# Systematic Hydraulic Study for the Preliminary Sizing of the Surge Tanks Mounted Close to the Pumping Station

CLAUDIU STEFAN NITESCU, ANCA CONSTANTIN
Hydraulic Engineering Department , Faculty of Civil Engineering
"Ovidius" University
Constanta, 22B Unirii Str.
ROMANIA
claudiu.nitescu@univ-ovidius.ro, aconstantina@univ-ovidius.ro,
http://www.univ-ovidius.ro/faculties/civil_eng

Abstract: - The surge tanks, assumed to have no hydraulic loss at the bottom connection to the main discharge duct, are the simplest and safest protective device. An important number of pumping stations, part of the Romanian irrigation systems, were conceived with a surge tank mounted by the pump, on the discharge duct, in order to protect the installation from hydraulic shock. In practice, the sizing of such protection device usually implies a large volume of hydraulic calculation. Special charts (function of the hydraulic characteristics: discharge, geodetic head, diameter and length of the discharge duct) were conceived on the basis of the mathematical model of the hydraulic shock. They make the sizing more efficient and significantly reduce the time for the surge tank dimensioning.

*Key-Words:* - Surge tank, Air chamber, Pumping station, Geodetic head, Head loss

## 1 Introduction

During the last 40 years, an important number of pumping stations from the Romanian irrigation systems were designed with a surge tank on the discharge duct, next to the pump as a protection device from water hammer. Thus, the study of these hydraulic outlines is justified.

Referring to the 70 pumping stations analyzed in [5], 22% use the surge tank as a protection device and other 8% are equipped with both air chamber and surge tank. The other pumping stations use only air chamber.

In practice the dimensioning of such solution of protection implies a large volume of hydraulic calculi.
In such cases, it is recommended to make a pre-dimensioning calculus in order to find quick optimal design solutions.

The first stage in the study of pumping stations analyzes the surge tank neglecting its bottom hydraulic resistance - as the basic safest protection means.

A high-performance computer program conceived for the discharge ducts equipped with surge tank and a set of variables appropriated to these hydraulic outlines allowed us to obtain special diagrams for the first approximate dimensioning of this protection device.

## 2 Problem Formulation

In the case of hydraulic study aiming to dimension the surge tank that means the determination of the surge tank diameter and height, there were used only the following technical characteristics of the pumping installation: L -length and D -diameter of the discharge duct, $H_g$ - geodetic head, Q –discharge.

The calculus was developed considering the most usual ranges for the characteristics of the pumping installations: length of the discharge duct L= 100m - 1500m; geodetic height $H_g$ = 10 - 25m; discharge duct diameter D = 500 – 2000 mm and discharge Q = 1 – 4 $m^3/s$ .

Numerical simulation used *Hammer,* a computer program that uses the characteristics method for solving the hydraulic shock.

We assumed that the hydraulic shock is consequently to the simultaneous stoppage of the pumps as the electrical power accidentally shuts down.

Centrifugal pumps were taken into account because the oscillatory movement of the water in the hydraulic system is less influenced by characteristics of the pumps.

The following assumptions were made for the numerical study:

-one dimension flow,

-compressible liquid, without dissolved air,

-elastic and deformable conduit,

-longitudinal head losses concentrated in the calculus nodes,

-surge tank mounted next to the pump (at maximum 30m downstream the pump),

-negligible local head loss in the bottom connection section of the surge tank to the protected discharge conduit.

## 2.1 Non Dimensional Fundamental Equations

We attempt to determine general non-dimensional relationships using the equations specific for the studied system.

Referring to the installation composed of pump, discharge duct, surge tank and reservoir, as it is represented in Fig.1, we adequately write the motion and the continuity equations under the form of finite differences, in order to use the characteristic method.

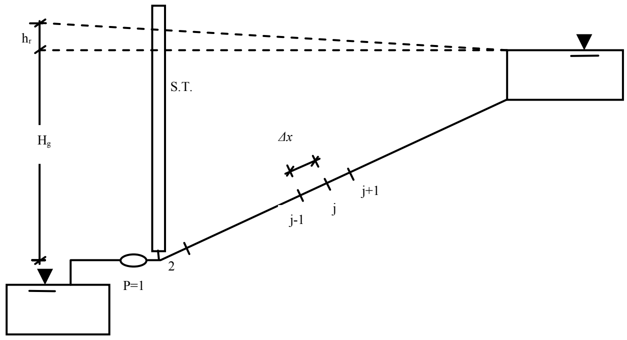


Fig.1 Pumping installation equipped with surge tank

In the intermediate section of the calculus node with surge tank, 3 unknowns occur: the speeds before and after this section l, $V'_{j,i+1}$ and respective $V''_{j,i+1}$ and water elevation in the surge tank $H_{j,i+1}$ (in this first stage of analysis we neglect the local head loss at the bottom of the surge tank).

$$\begin{cases} V'_{j,i+1} - V_{j-1,i} + \dfrac{g}{c}\left(H_{j,i+1} - H_{j-1,i}\right) + \\ \qquad + \dfrac{\lambda}{2D} V_{j-1,i}\left|V_{j-1,i}\right|\Delta t = 0 \\ V''_{j,i+1} - V_{j+1,i} + \dfrac{g}{c}\left(H_{j,i+1} - H_{j+1,i}\right) + \\ \qquad + \dfrac{\lambda}{2D} V_{j+1,i}\left|V_{j+1,i}\right|\Delta t = 0 \end{cases} \tag{1}$$

$$\frac{\Delta H}{\Delta t} = \frac{1}{F(H)}\left(V'_{j,i+1}\cdot f_1 - V''_{j,i+1}\cdot f_2\right) \tag{2}$$

The equation system (1) is solved and by the help of the rectangular net of the characteristics a single equation is obtained. Now we have two equations: movement equation,

$$V_{j,i+1} = \frac{1}{2}\Big[V_{j-1,i} + V_{j+1,i} + \frac{g}{c}\left(H_{j-1,i} - Hj+1,i\right) - \\ - \frac{\lambda\Delta t}{2D}\left(V_{j-1,i}\left|V_{j-1,i}\right| + V_{j+1,i}\left|V_{j+1,i}\right|\right)\Big] \tag{3}$$

and continuity equation for the surge tank node written under the form:

$$\frac{\Delta H}{\Delta t} = \frac{1}{F(H)}\left(V_{j-1,i+1}\cdot f_1 - V_{j+1,i+1}\cdot f_2\right) \tag{4}$$

where: *F(H)* – cross section area of the surge tank at the elevation *H* and time *i+1,*

$f_1$ and $f_2$ – cross section areas of the duct before and respective after the surge tank section [7], [9].

We determine bellow the non dimensional form of the equations, using the following notations:

$\vartheta$ - non dimensional speed;
$\zeta$ - non dimensional head;
$\tau$ - non dimensional time.

Thus, the non dimensional amounts will be:

$$V = V_*\vartheta = \frac{H_*}{T_*}\vartheta \tag{5}$$

$$H = H_*\zeta \tag{6}$$

$$\Delta T = \Delta\tau\cdot T_* \tag{7}$$

The equations become:

$$\vartheta_{j,i+1} = \frac{1}{2}\Big[\vartheta_{j-1,i} + \vartheta_{j+1,i} + \frac{gT_*}{c}\left(\zeta_{j-1,i} - \zeta_{j+1,i}\right) - \\ - \frac{\lambda H_*}{2D}\Delta\tau\left(\vartheta_{j-1,i}\left|\vartheta_{j-1,i}\right| + \vartheta_{j+1,i}\left|\vartheta_{j+1,i}\right|\right)\Big] \tag{8}$$

$$\frac{F}{f}\frac{d\zeta}{d\tau} = \vartheta_{j-1,i+1} - \vartheta_{j+1,i+1} \tag{9}$$

The following notations are added:

$$\alpha = \frac{gT_*}{c} \tag{10}$$

$$\beta = \frac{\lambda H_*}{2D} \tag{11}$$

$$\varphi = \frac{F}{f} \tag{12}$$

The non dimensional equation system written using the three above mentioned parameters and corresponding to the system made by the equations (3) and (4) will be:

$$\begin{cases} \vartheta_{j,i+1} = \dfrac{1}{2}\Big[\vartheta_{j-1,i} + \vartheta_{j+1,i} + \alpha\left(\zeta_{j-1,i} - \zeta_{j+1,i}\right) - \\ \qquad - \beta\left(\vartheta_{j-1,i}\left|\vartheta_{j-1,i}\right| + \vartheta_{j+1,i}\left|\vartheta_{j+1,i}\right|\right)\Big] \\ \varphi\dfrac{d\zeta}{d\tau} = \vartheta_{j-1,i+1} - \vartheta_{j+1,i+1} \end{cases} \tag{13}$$

Taking into account that the values for the reference amounts $T_*$ and $H_*$ may be arbitrarily chosen, there were adopted values which equal the parameters $\alpha, \beta$ to the unit. Therefore the calculus become more simple.

$$\left.\begin{array}{l} T_* = \dfrac{c}{g} \\ H_* = \dfrac{2D}{\lambda} \end{array}\right\} \Rightarrow \alpha, \beta = 1 \quad (14)$$

The known initial conditions we start from are

$$t = \tau_0 = 0 \quad \text{then} \quad \zeta_0 = \frac{H}{H_*}, \quad \vartheta_0 = \frac{T_*}{H_*} V_0 \quad (15)$$

Applying this calculus method, we find out the values $\zeta(\tau)$ respective $\vartheta(\tau)$ that lead us to their dimensional correspondents: *z(t)* and *v(t)*.

Approaching by this non dimensional method the extreme values $\zeta_{\max}$, $\zeta_{\min}$ are functions of three non dimensional parameters $\zeta_{\max}, \zeta_{\min} = f(\phi, \vartheta_0, \zeta_0)$. The first parameter is depends on the installation features and the other two represent the initial conditions.

#### 2.1.1 Technically and Economically Imposed Dimensions

The diameter of the surge tank section is continually increased starting from a value equal to the conduit diameter $\left(\varphi = 1 = \dfrac{F}{f}\right)$ in order to calculate the water elevation inside the surge tank [6].

When the pumping installation is out of operation, the water elevation in the surge tank is the same as in the reservoir. For economical reasons we imposed a maximum water level inside the surge tank during the operation maneuvers; this level determines the limit of the maximal jump. A maximal jump, close to the water level in the surge tank during exploitation, wouldn't lead to an over dimensioning of the surge tank, which is the case for the transitory flow.

$$(z_{\max} \cong h_r) \to (\zeta_{\max}), \quad \zeta_{\max} = \frac{H_{\max}}{H_*} \quad (16)$$

The highest level of water in the surge tank is reached by starting up the pumping station. Therefore, numerical computation is done in order to analyze the level variation for the start-up of a pumping station. Most pumping stations use parallel mounted pumps that are started gradually. Each pump is started only after the previous has reached its normal operation regime, or after a time interval imposed by the pump characteristics. There are also some pumping stations that allow a simultaneous start of up to half of the total number of pumps in installation.

The modernization of the pumping station as of late showcases the variable speed pumps, that offer the option of a more controlled and safer starting manner. A gradual increase of the pumped flow leads to a maximum jump that is less than the one recorded when the station is started at a maximum flow.

For technical and operational reasons we imposed a minimal water level in the surge tank during the pumping installation start up and stop manoeuvres. The surge tank is mounted close to the pumping station, at maximum 30 m, therefore it is necessary to have a safety minimal water level in the surge tank (approximately 30%). Thus, the water level in the surge tank must be $H_{\min\ water} \geq 0{,}3 H_g$ . Once this minimal level is established, the value for the minimal jump is implicitly imposed. .

$$z_{\min} = f\left(H^{ST}_{\min.water}\right)$$

$$z_{\min} \to \zeta_{\min}, \quad \zeta_{\min} = \frac{H^{ST}_{\min}}{H_*} \quad (17)$$

### 2.2 Numerical simulation of a specific case

The numerical calculi were made for the same outline of a pumping station, Fig.1, that has the following features: $H_g$=10m, L=250m and Q=1m$^3$/s. The discharge duct is made by steel and the longitudinal head loss coefficient was determined as depicted in [2], [3].

In the subsection that refers to the start-up of the pumping installation, we will consider that the discharge is delivered by three identical pumps, mounted in parallel. This assumption is consistent with the other calculation with respect to the shut-down of the installation, because in the hydraulic shock study all the parallel mounted pumps are considered equivalent to one single pump.

#### 2.2.1 Numerical simulation for the start-up of the pumping station with no surge tank

The numerical simulation is done considering the hydraulic layout presented in Fig. 1, but in the absence of any protection device. It was assumed a sudden stoppage of the pump.

There were considered two values for the discharge duct diameter: 600 mm and 900 mm. These values are in accordance with a water velocity range of 1,5-3 m/s. The discharge duct is divided in 10 equal longitudinal sections.

The pressure diagrams, Fig. 2-5 show the occurrence of cavitation in the upstream sections of the discharge duct. Consequently, a protection device is needed on the pipeline.

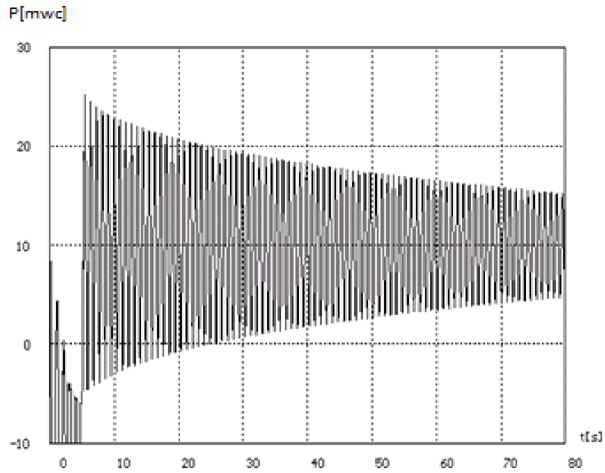


Fig.2 Pressure variation in node 1, for a duct diameter D=600 mm

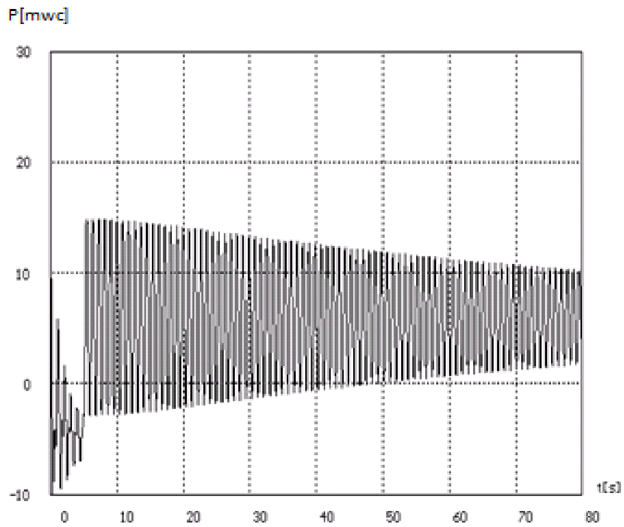


Fig.3 Pressure variation in node 5, for a duct diameter D=600 mm

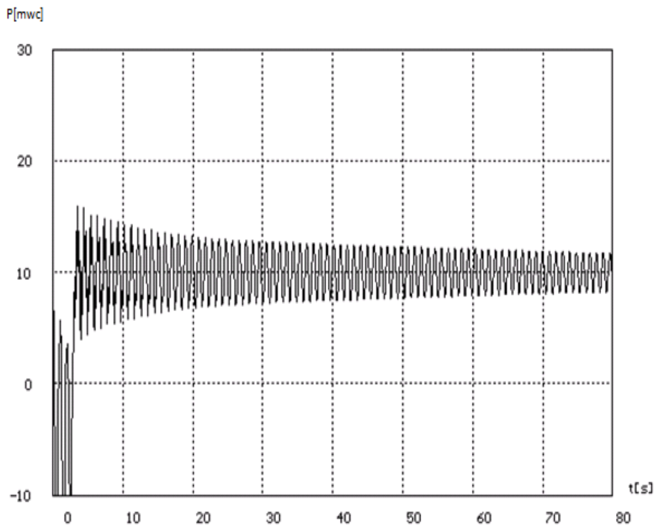


Fig.4 Pressure variation in node 1, for a duct diameter D=900 mm

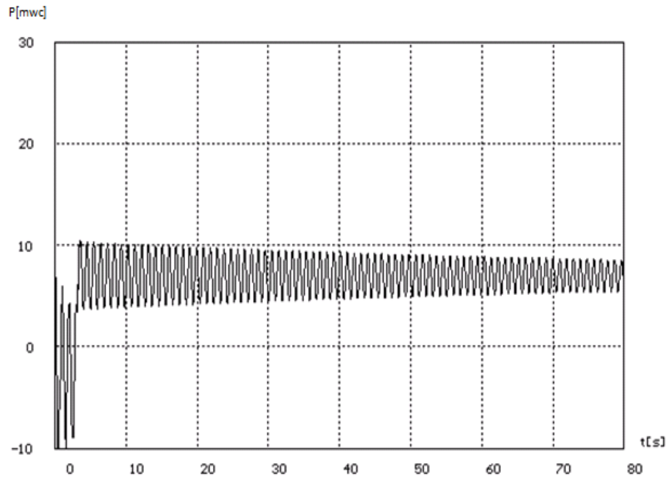


Fig.5 Pressure variation in node 4, for a duct

diameter D=900 mm

### 2.2.2 Numerical simulation for the start-up of the pumping station equipped with surge tank

The numerical simulation was conducted considering three parallel mounted pumps in the installation. The pumps start one by one.

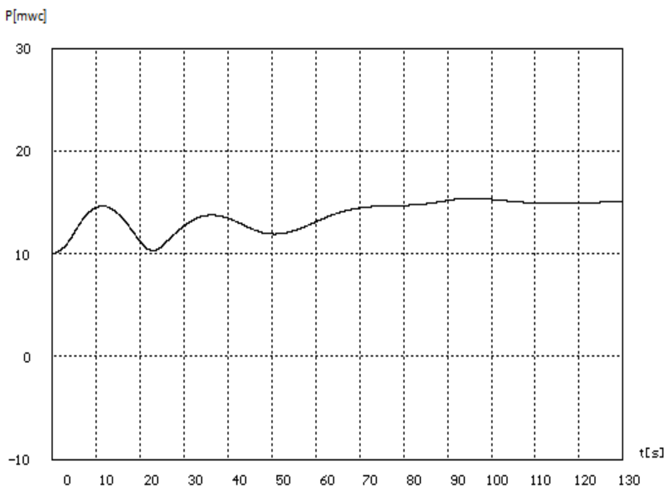


Fig.6 Water level inside the surge tank, in the case of 3 pumps discharging on a 600 mm pipeline. The pumps start one by one. Surge tank diameter $D_{ST}$=700mm.

There were considered discharge ducts with diameters of 600 mm and 900 mm protected by surge tanks of 700 mm and respective 1000 mm in diameter. It is obvious, even in the case of small surge tank diameter (close to the duct diameter value) that the maximum jump is smaller when the pumping installation gradually starts. The maximum jump value oscillates around the regime level value. The minimum water level in the surge tank is much higher than the imposed value ($H_{min}\ \ water \geq 3\ \ m$).

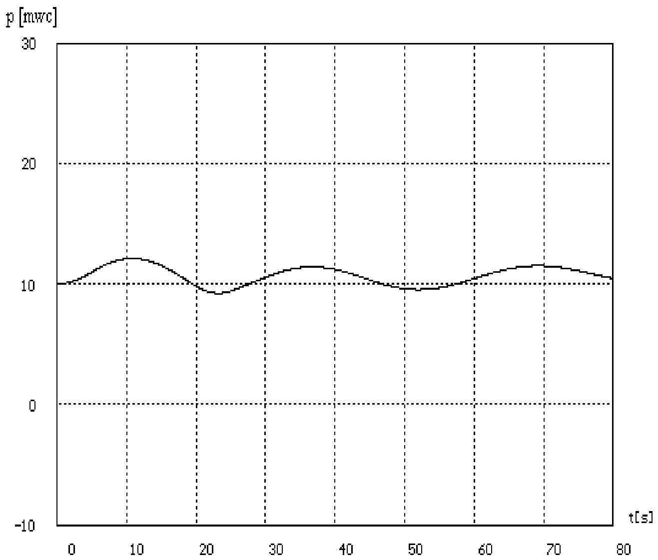


Fig.7 Water level inside the surge tank, in the case of 3 pumps discharging on a 900 mm pipeline. The pumps start one by one. Surge tank diameter $D_{ST}$=1000mm.

A gradual start-up of the pumps in installation offers significant advantages, especially when it is known the minimal time interval for the next pump to start-up safely.

Thus, a controlled start-up of the pumping installation doesn't request special conditions: there is assured the minimum level in the surge tank and the maximum level is governed by the pumping installation starting laws.

### 2.2.3 Numerical simulation for the sudden stoppage of the pumping station with surge tank

Tables 1-4 show the minimal and maximal water levels in the surge tank during a sudden stopping of the pump for four values of the discharge conduit diameter.

Table 1

| D | $D_{ST}$ | $N_{Min}$ | $N_{Max}$ |
|---|---|---|---|
| 0.60 | 0.60 | -3.328 | 20.133 |
| | 0.70 | -1.073 | 18.143 |
| | 0.80 | 0.595 | 16.722 |
| | 0.90 | 1.906 | 15.641 |
| | 1.10 | 3.826 | 14.126 |
| | 1.30 | 5.145 | 13.141 |
| | 1.50 | 6.098 | 12.462 |
| | 1.70 | 6.810 | 11.975 |
| | 1.90 | 7.356 | 11.615 |
| | 2.10 | 7.782 | 11.340 |
| | 2.30 | 8.120 | 11.129 |

Table 2

| D | $D_{ST}$ | $N_{Min}$ | $N_{Max}$ |
|---|---|---|---|
| 0.70 | 0.70 | -0.562 | 18.757 |
| | 0.90 | 1.936 | 16.408 |
| | 1.10 | 3.586 | 14.909 |
| | 1.30 | 4.744 | 13.891 |
| | 1.50 | 5.600 | 13.163 |
| | 1.70 | 6.254 | 12.623 |
| | 1.90 | 6.771 | 12.210 |
| | 2.10 | 7.187 | 11.886 |
| | 2.30 | 7.528 | 11.628 |
| | 2.50 | 7.812 | 11.418 |
| | 2.70 | 8.052 | 11.246 |

Table 3

| D | $D_{ST}$ | $N_{Min}$ | $N_{Max}$ |
|---|---|---|---|
| 0.80 | 0.80 | 1.593 | 17.346 |
| | 0.90 | 2.535 | 16.426 |
| | 1.10 | 3.946 | 15.069 |
| | 1.30 | 4.952 | 14.118 |
| | 1.50 | 5.697 | 13.427 |
| | 1.70 | 6.274 | 12.901 |
| | 1.90 | 6.731 | 12.493 |
| | 2.10 | 7.101 | 12.168 |
| | 2.30 | 7.408 | 11.903 |
| | 2.50 | 7.665 | 11.685 |
| | 2.70 | 7.884 | 11.503 |
| | 2.90 | 8.072 | 11.350 |
| | 3.10 | 8.236 | 11.218 |
| | 3.50 | 8.379 | 11.105 |

Table 4

| D | $D_{ST}$ | $N_{Min}$ | $N_{Max}$ |
|---|---|---|---|
| 0.90 | 0.90 | 3.180 | 16.203 |
| | 1.10 | 4.403 | 14.995 |
| | 1.30 | 5.282 | 14.135 |
| | 1.50 | 5.937 | 13.500 |
| | 1.70 | 6.446 | 13.011 |
| | 1.90 | 6.851 | 12.626 |
| | 2.10 | 7.181 | 12.316 |
| | 2.30 | 7.454 | 12.061 |
| | 2.50 | 7.684 | 11.849 |
| | 2.70 | 7.881 | 11.669 |
| | 2.90 | 8.050 | 11.516 |
| | 3.10 | 8.198 | 11.383 |
| | 3.30 | 8.328 | 11.269 |
| | 3.50 | 8.443 | 11.168 |

Minimal and maximal water levels are measured with respect to the pump axis.

The geodetic height influences the pressure values developed in stationary regime of flow. This influence diminishes if we use the over and under pressures measured with respect to the water level in the discharge reservoir (that means maximal and minimal jump notation is used).

The total height of the surge tank is determined by summation of the geodetic height with the head loss and the additional safety height ( $H_{ST}=H_g+h_r+h_s$ ).

In the case of high geodetic height and long discharge conduits (the longitudinal head loss is proportional to the conduit length) the maximal value of the head loss is limited in order to avoid the extreme dimensions of the surge tanks. It is known that surge tanks higher then 25m aren't recommended [8].

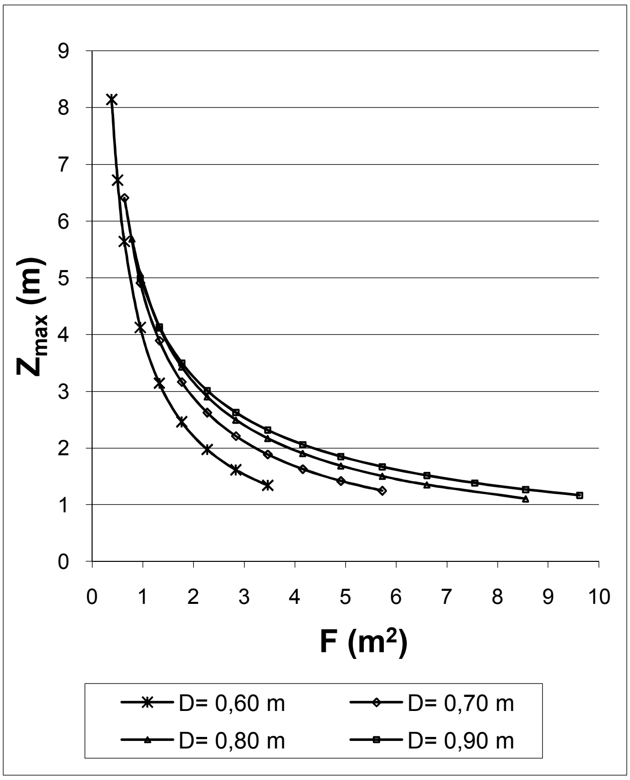


Fig.8.Maximal jump of the water level as a function of the surge tank cross section area

The graphical representation of these values lead to the bellow diagram, where:

$Z_{max}$ – maximal jump of water level in the surge tank;

$Z_{min}$ –minimal jump of water level in the surge tank, (both measured from the water level in the discharge reservoir);

F – cross-section aria of the surge tank [6].

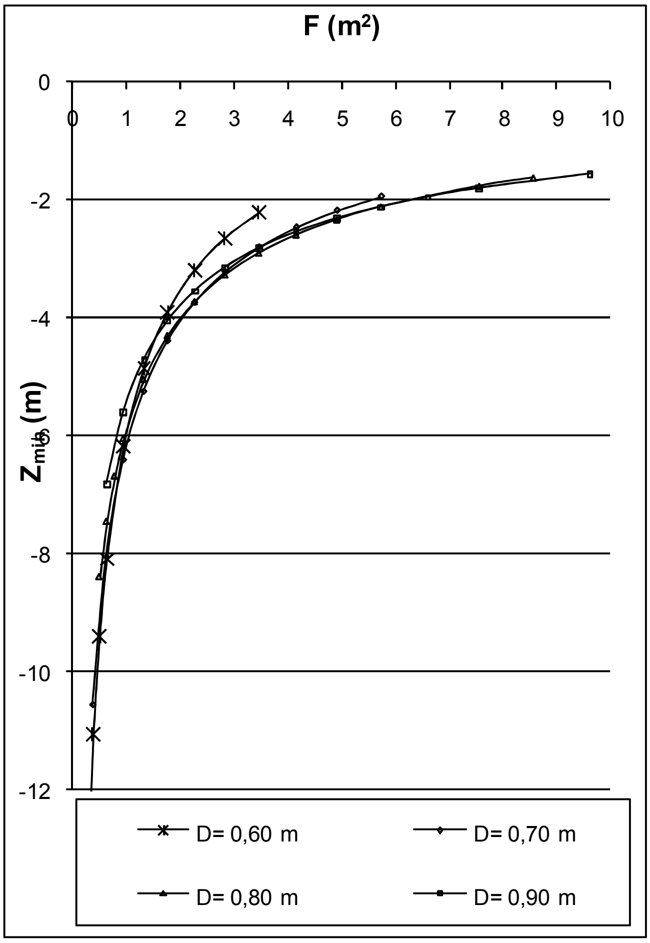


Fig.9. Minimal jump of the water level as a function of the surge tank cross section area

It is observed that at a sudden stopping of the pump the minimal jump varies in an acceptable range of values, for diameters corresponding to usual values of water speed in the discharge duct. It may be concluded that the minimal level of water in the surge tank is assured.

Consequently, the protection device will be dimensioned according to the maximal jump in the surge tank (maximal water level). After the determination of the surge tank dimensions, the minimal water level must be checked.

# 3 Problem Solution

## 3.1 Diagram Construction

The calculus method based on the jumps ($z_{max}$ and $z_{min}$) measured from a reference plane placed at the water level in the discharge reservoir offers the advantage of an easier determination of the extreme jumps for different geodetic heights. Consequently, superposing the two above diagrams we may obtain a general one which allows us to determine $D_{ST}$ as a function of the non dimensional amounts $\varphi$ and $\zeta$.

Knowing the values $\zeta_{max}$, $\zeta_{min}$ deduced from the imposed minimal and maximal jumps, we find out the value of $\varphi$ and then the surge tank diameter, as a function of conduit cross section area *f*. The obtained diameter offers protection to the discharge conduit and assures both minimal and maximal necessary water levels.

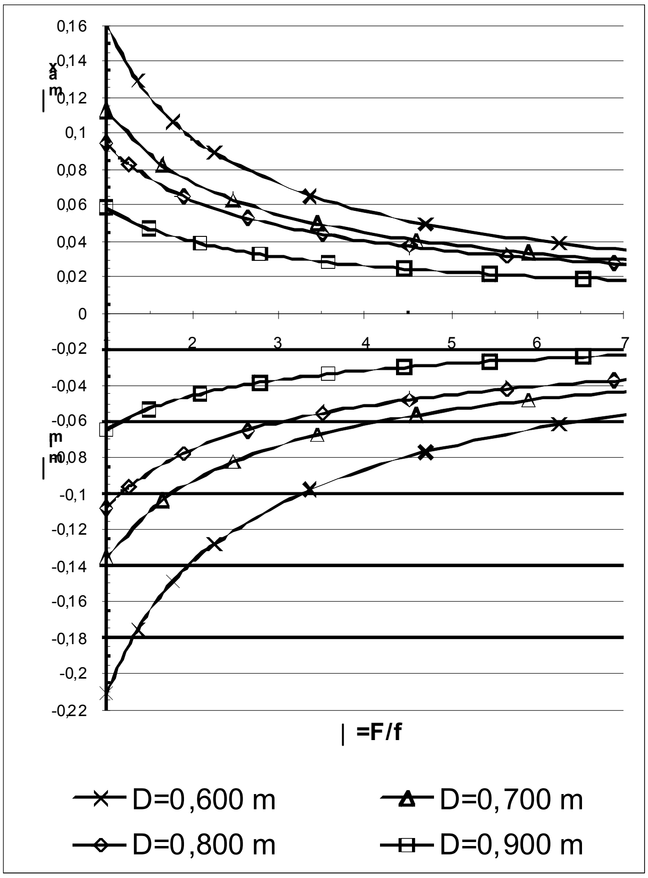


Fig.10. General diagram for surge tank dimensioning, for Q=1m$^3$/s, L=250m and 1<φ<7

$$\zeta_{max} = z_{max} \frac{\lambda}{2D} \tag{18}$$

$$\zeta_{min} = z_{min} \frac{\lambda}{2D} \tag{19}$$

In the case of a greater discharge (Q=2m$^3$/s, 3m$^3$/s, 4m$^3$/s) and the same duct length, results a range limited by a minimal and a maximal diameter according to the usual velocity field. With these known data (Q, L, $D_{max}$, and $D_{min}$) similar diagrams may be drawn.

Following the same principle, diagrams for different lengths of duct (L=100m, 250m, 500m, 1000m, 1500m) may be analogously drawn for each single value of the discharge flow rate (Q=1-4m$^3$/s). [4]

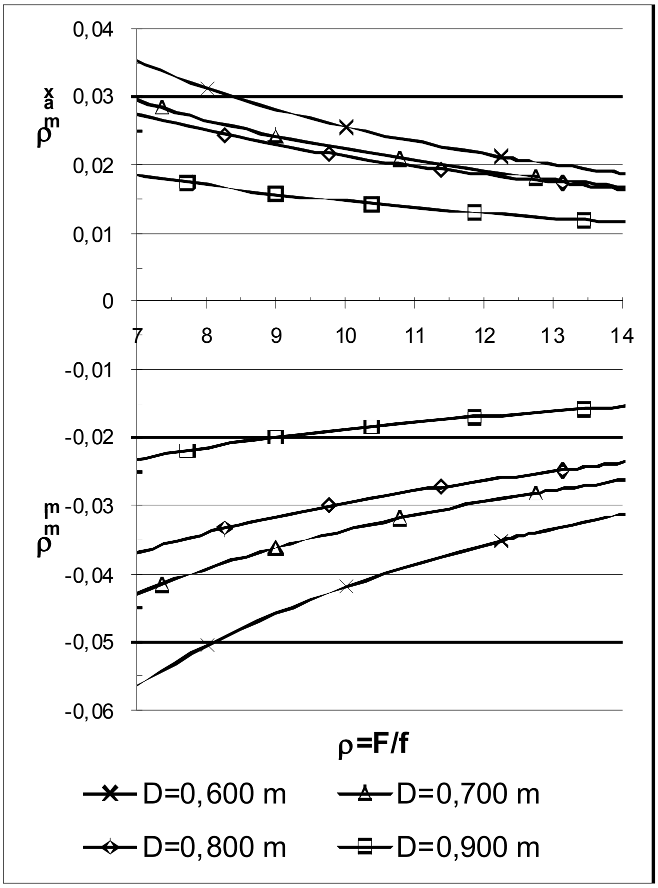


Fig.11. General diagram for surge tank dimensioning, for Q=1m$^3$/s, L=250m and 7<φ<14

In the case of pumping stations with discharge duct of other diameter value than those found in the diagrams, an extrapolation or interpolation can be made.

### 3.2 How to Use the Diagrams

The dimensioning of the protection device may be carried out using a few sets of diagrams.

The starting data are: the discharge, Q, the length of the discharge duct, L, the diameter of the discharge duct, D, the non dimensional amount $\zeta_{max}$ previously determined by the relation (18) with λ obtained according to [2] and the imposed value of water level, $z_{max}$.

The procedure implies the work with two sets of diagrams, in parallel.

- First set is composed of two diagrams, one for a smaller and the other for a larger discharge than Q, ($Q_1<Q<Q_2$) and the duct's length $L_1$ smaller than L. In accordance with $\zeta_{max}$, we can determine $F_1$ and $F_2$, respective $\zeta^1_{min}$ and $\zeta^2_{min}$ from the two

diagrams. By linear interpolation we may obtain the value of the surge tank area F* and the coefficient $\zeta^*_{min}$, for Q and L1 (intermediate values);

- The second set is composed by other two diagrams, one for a smaller and the other for a larger discharge than Q, ($Q_1<Q<Q_2$) and the duct's length $L_2$ greater than L. Fron this two diagrams we determine the values $F_3$ and $F_4$, and respective $\zeta^3_{min}$ and $\zeta^4_{min}$. (according to the same $\zeta_{max}$ ) . A second linear interpolation allows us to find out the surge tank area F** and the coefficient $\zeta^{**}_{min}$, for the discharge Q, and $L_2$ (intermediate values);
- Finally we determine the values of F and $\zeta_{min}$ for the exact length of the duct, L. By interpolation between the values F* and F** results F, and between $\zeta^*_{min}$ and $\zeta^{**}_{min}$ results $\zeta_{min}$.

F* respective F** can be determined according to the duct's length L, and in the end to determine F for the discharge Q.

The stages of calculation are the same, thus the final results are identical. The errors of the results obtained by this graphical method versus the results obtained by running the software Hammer are admissible and belong to the field (-5% ÷ +5%).

### 3.3 Numerical calculus example

The numerical calculus aims to determine the surge tank diameter, neglecting the hydraulic resitance of the bottom connection to the duct, in the case of a water pumping station that has to be protected from hydraulic shock. The pumping station has the following features:

- discharge, Q = 2.3 $m^3/s$ ;
- length of the discharge duct, L = 425 m ;
- diameter of the discharge duct, D = 1.100 m ;
- pumping geodetic height, $H_g$ = 17 m ;
- longitudinal head losses, $h_r$ = 1.8455 m ;
- friction coefficient $\lambda = 0.016$ (calculated according to [2]).

There are imposed a 2 m maximum water jump in the surge tank and a minimum one of at least 3 m.

On the basis of these given data and by the use of the diagrams, it will be determined the value of the surge tank diameter that satisfies the imposed conditions.

It is determined $\zeta_{max} = \frac{z_{max} \cdot \lambda}{2D} = 0.0145$

This value $\zeta_{max} = 0.0145$ is inserted in the diagrams corresponding to the flow rates of 2 $m^3/s$ and 3 $m^3/s$ and a length of the discharge pipe of 250 m; a horizontal line is drawn through this point until it crosses the curve for 1100 mm diameter. In the case there is not a curve drawn for the requested diameter, interpolation has to be done.

From the 2 $m^3/s$ discharge diagram result $\varphi_1 = 11.25 \rightarrow F_1 = \varphi_1 \cdot f = 10.6912 m^2$ and $\zeta^1_{min} = -0.01875$ .

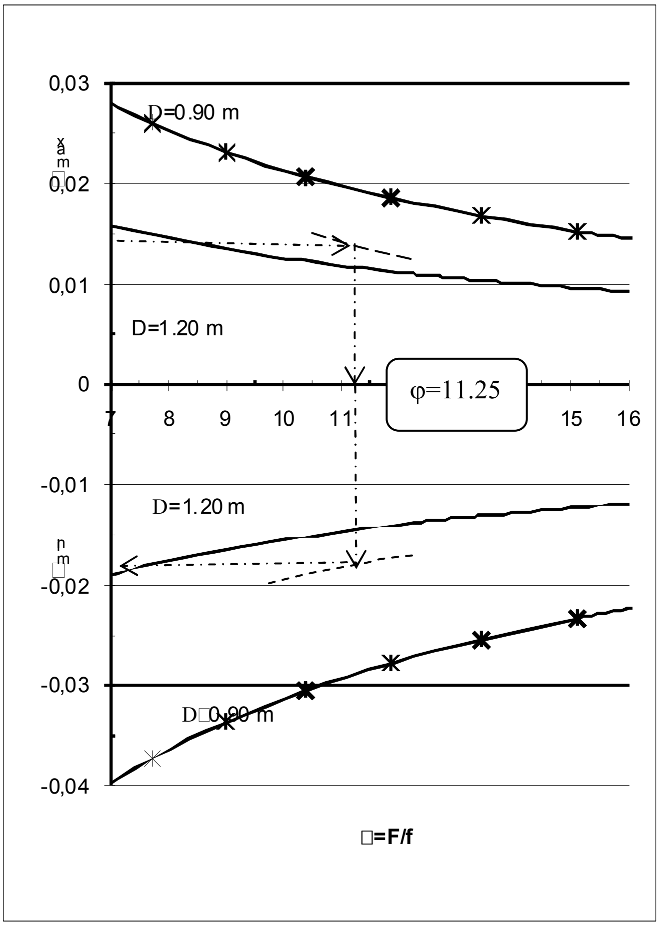


Fig.12. Graphical determination on the diagram for Q=2m³/s, L=250m and D=1.10 m

From the 3 $m^3/s$ discharge diagram result $\varphi_2 = 14.20 \rightarrow F_2 = \varphi_2 \cdot f = 13.4947 m^2$ and $\zeta^2_{min} = -0.02136$ .

The values obtained from the above two diagrams, φ=11.25, $\zeta^1_{min} = -0.01875$ and φ=14.20, $\zeta^2_{min} = -0.02136$ allow the determination (by linear interpolation) of the surge tank cross area value

*F*=11.5322 m²* and $\zeta^{*}_{\min} = -0.01953$ for a discharge of *Q=2.3 m³/s* .

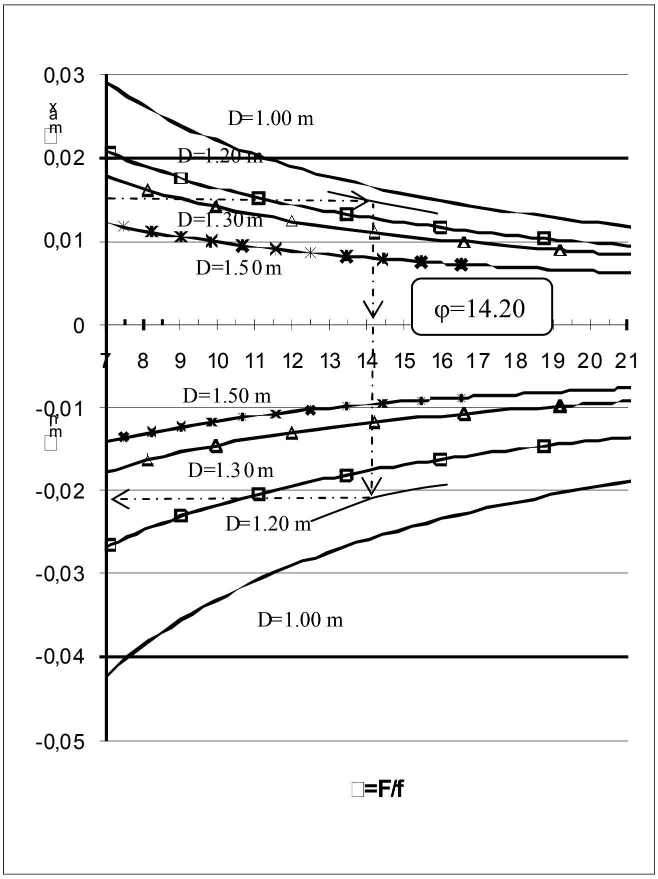


Fig.13. Graphical determination on the diagram for Q=3m³/s, L=250m and D=1.10 m

A similar procedure is used for the pipeline length of 500 m using diagrams corresponding to the flow rates of 2m³/s and 3m³/s. The following values result: $\varphi_3 = 13.20 \rightarrow F_3 = \varphi_3 \cdot f = 12.54 m^2$, $\zeta^{3}_{\min} = -0.022$ and $\varphi_4 = 16.60 \rightarrow F_4 = \varphi_4 \cdot f = 15.77 m^2$, $\zeta^{4}_{\min} = -0.02294$ for the same value of $\zeta_{\max} = 0.0145$. By linear interpolation of these values results *F**=13.505 m²* and $\zeta^{**}_{\min} = -0.02228$ for the discharge *Q=2.3 m³/s* .

According to these intermediary data, F and $\zeta_{\min}$ values can be determined for a length L = 425 m. A new interpolation between the values $F^*$ and $F^{**}$ is needed. We obtain *F=12.913 m²*.

According to the values $\zeta^{*}_{\min}$ and $\zeta^{**}_{\min}$ results $\zeta_{\min} = -0.02145$.

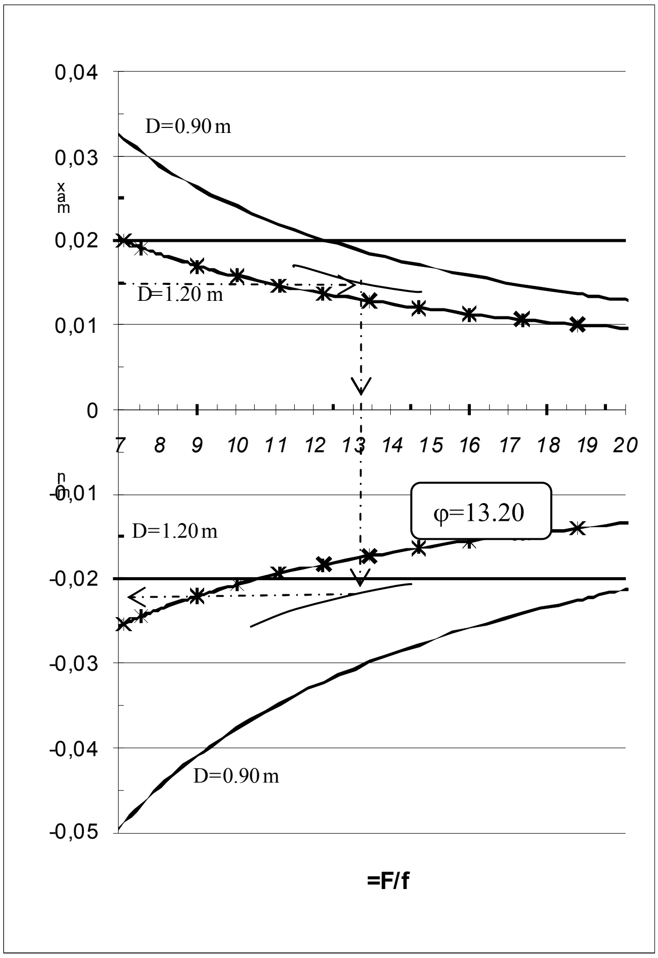


Fig.14. Graphical determination on the diagram for Q=2m³/s, L=500m and D=1.10 m

$$F = \frac{\pi D_{ST}^2}{4} \Rightarrow D_{ST} = 4.06 m$$

$$z_{\min} = -2.949 m$$

The Hammer program gives a maximum jump of $z_{max}$=2.054 m and a minimal one of $z_{min}$= -2.963m, using the same initial data and the surge tank diameter $D_{ST}$=4.06m. The differences between the results obtained by the use of the diagrams and by the use of the computing program are:

$$\varepsilon\%_{z\max} = \frac{|2 - 2.054|}{2.054} \cdot 100 = 2.62\%$$

$$\varepsilon\%_{z\min} = \frac{|2.949 - 2.963|}{2.963} \cdot 100 = 0.64\%$$

The errors are below the value $|5\%|$ . They depend on the ability of using the diagrams and on the diagrams accuracy (the errors are larger in the zones with small distance between curves).

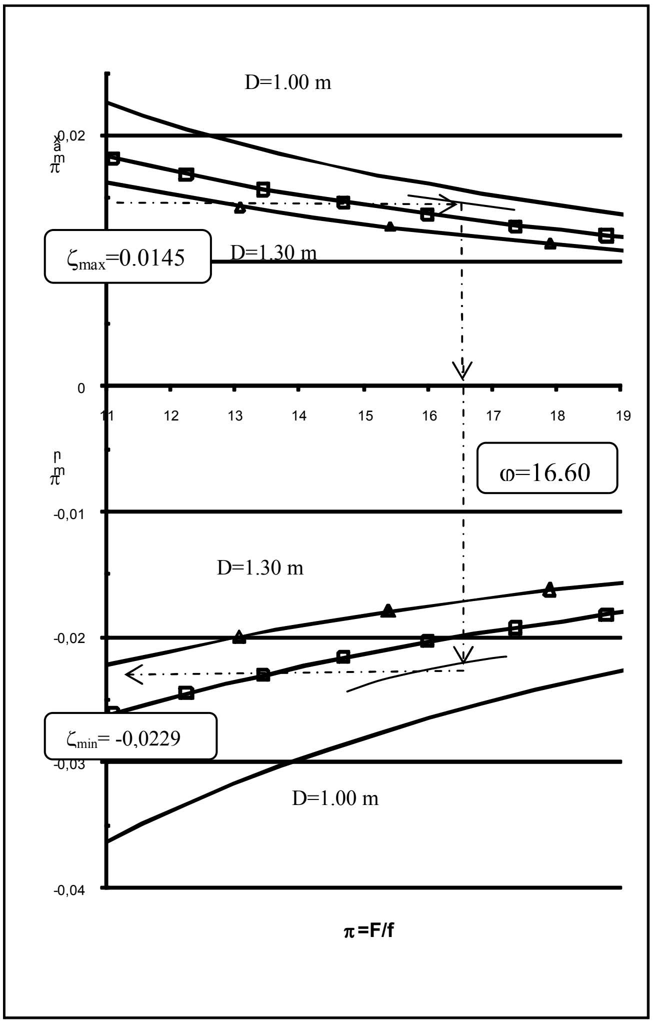


Fig.15. Graphical determination on the diagram for $Q=3m^3/s$, L=500m and D=1.10 m

$F^*$ and $F^{**}$ can be determined ($\zeta^*_{min}$ and $\zeta^{**}_{min}$ respectively) according to the pipeline length $F^*=f(F_1;F_3)$ and $F^{**}=f(F_2;F_4)$, ($\zeta^*_{min}=f(\zeta^1_{min};\zeta^3_{min})$ and $\zeta^{**}_{min}=f(\zeta^2_{min};\zeta^4_{min})$ respectively). After a new interpolation $F$ and $\zeta_{min}$ can be calculated, correspondent to the required water discharge.

The errors between the two methods are negligible and they are a results of the approximations used at interpolations.

# 4 Conclusion

The diagrams obtained using the method based on the non-dimensional equations of unsteady movement in a discharge duct allow us to easily and efficiently calculate the diameter and the height of a surge tank that offers stability to the hydraulic system. Only a few starting data for the pumping station are required, namely the water discharge, Q, the length L, and diameter D, of the pipeline, and some technical and operational conditions.

The main advantages of the proposed method are the rapidness of the calculus and the accuracy of the results in spite of the complexity of the phenomena. It is a performance tool for engineering designers as it offers accurate results for a small amount of known data and considerably reduces the number of trial calculation.

When considering the hydraulic resistance at the bottom of the surge tank, the results obtained by the above presented method might be used as starting data for the subsequent stages of the calculus.

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