# Simulation of Water Hammer Oscillations in Single Pipe Line due to Sudden Valve Closure

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**Abstract:** Modeling and simulation are very powerful tools and have become an integral part in the design and development of engineering systems. The objective of this research is to develop a computer program to simulate water hammer oscillations in a single pipe line. A mathematical model has been developed to simulate water hammer in one dimensional single pipe line. A new FORTRAN program called HAM01 is developed to achieve the present work goal. The program is used to predict the discharge and pressure distribution in single pipe line. The predictive results are compared with previous numerical results. The results show the damping of pressure and discharge with time after fast close of the valve.
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**Keywords:** water hammer, surge pressure, hydraulic transient.

## 1. Introduction

Most engineers involved in the planning of pumping systems are familiar with the terms “hydraulic transient”, “surge pressure” or, in water applications,” water hammer”. In a hydropower station, the hydro-turbine frequently adjusts the discharge according to the electricity load. In some cases the unit will shutdown in emergency, and the "water hammer" phenomenon will inevitable happen in the pressure pipeline (penstock).This phenomenon may occur in all of pressure pipeline system, often bringing about strong vibration and damage on the pipeline. Therefore, the calculation of water hammer plays an important role in the design and operation of a hydropower station (in a broad scope, including all pressurized piping systems) [1]. If sudden velocity and pressures variations are caused in a pipe system, for example when pumps are shut off or valves are closed, a pressure wave develops which is transmitted in the pipe at a certain velocity that is determined by fluid properties and the pipe wall material. This phenomenon, called water hammer, can cause pipe and fittings rupture. The intermediate stage flow, when the flow conditions are changed from one steady state condition to another steady state, is called transient state flow or transient flow; water hammer is a transient condition caused by sudden changes in flow velocity or pressure [2]. Figure (1) shows the water hammer description with different valve positions.

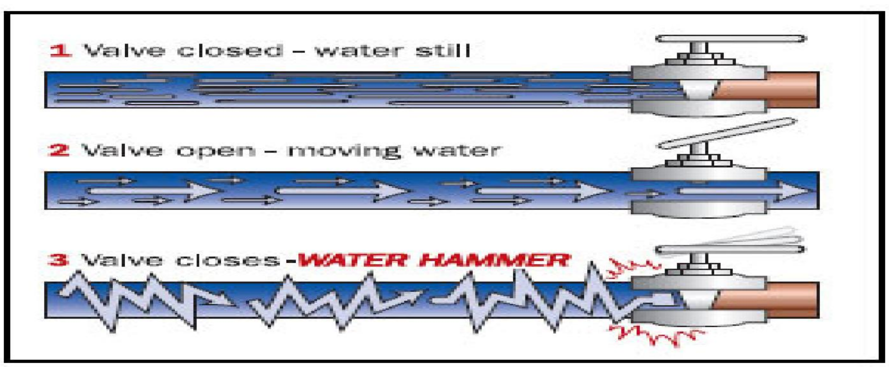


Figure (1) Water hammer phenomena

Unsteady flow in pipe networks is usually analyzed by means of one-dimensional models. Silva-Araya [3] developed a model to simulate unsteady friction in transient flow which was tested in simple pipe – valve systems only. The model approximates the velocity profiles during the transient flow and computes the actual energy dissipation. The ratio of the energy dissipation at any instant and the energy dissipation obtained by the quasi-steady approximation is defined as the Energy

Dissipation Factor. This is a time-varying, non-dimensional parameter that is incorporated in the friction term of the transient flow governing equations

Arangoitia Valdivia [4] developed a mathematical model for the calculation of water hammer considering the energy dissipation. He studied the water hammer produced by the instantaneous closure of a valve, located at the end of a pipe line connected to a constant head reservoir. The model consists of linearizing the equations of transient flow, to establish an equation similar to the one corresponding to an oscillatory mechanical system and to use this equation as a boundary condition, in the method of characteristics.

Zhao and Ghidaoui [5] formulated, applied, and analyzed first and second order explicit finite volume Godunov-type schemes for water hammer problems. The finite volume formulation ensures that both schemes conserve mass and momentum and produce physically realizable shock fronts. The implementation of boundary conditions, such as, valves, pipe junctions, and reservoirs, within the Godunov approach is similar to that of the method of characteristics approach. The schemes were applied to a system consisting of a reservoir, a pipe, and a valve and to a system consisting of a reservoir, two pipes in series, and a valve.

Vardy and Brown [6] presented a method for evaluating wall shear stress from known flow histories in unsteady pipe flows. The method builds on previous work by Trikha, but has two important differences. One of these enables the method to be used with much larger integration time steps than are acceptable with Trikha's [7] method. The other, a general procedure for determining approximations to weighting functions, enables it to be used at indefinitely small times. The method is applicable to both laminar and turbulent flows.

Most of unsteady flow models with unsteady friction have been tested in simple pipe systems, typically a valve-reservoir system. Recently, some attempts have been done to apply them in pipe networks. In the present work a mathematical model has been established to simulate water hammer oscillations in single pipe line. A new computer program has been devolved to solve this model.

## 2. Mathematical model

A mathematical model to simulate water hammer in one dimensional single pipe line has been developed. This is based on schematic diagram shown in Figure (2). The numerical set up for the present work is described in figure (3).The principles of conservation of mass and momentum are used in the present model. The following assumptions were considered:

1. Incompressible flow and constant viscosity
2. The convective terms are not considered
3. The second order spatial derivative term is not considered.
4. The velocities in the radial direction can be neglected.

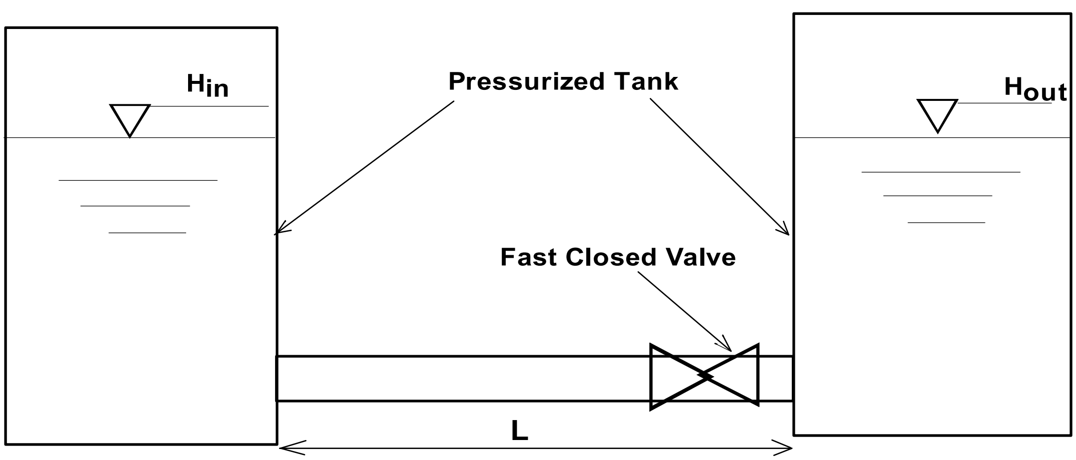


Figure (2): Present work schematic diagram

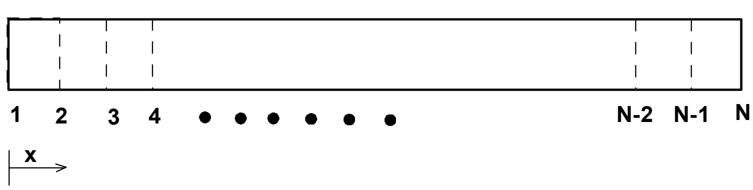


Figure (3): Numerical set up

The mathematical model used in this paper is as follows:

**Continuity Equation**

$$\frac{\partial p}{\partial t}+V\frac{\partial p}{\partial x}+\rho a^2\frac{\partial V}{\partial x}=0 \qquad (1)$$

Where: $p$ = pressure intensity, $V$ = mean flow velocity, $a$ = wave speed, $\rho$ = fluid density, $t$ = time, $x$ = coordinate axis along conduit length.

**Momentum Equation**

$$\frac{\partial V}{\partial t}+V\frac{\partial V}{\partial x}+\frac{1}{\rho}\frac{\partial p}{\partial x}+g\sin\theta+\frac{fV|V|}{2D}=0 \qquad (2)$$

Where: $g$ = acceleration due to gravity, $\theta$ = pipe angle with respect to the horizontal $D$ = pipe diameter, $f$ = Darcy-Weisbach friction factor.

In most engineering applications, the convective acceleration terms, ( $V\frac{\partial p}{\partial x}$ ) and ( $V\frac{\partial V}{\partial x}$ ), are very small compared to the other terms and may be neglected. Therefore by dropping these terms Equation (1 ) becomes:

$$\frac{\partial p}{\partial t}+\rho a^2\frac{\partial V}{\partial x}=0 \qquad (3)$$

,and Equation (2) becomes:

$$\frac{\partial V}{\partial t}+\frac{1}{\rho}\frac{\partial p}{\partial x}+g\sin\theta+\frac{fV|V|}{2D}=0 \qquad (4)$$

Expressing pressures in the pipeline in terms of the piezometric head, above a specified datum, and using the discharge, Q = VA, instead of the flow velocity and considering the energy dissipation factor in the equation of momentum. Equation (3) becomes:

$$\frac{\partial H}{\partial t}+\frac{a^2}{gA}\frac{\partial Q}{\partial x}=0 \qquad (5)$$

,and equation (4) is modified as:

$$\frac{\partial Q}{\partial t}+gA\frac{\partial H}{\partial x}+e_f\frac{fQ|Q|}{2DA}=0 \qquad (6)$$

Where:

Q = flow discharge, $e_f$ = energy dissipation factor which is a non-dimensional and time dependent parameter to compute the variation of the friction losses in space and time. A = pipe cross sectional area, H = pressure static head, f = Darcy-weisbach friction factor, This parameter is set equal to one if the quasi-steady approach in transient pipe flow is used.

The previous partial differential equations transforms into ordinary differential equations along characteristics line. Equations5 and 6 are presented as the following finite difference equations for pressure head H and discharge Q:

$$H_i^{j+1}=H_i^j-\frac{\Delta t a^2}{gA\Delta x}\left(Q_{i+1}^j-Q_i^j\right) \qquad (7)$$

$$Q_i^{j+1}=Q_i^j-\frac{\Delta t gA}{\Delta x}\left(H_{i+1}^j-H_i^j\right)+\frac{\Delta t f Q_i^j\left|Q_i^j\right|}{2DA} \qquad (8)$$

The initial boundary conditions for the above governing equations, which relate to one dimensional flow in a single pipe line with length "L" and diameter "D" and subjected to sudden valve closure, are imposed as in the following description.

**Initial condition:**

The calculation starts from steady state conditions towards transient conditions, so the initial condition refers to steady state conditions:

$$Q_i^0=\sqrt{\frac{\left(H_{in}-H_{out}\right)\left(2gdA^2\right)}{fL}} \qquad (9)$$

$$H_i^0=H_{in}-\frac{H_{in}-H_{out}}{L}(i-1)\Delta x\,, \quad (i=1,2,3,\ldots\ldots\ldots\ldots,N) \qquad (10)$$

**Boundary condition:**

$$At\ x=0,\quad Q_1^{j+1}=Q_1^j-\frac{\Delta t gA}{\Delta x}\left(H_2^j-H_1^j\right)+\frac{\Delta t f Q_1^j\left|Q_1^j\right|}{2DA},\quad H_1^{j+1}=H_{in}$$

$$At\ x=L,\quad Q_L^{j+1}=0,\quad H_L^{j+1}=H_L^j-\frac{\Delta t a^2}{gA\Delta x}\left(Q_{L-\Delta x}^j-Q_L^j\right)$$

**3. Program Algorithm**

The following steps describe the program algorithm:

1. Start program,
2. Read the following input data :
   - Length of the pipeline.
   - Diameter of the pipeline.
   - Inlet level and outlet level.
   - Frictional coefficient.

- Wave velocity.
- .Total number of nodes.
- Time of finishing calculation.

3. Compute time step and cell width.
4. Calculate steady state and initial conditions.
5. Calculate transient state and boundary conditions.
6. Output data for a certain node.
7. If the time less than time of finishing calculation go to step 5 or go to step 8.
8. Output the variation of the head and flow rate with the change in pipe length.
9. End program.

**4. Results and discussion**

The numerical results obtained by Li .Jinping et al [8] was used for the verification of the mathematical model which is generalized in a single pipe system with vale fast closure. The pipe length is 600m, with diameter 1m. The results will be compared to verify the validation of the simulation of water hammer. The initial condition is that, the upstream pressure is 150m, and the downstream pressure is about 143.45m. The acceleration of gravity is 9.806m/s$^2$ and the velocity of the water hammer wave is 1200m/s. During the simulation, the friction is ignored, that is to say, taking the water as an inviscid flow. Boundary conditions of the numerical simulation are taken as described in the previous section.

Figure (4) shows the pressure fluctuation at the outlet of pipe. Fair agreement between the present model results and the numerical results obtained by Li Jinping et al [8] for pressure fluctuation at the outlet of pipe with time is noticed.

Figure (5) shows a comparison between the result of present model and results obtained by Li Jinping et al [8] for average cross-section velocity at the inlet. Good agreement in direction and magnitude for fluctuation of average cross-section velocity at the inlet with time is observed.

Figure (6) shows the present work results for the change of pressure distribution at the last 25 meter at the pipe line end with different time at (0, 1, 2, 3,4,5,6, and 7 seconds). The pressure fluctuation at the outlet of pipe line is cleared. From the pressure fluctuation at the outlet of pipe, we can know that, the maximum and minimum water hammer pressure is very consistent at the first propagation stage of water hammer wave. But with the time increasing (the water hammer is propagating), there has occurred a slight distortion when pressure fluctuating pulse maintain at maximum and minimum water hammer pressure.

Figure (7) shows the change in water flow discharge at the last 25 meter in the pipe line end with different time at (0, 1, 2, 3,4,5,6, and 7 seconds). At the beginning the flow rate is constant on the whole conduit. The water hammer starts after one second of valve sudden closure and the flow change its magnitude and direction. Consequently the fluctuation in direction and magnitude increases.

The present work model results are in good agreement with the results obtained by Li Jinping et al [8].

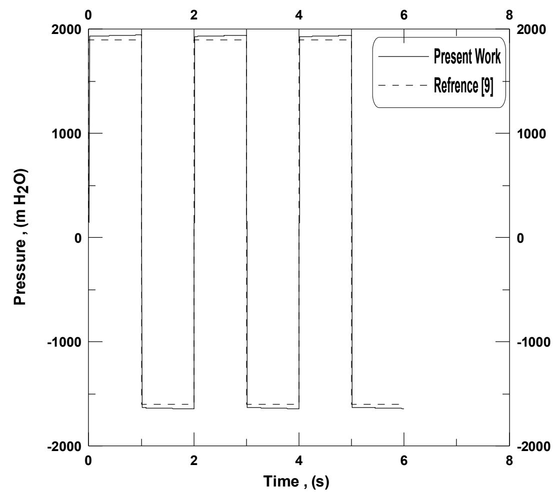


Figure (4): Pressure fluctuations at the pipe line outlet

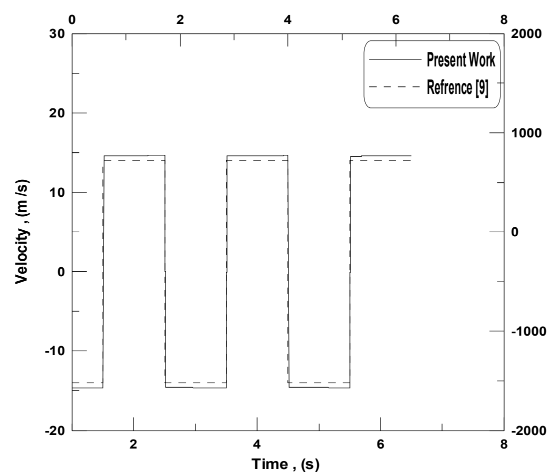


Figure (5): Water velocity fluctuations at the inlet

**Present model output**

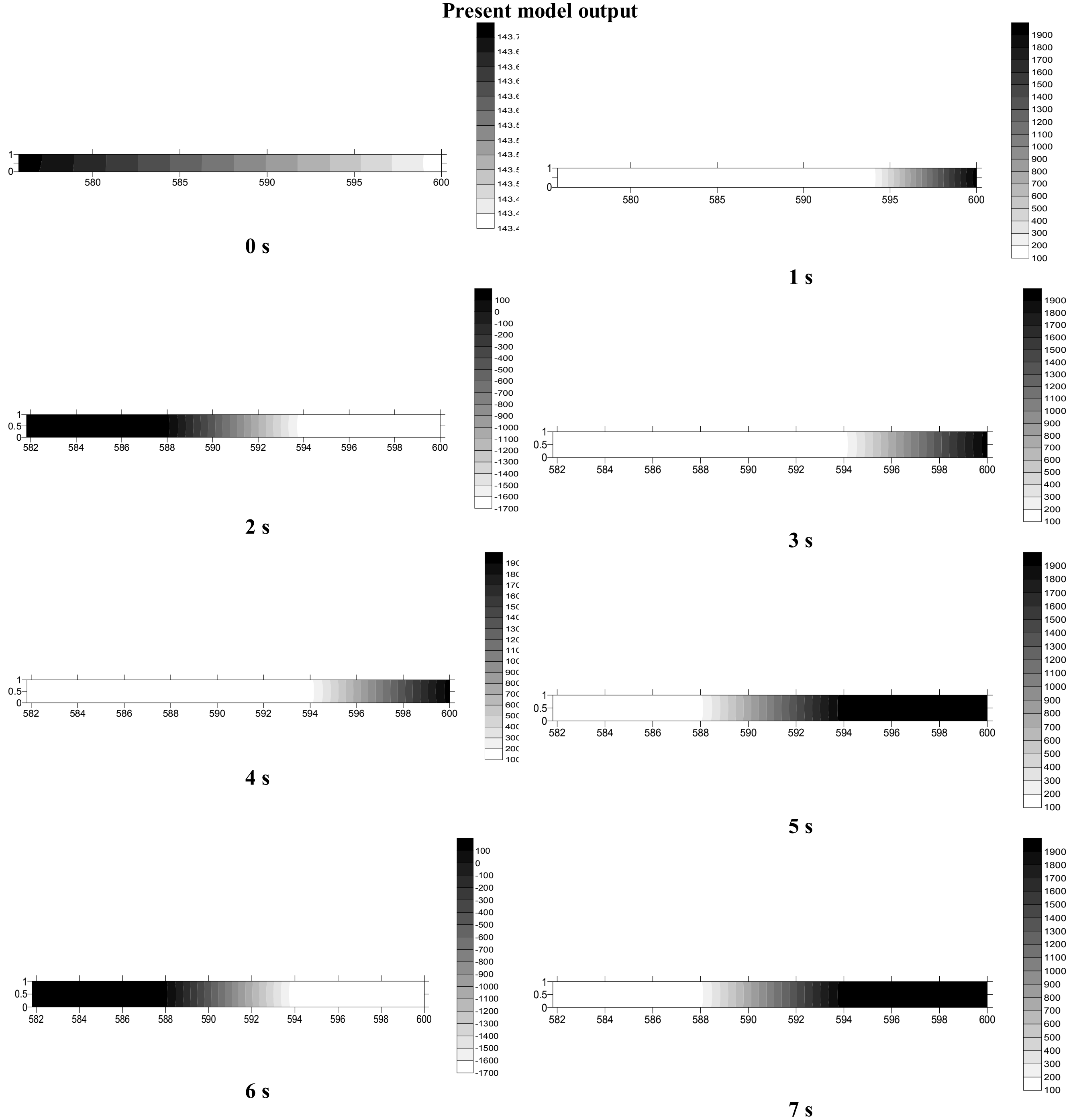


Figure (6): The change of pressure distribution at the last 25 meter at the pipe line end with different time

**Present model output**

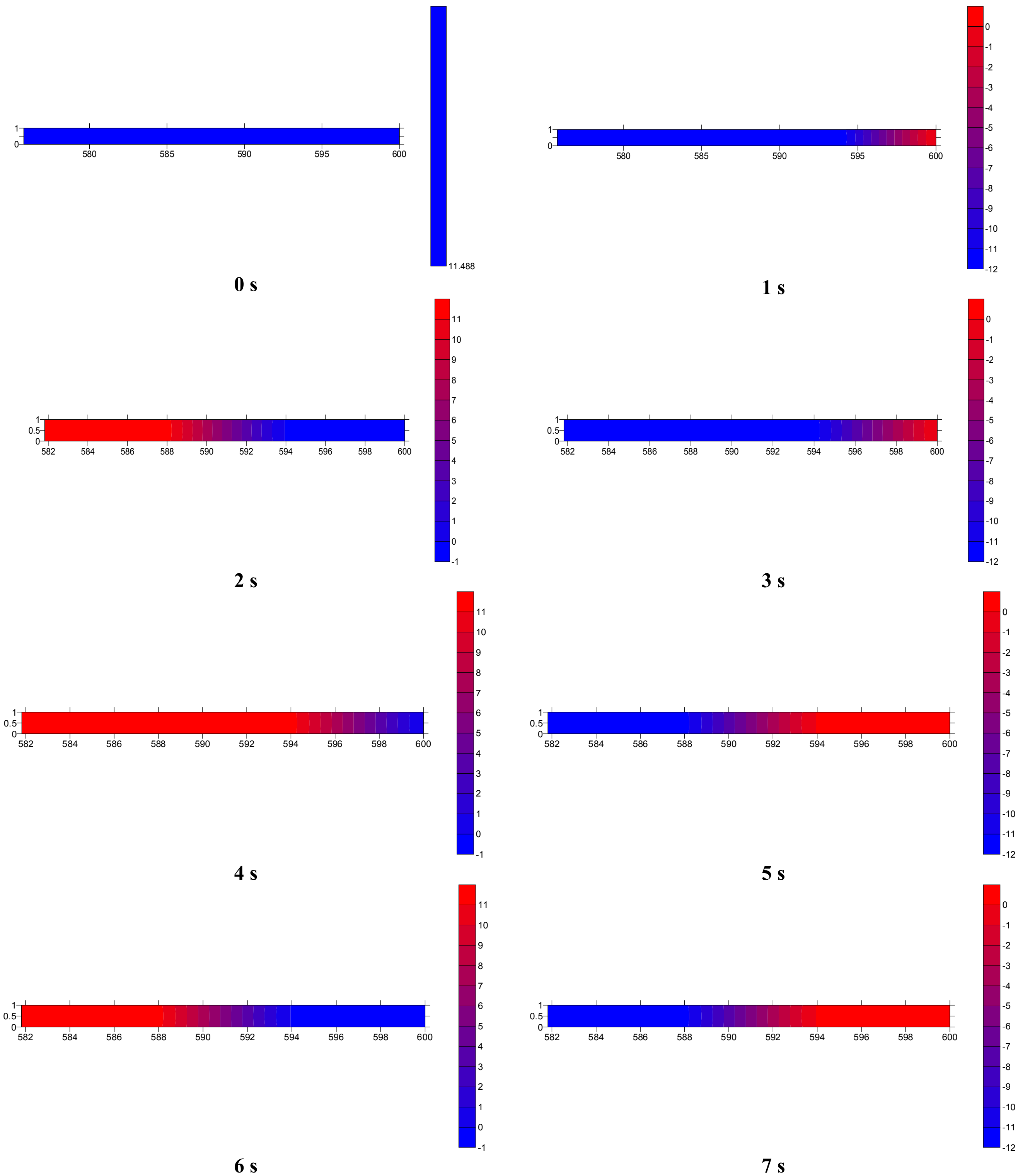


Figure (7): The change in water flow discharge at the last 25 meter at the pipe line end with different time

## 5. Conclusion

The present work creates a mathematical modeling and numerical simulation of water hammer in single pipe line when valve fast close. A new FORTRAN program called HAM01 was developed to achieve the present work goal. The program is used to predict the discharge and pressure distribution in single pipe line. The results of the modeling and simulation may be briefly summarized as:

1. The present work model can be used for simulation of water hammer in single pipe line when valve fast close.
2. The present model describes the pressure fluctuation and the discharge fluctuation through the pipe line at different time.

**Nomenclature**

| Symbo | Definition | Dimension |
|---|---|---|
| l | Pipe cross sectional area | $m^2$ |
| A | Wave speed | m/s |
| a | Pipe diameter | m |
| D | Darcy-Weisbach friction factor | - |
| f | Acceleration due to gravity | $m/s^2$ |
| g | Pressure static head | m |
| H | Index indicate x-direction | - |
| i | Time step index | - |
| j | Total conduit length | m |
| L | Total node number | - |
| N | Pressure intensity | $N/m^2$ |
| P | Flow discharge | $m^3/s$ |
| Q | Time | s |
| t | Mean flow velocity | m/s |
| V | Coordinate axis along conduit | m |
| x | length | $kg/m^3$ |
| ρ | Fluid density . | degree |
| Θ | Pipe angle with respect to the | |
| | horizontal | |

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