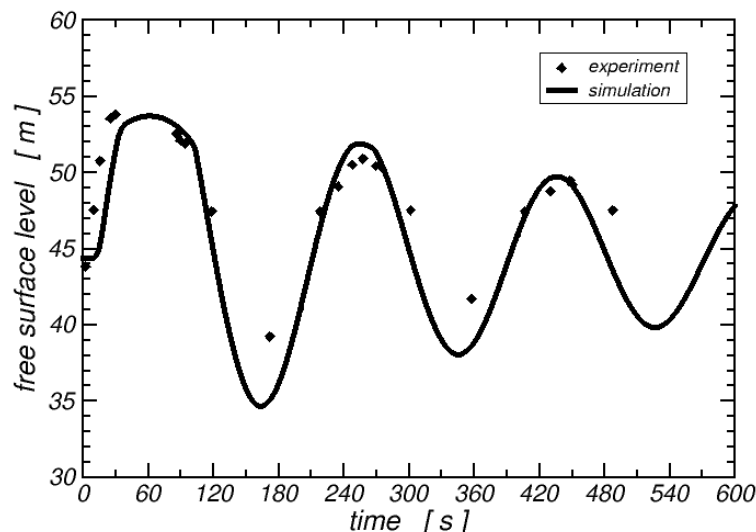


**Figure 2**. *Water level oscillation in the surge tank of HPP Jablanica for initial conditions $Q_0$ = 72 m$^3$/s, $H_0$ = 44,41 m above the tunnel axis.*

**3.2 Generic case**

A cylindrical surge tank whose diameter is 10 m is installed at the end of the headrace tunnel which is 2000 m long and whose diameter is 5 m. Three different types of turbine wicket-gate closure are considered: *linear*, *smooth* (described by a cosine function) and *sudden* closure. The linear and smooth closure are completed within 5 s. The calculated static pressure variation at the junction of the surge tank and the headrace tunnel, obtained for three different initial water levels, 50 m, 40 m, and 30 m above the tunnel axis, are shown in Fig. 3. The initial water flow rate is adopted to be 50 m$^3$/s. While the maximum pressure depends on the initial water level (higher initial water level,

higher static pressure), the pressure rise during the closure is more-or-less the same in all three cases and amounts to about 0.8 bar. Obviously, the difference between the linear and smooth closure is negligible, while the sudden closure implies slightly higher maximum pressure values with slightly faster variation in time.

In Fig. 4, the variation of the static pressure for different closure times at three different initial water levels is shown. The maximum pressure reduces with increase of the closure time, as expected, and the differences in the maximum pressure are found to be about 0.1 bar or less than that for the closure times between 5 s and 10 s.
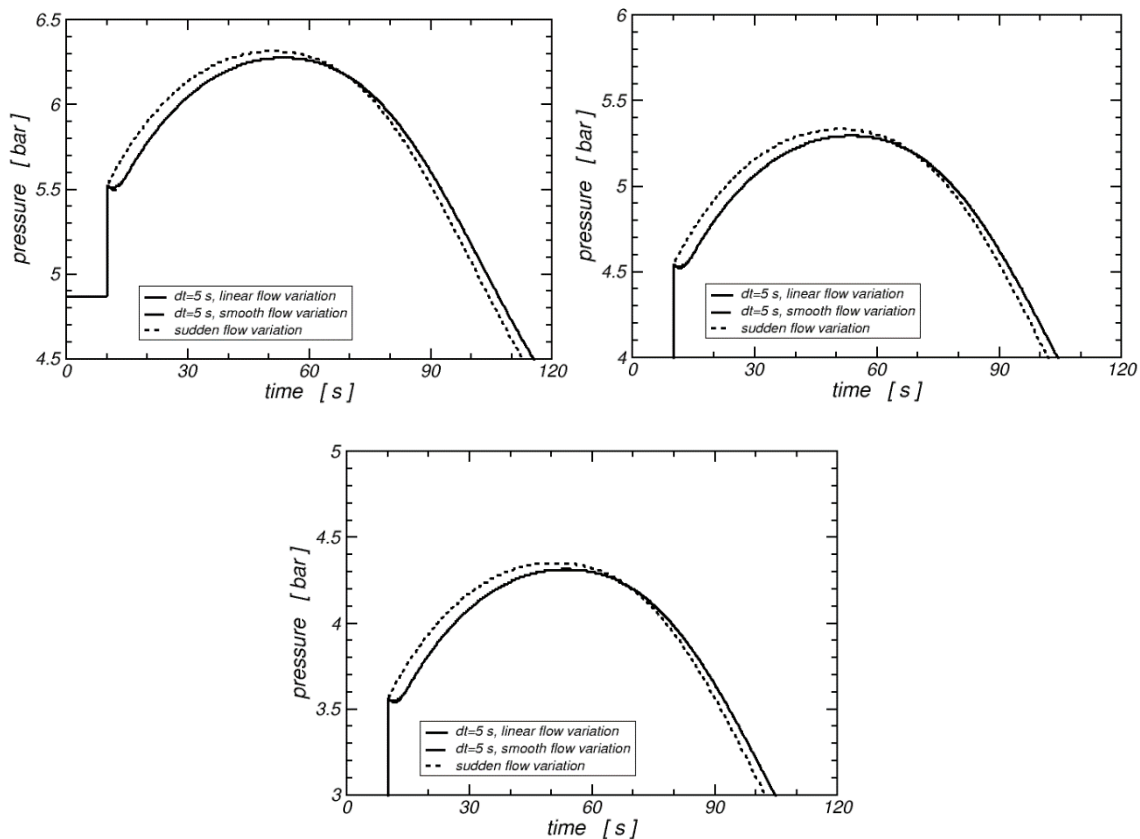


***Figure 3**. Static pressure variation in the junction for three different initial water levels in the tank: 50 m (top left), 40 m (top right), 30 m (bottom)*
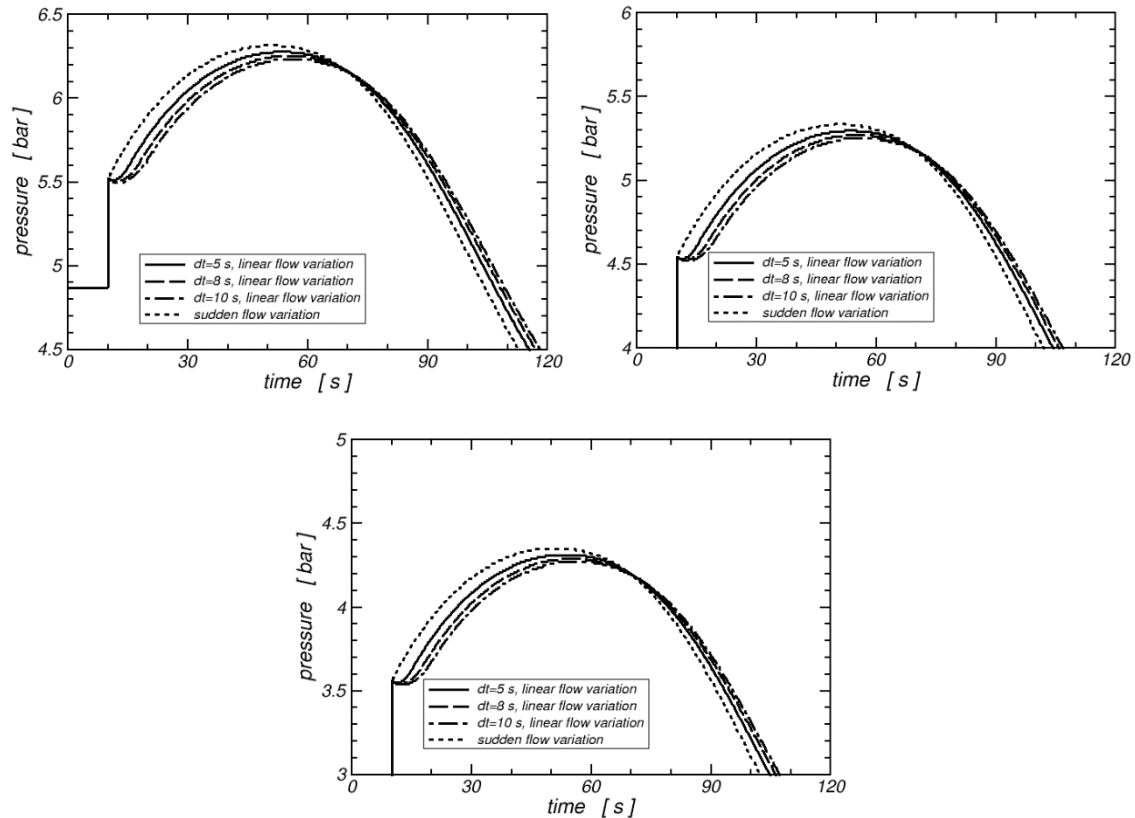
**Figure 4**. *Static pressure variation in the junction for different closure times ate three different initial water levels in the tank: 50 m (top left), 40 m (top right), 30 m (bottom)*
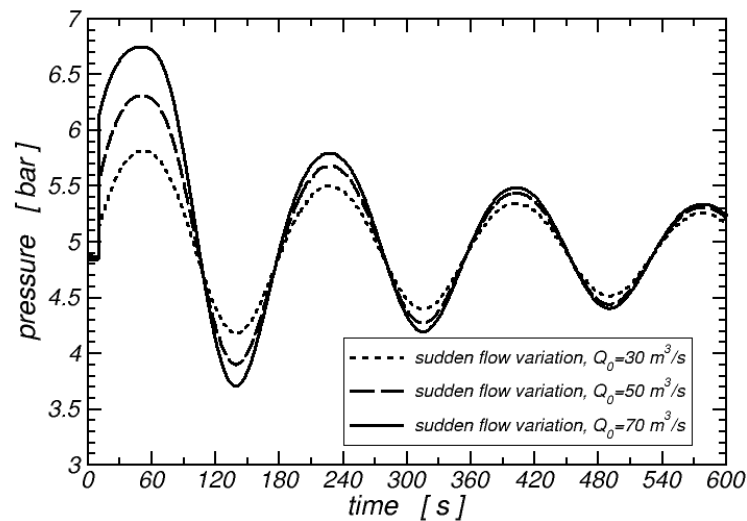


**Figure 5**. *Static pressure variation in the junction during sudden closure for three different initial flow rates, at the initial water level of 50 m above the tunnel axis.*

Fig. 5 shows the static pressure variation in the junction for three different initial water flow rates through the headrace tunnel. As expected, the pressure rise is larger for larger initial flow rates. As the results shown reveal, the maximum pressure during the first rise seems to be proportional to the initial water flow rate, while in the subsequent cycles it is not; a certain damping of the maximum pressure is detected, apparently being ever stronger in the further cycles, so that the effect of the initial flow rate diminishes. Interestingly, the period of oscillations remains the same for all initial

flow rates and throughout the entire simulated time.

**4. CONCLUSION**

A model based on numerical solution of two ordinary differential equations is used to obtain the water level in the surge tank and the static pressure in the headrace tunnel – the properties of essential importance for the functioning of the water supply system during the turbine shut-off. It was validated by comparing the numerical results with the data available from the experiments conducted under real conditions in a surge tank of the HPP Jablanica. The method allows a fast and reliable simulation of the hydraulic processes in the headrace tunnel and the surge tank

This model is used to analyze of influence of different parameters on variations of water level oscillation in the surge tank and the static pressure in the headrace tunnel. Influence of different types of turbine wicket-gate closure, different initial water levels above the tunnel axis, and different initial water flow rate are considered. Results of analysis obtained by model presented in this paper could be used during projecting a new hydro power plant as well as for optimal exploitation regimes of existing hydro power plants.

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