# EFFECT OF PIPES NETWORKS SIMPLIFICATION ON WATER HAMMER PHENOMENON

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## ABSTRACT

Simplification is an important design step of water supply and irrigation pipes networks. It is recognized by making the original network easier to be understood and analyzed. Water hammer in water-supply networks may give rise to high and low pressures, due to the superposition of reflected pressure waves. The effect of pipes networks' simplification on water hammer phenomenon is investigated. This study uses a simple two loops pipes network composed of 12 high density polyethylenes (HDPE) pipes with different diameters, thicknesses, and roughness coefficients representing of a general parallel/series system. The network is fed from a boundary head reservoir and loaded by either distributed or concentrated boundary water demands. According to both hydraulic and hydraulic plus water quality equivalence, three levels of simplifications on the original network are performed. Also, the effect of water demands' concentration on the transient flow is checked. The transient flow in the network is initialized by either concentrated or distributed boundary water demands which are suddenly shut-off or released. Water hammer and mass oscillation (WHAMO) software which uses the implicit finite difference scheme for solving the momentum and continuity equations at unsteady-state case is used in the simulation. All scenarios produced results showed that both hydraulic equivalence and demands' concentration simplifications increase the transient pressure and flow rate in the simplified network compared with the original one. However, hydraulic plus water quality equivalence simplification results in an adverse effect. It was found that, as the degree of simplification increases the transient pressure head and flow rate of the simplified network deviate more from those of the original network. Therefore, simplifications of the distribution networks should be done with very careful caution.

**KEYWORDS:** Water hammer; pipes network; simplification; demands variations.

# 1.  INTRODUCTION

The potable water distribution system is one of the most significant hydraulic engineering accomplishments. Potable water can be delivered to water users through distribution systems. However, variable water demands and water usage patterns can produce significant variations of pressure in the distribution system, especially when

the changes are sudden. Sudden changes of water demands can create transient flow that could make so many undesirable consequences such as backflow, negative pressure, or excessive high pressure. Therefore, it is important for engineers to explore the various transient flow effects and to develop the emergency response strategies in order to minimize the negative impacts (Kwon [11]). The total force acting within a pipe is obtained by summing the steady-state and transient pressures in the line. The severity of transient pressures must be accurately determined so that water mains can be properly designed to withstand these additional loads (Jung et al. [9]). Many researchers studied the water hammer phenomenon along the last decades with different viewpoints, among of them Abd el-Gawad [1], Ali et al. [2], Jönnsson [8], Stephenson [18], Yang [19], and many others.

Al-Khomairi [3] discussed the use of the steady-state orifice equation for the computation of unsteady leak rates from pipes through crack or rapture. It has been found that the orifice equation gives a very good estimation of the unsteady leak rate history for normal leak openings. Fouzi and Ali [7] studied water hammer in gravity piping systems due to sudden closure of valves, using both the most effective numerical methods for discretizing and solving the problem; the finite difference method using WHAMO program and the method of characteristics with software AFT impulse. They showed that pressure fluctuations vary dangerously especially in the case of pipes which has variable characteristics (section changes with a divergence, a convergence or a bifurcation). Jung et al. [10] studied the effect of pressure-sensitive demand on transient pressure. They concluded that a pressure-sensitive demand formulation should be used for surge analysis to adequately evaluate both system performance and the ultimate cost of system protection.

Mohamed [12] introduced the effect of different parameters such as time of valve closure, pipes' material rigidity, and pipes roughness on the transient pressure damping. It was found that the pipe friction factor and the closing time of the valve have a significant effect on the transient pressure reduction and the elastic pipes such as PVC are better than rigid pipes in pressure damping.

Ramos et al. [15] carried out several simulations and experimental tests in order to analyze the dynamic response of single pipelines with different characteristics, such as pipes' material, diameters, thicknesses, lengths and transient conditions. They concluded that being the plastic pipe with a future increasing application, the viscoelastic effect must be considered, either for model calibration, leakage detection or in the prediction of operational conditions (e.g. start up or trip-off electromechanical equipment, valve closure or opening).

Samani and Khayatzadeh [16] employed the method of characteristics to analyze transient flow in pipe networks. They applied various numerical tests to examine the accuracy of these methods and found that the method in which the implicit finite difference was coupled with the method of characteristics to obtain the discretized equations is the best compared to the others.

According to the aforementioned studies, water hammer in pipes networks has been studied from different viewpoints. However, each water supply network has its own special characteristics which make it different from other networks. Also, due to the lack of field measurements which are costly, it becomes important to use numerical models to gain an indication about the behavior of networks under transient effects.

This study aims to investigate the effect of the hydraulic equivalence, hydraulic plus water quality equivalence, and demands' concentration simplifications of pipes networks on the transient pressure head and flow rate induced from sudden demands shut-off or release.

# 2. THEORETICAL CONSIDERATIONS

Because of the difficulty in solution of water hammer governing equations, engineers in pipelines design may neglect this phenomenon. Recently a number of numerical methods which may be used to solve these equations and suitable for digital computer analyses have been reported in the literature (Chaudhry and Yevjevich [5]).

## 2.1 Governing Equations for Unsteady Flow in Pipelines

The governing equations for unsteady flow in pipeline are derived under the following assumptions including; (1) one dimensional flow i.e. velocity and pressure are assumed constant at a cross section; (2) the pipe is full and remains full during the transient; (3) no column separation occurs during the transient; (4) the pipe wall and fluid behave linearly elastically; and (5) unsteady friction loss is approximated by steady-state losses.

The unsteady flow inside the pipeline is described in terms of unsteady mass balance (continuity) equation and unsteady momentum equation, which define the state of variables of $V$ (velocity) and P (pressure) given as Simpson and Wu [17];

$$\frac{\partial \rho}{\partial t} + V \frac{\partial \rho}{\partial x} + \rho \frac{\partial V}{\partial x} + \frac{\rho}{A} \frac{dA}{dt} = 0 \tag{1}$$

$$\frac{\partial V}{\partial t} + V \frac{\partial V}{\partial x} + \frac{1}{\rho} \frac{\partial P}{\partial x} - g \sin \alpha + \frac{f|V|V}{2D} = 0 \tag{2}$$

Where $x$ = distance along the pipeline; $t$ = time; $V$ = velocity; $P$ = hydraulic pressure in the pipe; $g$ = acceleration due to gravity; $f$ = Darcy-Weisbach friction factor; $\rho$ = fluid density; $D$ = pipe diameter; $\alpha$ = pipe slope angle, and $A$ = cross sectional area of the pipe.

Equation (1) is the continuity equation and takes into account the compressibility of water and the flexibility of pipe material. Equation (2) is the equation of motion. In Eq.(1), the terms $\frac{1}{\rho}\left(\frac{\partial \rho}{\partial t} + \frac{\partial \rho}{\partial x} V\right)$ are replaced by equivalent $\frac{1}{\rho} \frac{d\rho}{dt}$ where $V = \frac{dx}{dt}$, $\frac{d\rho}{dt} = \frac{\rho}{K} \frac{dP}{dt}$, and $K$ is the bulk modulus of the fluid. Also, the fourth term in Eq.(1) can be expressed as $(1 - v^2)\frac{\rho D}{eE} \frac{dP}{dt}$, where $v$ is the poison's ratio of the pipe, $e$ is the pipe wall thickness and $E$ is the Young's modulus of elasticity of the pipe. Substitution by these abbreviations in Eq.(1), it can be reduced to the following formula;

$$\frac{dP}{dt}\left[\frac{1}{K}+\left(\frac{1-v^2}{E}\right)\frac{D}{e}\right]+\frac{\partial V}{\partial x}=0 \tag{3}$$

Wave speed can be defined as the time taken by the pressure wave generated by instantaneous change in velocity to propagate from one point to another in a closed conduit. Wave speed (c) can be expressed as;

$$\frac{1}{\rho c^2}=\left[\frac{1}{K}+\left(\frac{1-v^2}{E}\right)\frac{D}{e}\right]=\frac{1}{K}\left[1+\frac{Kc_1D}{Ee}\right] \tag{4}$$

Where: $c1 = (1 - v2)$. Substitution by Eq.(4) in Eq.(3) and dividing the result by $\gamma$ yields;

$$\left[\frac{\partial H}{\partial t}+\frac{\partial H}{\partial x}V\right]+\frac{c^2}{g}\frac{\partial V}{\partial x}=0 \tag{5}$$

Where $H$ is the piezometric head, i.e. pressure head plus the elevation head. The term $\frac{\partial H}{\partial x}$ is small compared to $\frac{\partial H}{\partial t}$ and it is often neglected. Thus the simplified form of the continuity equation in terms of discharge, Eq.(5) becomes;

$$\frac{\partial H}{\partial t}+\frac{\partial Q}{\partial x}\frac{c^2}{gA}=0 \tag{6}$$

By the same way, the momentum equation, i.e. Eq.(2) can be simplified and written in terms of discharge and piezometric head as follows;

$$\frac{\partial H}{\partial x}+\frac{1}{gA}\frac{\partial Q}{\partial t}+\frac{fQ|Q|}{2gDA^2}=0.0 \tag{7}$$

## 2.2 Implicit Finite Difference Solution Method

The continuity and momentum equations form a pair of hyperbolic, partial differential for which an exact solution cannot be obtained analytically. However other methods have been developed to solve water hammer equations. If the equations are hyperbolic, it means the solutions follow certain characteristic pathways. For water hammer equations, the wave speed is the characteristic. The implicit finite difference method is a numerical method used for solving water hammer equations. The implicit method replaces the partial derivatives with finite differences and provides a set of equations that can then be solved simultaneously. The computer program WHAMO uses the implicit finite-difference technique but converts its equations to a linear form before it solves the set of equations (Fitzgerald and Van Blaricum, [6]).

The solution space is discretized into the *x-t* plane, so that at any point on the grid *(x, t)* there is a certain H and Q for that point, *H(x, t)* and *Q(x, t)* as shown in **Fig. (1)**.
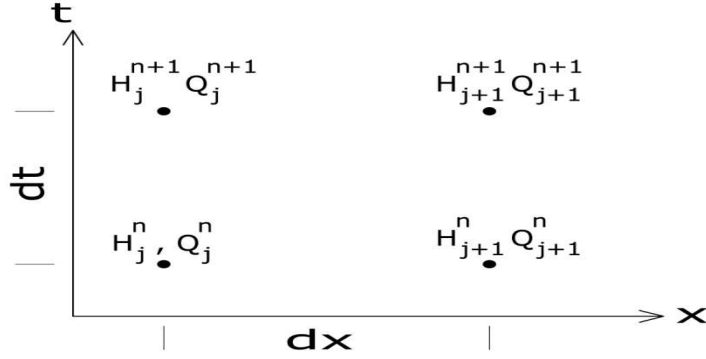
Fig. 1. The finite difference grid.

The momentum equation and the continuity equation can be represented in a short form by introducing the following coefficients for the known values in a system;

$$\alpha_j = \frac{2\Delta t c_j^2 \theta}{g A_j \Delta x_j} \tag{8}$$

$$\beta_j = (H_{n,j+1} + H_{n,j}) + \frac{(1-\theta)}{\theta} \alpha_j (Q_{n,j} - Q_{n,j+1}) \tag{9}$$

$$\gamma_j = \frac{\Delta x_j}{2g\theta A_j \Delta t} \tag{10}$$

$$\delta_j = \frac{(1-\theta)}{\theta}(H_{n,j} - H_{n,j+1}) + \gamma_j(Q_{n,j} + Q_{n,j+1})$$
$$-\frac{\Delta x_j f_j}{4g\theta D_j A^2{}_j}(Q_{n,j}|Q_{n,j}| + Q_{n,j+1}|Q_{n,j+1}|) \tag{11}$$

Where $\theta$ is a weighing factor included for numerical stability. All parameters for the coefficients should be known from the properties of the pipe or the values of head and flow at the previous time step. With the coefficients, the momentum and continuity equations of the $j^{th}$ segment of the pipe become as given by Batterton [4] as follows;

Momentum: $-H_{n,j+1} + H_{n+1,j+1} + \gamma_j(Q_{n+1,j} + Q_{n+1,j+1}) = \delta_j$ (12)

Continuity: $H_{n,j+1} + H_{n+1,j+1} + \alpha_j(Q_{n+1,j+1} - Q_{n+1,j}) = \beta_j$ (13)

Now, with equations for the all links and nodes in the system, the initial and boundary conditions, a matrix of the linear system of equations can be set up to solve for head and flow everywhere, simultaneously, for the first time step. The process is repeated for the next time step, and again for the next step until the specified end of the simulation.

## 3. APPLICATIONS

The simple pipes network shown in **Fig. (2)** consisting of 11 joints (J1~J11) and 12 high density polyethylene (HDPE) pipes (C1~C12) at the same elevation is representative of a general parallel/series system. HDPE pipes with their common low Young's modulus were preferred in this application to avoid negative pressure waves to drop to the saturated vapor pressure of the water which form a cavity in the fluid as the simulation program (WHAMO) does not allow for the effect of cavitation. Other strong pipes networks with high Young's modulus must employ systems to help control increase and decrease in pressure due to water hammer. The Young's modulus for the HDPE material and water were taken as 0.80 and 2.20 GPa, respectively. Joint J10 only has a boundary concentrated demand of 126 L/s and the network is fed by a reservoir with a boundary head of 59 m. Each pipe from C1 to C12 has a circular cross section. **Table (1)** gives lengths, diameters and Darcy-Weisbach friction factors (*f*) for all pipes in the network. The thicknesses of the pipes' walls were taken according to their diameters to suit for a working pressure of 10 bars.
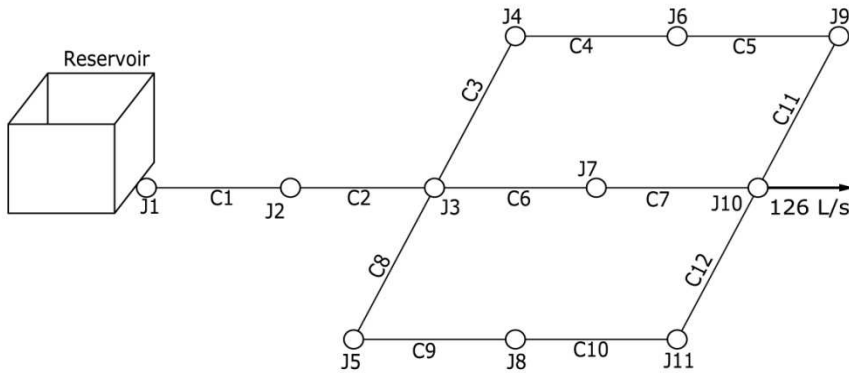


Fig. 2. A Simple pipes network (the original network).

**Table 1. Lengths, diameters and friction factors for all pipes of the original network.**

| Pipe ID | Length (m) | Diameter (mm) | Darcy-Weisbach friction factor (*f*) |
|---------|------------|----------------|----------------------------------------|
| C1 | 305 | 300 | 0.026 |
| C2 | 305 | 300 | 0.026 |
| C3 | 305 | 300 | 0.024 |
| C4 | 305 | 200 | 0.020 |
| C5 | 305 | 150 | 0.018 |
| C6 | 305 | 200 | 0.020 |
| C7 | 305 | 150 | 0.018 |
| C8 | 305 | 250 | 0.022 |
| C9 | 305 | 200 | 0.020 |
| C10 | 305 | 150 | 0.018 |
| C11 | 305 | 300 | 0.024 |
| C12 | 305 | 250 | 0.022 |

# 4. SIMPLIFICATION METHODS

## 4.1 The Hydraulic Equivalence Simplification Method

Using conservation of energy across a set of pipes in parallel or series, equivalent pipes relationships can be derived. Since these relationships are developed from conservation of energy, the equivalent pipes have consistent flow and pressure losses as the original set of pipes. Typically, an equivalent diameter is determined by fixing the equivalent pipe's length and roughness, (Mohamed and Ahmed [13]). Equations (14) and (15) can be used for calculating the hydraulic equivalent diameter for $n$ pipes in series and in parallel, respectively.

$$\frac{1}{D_e^5} = \sum_{i=1}^{n} \frac{1}{D_i^5} \left( \frac{f_i}{f_e} \right) \frac{L_i}{L_e} \tag{14}$$

and

$$D_e = \left[ \sum_{i=1}^{n} \left( \frac{f_e}{f_i} \right)^{0.5} \left( \frac{L_e}{L_i} \right)^{0.5} D_i^{2.5} \right]^{0.4} \tag{15}$$

Where $f_i$, $D_i$ and $L_i$ are the Darcy-Weisbach friction factor, diameter, and length of the pipe $i$ in series or parallel and $f_e$, $D_e$ and $L_e$ are the same parameters for the hydraulic equivalent system. By fixing two of the three parameters, the third can be determined using a form of the above equations.

## 4.2 The Hydraulic and Water Age Equivalence Simplification Method

In general, water quality has an adverse relation with its age thus the travel time of water in pipes could be used to indicate its quality. Raczynski et al. [14] developed the following equation for computing the water age equivalent diameter, $D_{ew}$.

$$D_{ew} = \left[ \frac{\sum_{i=1}^{n} \left( D_i^2 L_i \right)}{L_e} \right]^{0.5} \tag{16}$$

$D_{ew}$ ensures that the travel time in the equivalent pipe will equal that of the series or parallel pipes. However, it does not ensure that the system will be hydraulically equivalent. Since Eq. (16) shows that $D_{ew}$ is independent on $f$, it is possible to find an equivalent hydraulic system without affecting the travel time equivalence by modifying the pipe roughness. To do so, rather than solving for $D_e$ for a defined value of $f_e$ in hydraulic equivalence equations Eqs.(14) and (15), $D_e$ is set to $D_{ew}$, and $f_e$ is solved for as an unknown term.

## 4.3 Demands' Concentration Simplification Method

Simplification is an important primary design step of water supply and irrigation pipes networks. In most real networks, the demands leave through most parts of the pipe lines. However, an indispensible design step of pipes networks is the demands' concentration. The demands that leave the network at distributed locations over its pipes' lengths are replaced with equivalent concentrated demands that reallocated at the joints of the simplified network.

## 4.4 Applied Simplifications on The Used Network

The effect of the equivalence simplification methods are evaluated for the used pipes network with different three levels of simplifications (aggregations or skeletonizations) and demands' concentration. The simplification is loosely defined as the removing of pipes and nodes from a network to make the model simpler. In this study, the aggregation simplifies the system by replacing a series or parallel set of pipes with a single pipe. The first level of simplification (Level 1) as shown in **Fig. (3-a)** aggregates the two series pipes between nodes (J4~J9), (J3~J10), and (J5~J11) and removes joints J6, J7, and J8. Since there are no demands at these nodes, no demands reallocation are required. The second level of simplification (Level 2) is shown in **Fig. (3-b)** which aggregates the upper and lower series pipes between nodes J3 and J10 to only one pipe on each upper and lower side. The third level of simplification (level 3) as shown in **Fig. (3-c)** replaces the three parallel pipes of level 2 with a single pipe. **Table (2)** shows the calculated properties of the pipes of the simplified network for the three levels of simplifications according to both hydraulic and hydraulic plus water quality equivalence. The other simplification type is performed only on the original network, which includes concentrating distributed demands with a value of 14 L/s loaded on 9 nodes (J3~J11) to be at the end node (J10) with a total concentrated equivalent value of 126 L/s.

# 5. RESULTS AND DISCUSSIONS

To show the effect of the pipes network simplification on water hammer phenomenon, three scenarios of transient flows on simplified and original networks were simulated and compared. The original network was simplified up to three levels according to both hydraulic and hydraulic plus water quality equivalence, moreover the original network is loaded by either concentrated or distributed water demands at their joints. The transient flow was initialized through linearly and suddenly shut-off or release of concentrated or distributed water demands through a short period of two seconds. WHAMO software which uses the implicit finite difference scheme for solving the momentum and continuity equations at unsteady-state case was used in the simulation.
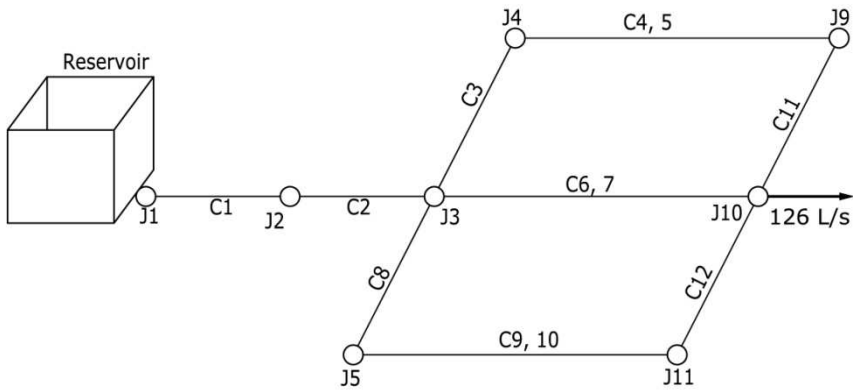
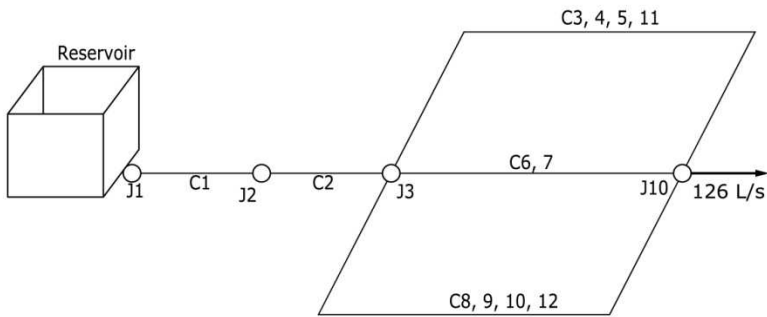Fig. 3-a. The first simplification level.
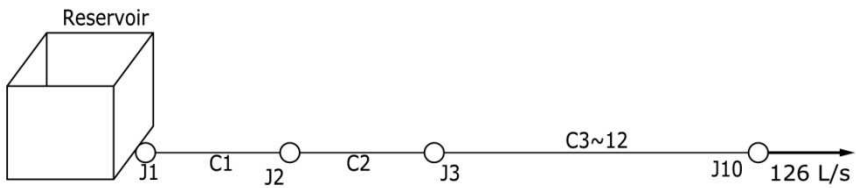


Fig. 3-b. The second simplification level



Fig. 3-c. The third simplification level.

Fig. 3. The different three levels of simplifications for the used pipes network.

**Table 2. Lengths, diameters and friction factors of pipes for the three simplification levels.**

**Level 1**

| Pipe's ID | | | C1 | C2 | C3 | C4, 5 | C6, 7 | C8 | C9,10 | C11 | C12 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | H. Equiv. | L | 305 | 305 | 305 | 610 | 610 | 305 | 610 | 305 | 305 |
| | | D | 300 | 300 | 300 | 164.4 | 164.4 | 250 | 164.4 | 300 | 250 |
| | | f | 0.026 | 0.026 | 0.024 | 0.018 | 0.018 | 0.022 | 0.018 | 0.024 | 0.022 |
| | H. & Q. Equiv. | L | 305 | 305 | 305 | 610 | 610 | 305 | 610 | 305 | 305 |
| | | D | 300 | 300 | 300 | 176.8 | 176.8 | 250 | 176.8 | 300 | 250 |
| | | f | 0.026 | 0.026 | 0.024 | 0.026 | 0.026 | 0.022 | 0.026 | 0.024 | 0.022 |

**Level 2**

| Pipe's ID | | | C1 | C2 | C3, 4, 5, 11 | C6,7 | C8, 9, 10, 12 |
|---|---|---|---|---|---|---|---|
| | H. Equiv. | L | 305 | 305 | 1220 | 610 | 1220 |
| | | D | 300 | 300 | 186.5 | 164.4 | 183.7 |
| | | f | 0.026 | 0.026 | 0.018 | 0.018 | 0.018 |
| | H. & Q. Equiv. | L | 305 | 305 | 1220 | 610 | 1220 |
| | | D | 300 | 300 | 246.2 | 176.8 | 216.5 |
| | | f | 0.026 | 0.026 | 0.072 | 0.026 | 0.041 |

**Level 3**

| Pipe's ID | | | C1 | C2 | C 3 ~ C12 |
|---|---|---|---|---|---|
| | H. Equiv. | L | 305 | 305 | 610 |
| | | D | 300 | 300 | 251.8 |
| | | f | 0.026 | 0.026 | 0.018 |
| | H. & Q. Equiv. | L | 305 | 305 | 610 |
| | | D | 300 | 300 | 504 |
| | | f | 0.026 | 0.026 | 0.536 |

- $L$ = Pipe length (*m*), $D$ = Pipe diameter (*mm*), and $f$ = Friction factor.
- *H. Equiv. = Hydraulic equivalence, H. & Q. Equiv. = Hydraulic plus quality equivalence*.

Three scenarios producing transient flows were performed for both simplified and original networks. In the 1st scenario, a concentrated boundary demand at joint J10 is linearly decreased from 126 L/s to 0 L/s through two seconds period. In the 2nd scenario, a concentrated boundary demand at joint J10 is linearly increased from 0 L/s to 126 L/s through two seconds period. Both the first and second scenarios are applied on the original network and three levels of simplified networks according to both hydraulic and hydraulic plus water quality equivalence. In the 3rd scenario, a concentrated boundary demand of 126 L/s at the end point (J10) was distributed equally with a boundary value of 14 L/s on nine nodes of the original network (J3~J11) and both the concentrated and distributed demands were suddenly shut-off or released through a short period of two seconds. The third scenario was applied only on the original network.

## *5.1 Effect of Suddenly Concentrated Demand Shut-off (1^St scenario)*

To examine the effect of network simplifications on water hammer phenomenon when a concentrated boundary demand is suddenly shut-off, the valve at node J10 was assumed to be linearly closed in a short period of 2 seconds. Before closing the valve, the flow in the network will be at steady-state with the pressure head controlled by friction losses in the pipes, minor losses in the fittings, and the type of the valve and its opened area. When the valve is closed instantaneously the liquid next to the valve comes to a halt. The liquid is then compressed by the liquid upstream which is still flowing. This compression causes a local increase in the pressure of the liquid. The total pressure acting within the pipes equals the summing of the steady-state and the water hammer induced pressure. The walls of the pipes around the fluid are stretched by the resulting excess pressure. A chain reaction then takes place along the lengths of the pipes with each stationary element of fluid being compressed by the flowing fluid upstream. When the pressure wave reaches the reservoir, the fluid in the pipes is now at rest and the pressure cannot exceed the boundary water depth in the reservoir thus water starts to flow out of the pipes into the reservoir. An unloading pressure wave now travels back along the pipes towards the valve. When the unloading wave reaches the valve, the water in the pipes is now flowing out of the pipe into the reservoir, but at the closed valve the water must be at rest. This now causes a negative pressure wave to travel back up the pipes towards the reservoir. When the pressure wave hits the reservoir the flow in the pipes will be at rest, but the pressure head is now below the reservoir level, flow reverses in the pipes and another unloading wave travels back along the pipes towards the valve. A cycle of pressure waves (positive – unloading – negative – unloading) now travels up and down the lengths of the pipes.

The pressure wave travels along the pipes network with a certain velocity, which is called the celerity. The pressure wave's celerity is affected by the modulus of elasticity of fluid and pipes' material, water density, pipes' diameter, and pipes' wall thickness. For instantaneous valve closure the transient increase and decrease in water pressure due to water hammer depend mainly on the celerity of the wave, water density, and water velocity in the pipes under the steady-state conditions.

**Figures (4 to 6)** show the transient pressure head at node J10 due to its suddenly demand shut-off for the different three levels of simplifications compared with that of the original network through a duration of simulation of 100 seconds just after the valve closure. As shown in each figure, for both simplified and original networks the peak pressure values occur in the first cycle thus demonstrate the effect of friction on damping pressure waves. From the figures it is noticeable that, the hydraulic equivalence simplification increases the peak values of the transient pressure head compared with those of the original network, however, the simplification according to hydraulic plus water quality equivalence reduces the peak values.

In comparison between the figures, it is clear that as the level of simplification increases, the transient pressure head of the simplified network deviates more from that of the original network. The figures illustrate that, in the case of the hydraulic equivalence the frequency of the transient pressure waves increases as the simplification level increases. However, the frequencies of the transient pressure waves for the original and hydraulic plus water quality simplified networks are the same. For the hydraulic plus water quality equivalence simplification as the water age is a

controlling parameter, the frequencies of the transient pressure waves for both original and simplified networks should have the same trend. In all cases it is clear that, the simulation period (100 seconds) is not sufficient to achieve the steady-state flow conditions in both simplified and original networks.
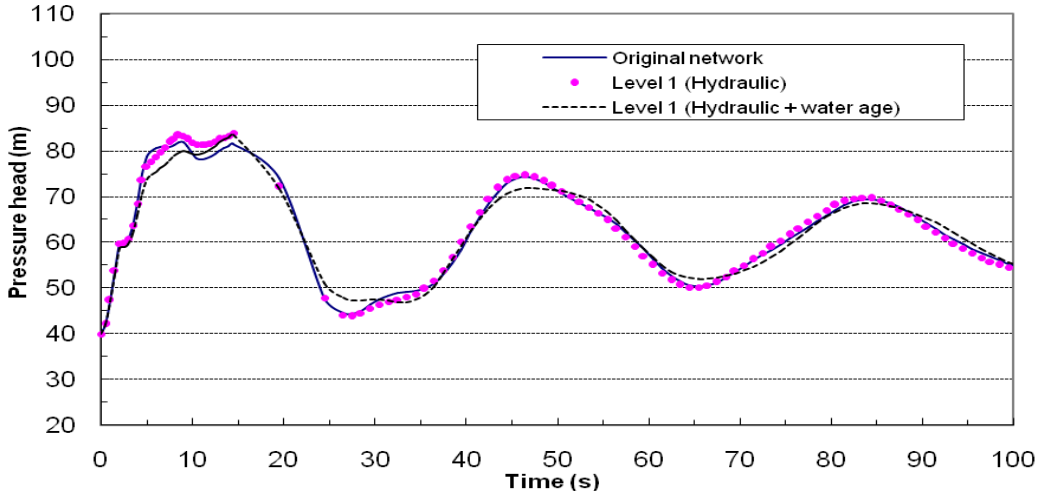


Fig. 4. Transient pressure head for the simplified (level 1) and original networks at node J10 due to its sudden shut-off demand.
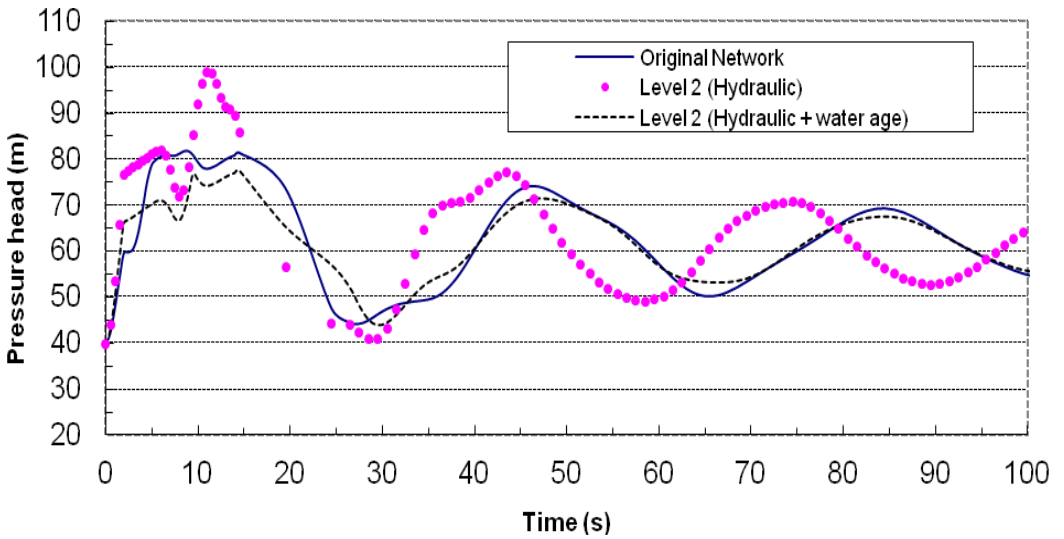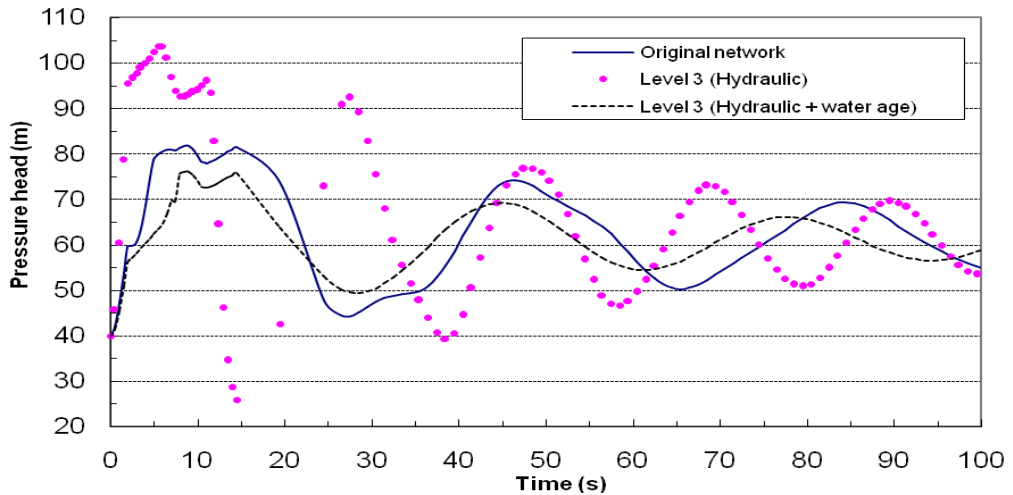


Fig. 5. Transient pressure head for the simplified (level 2) and original networks at node J10 due to its sudden shut-off demand.

Fig. 6. Transient pressure head for the simplified (level 3) and original networks at node J10 due to its sudden shut-off demand.

Just after the valve is suddenly closed, a cycle of pressure waves (positive – unloading – negative – unloading) has been induced. Positive and negative waves start at node J10 while the unloading waves start at the reservoir. At the time of the positive and second unloading pressure waves the direction of the transient flow of the liquid remains towards the valve. Through the first unloading and negative pressure waves the liquid reverses towards the reservoir (backflow).

**Figures (7 to 9)** show the transient flow rate at node J2, as an example, due to suddenly concentrated boundary demand shut-off at node J10 (linearly closed in a short period of 2 seconds) through a duration of simulation of 100 seconds for the different levels of simplifications compared with that of the original network. As shown in the figures at time zero and before closing the valve, the flow was at the steady-state with the boundary flow rate of 126 L/s. From each figure, it can be seen that the hydraulic equivalence simplification increases the peak values of the transient flow rate compared with those of the original network, however, the simplification according to the hydraulic plus water quality equivalence reduces the peak values. In comparison between these figures, it is noticeable that as the level of simplification increases the transient flow rate of the simplified network deviates more from that of the original network.
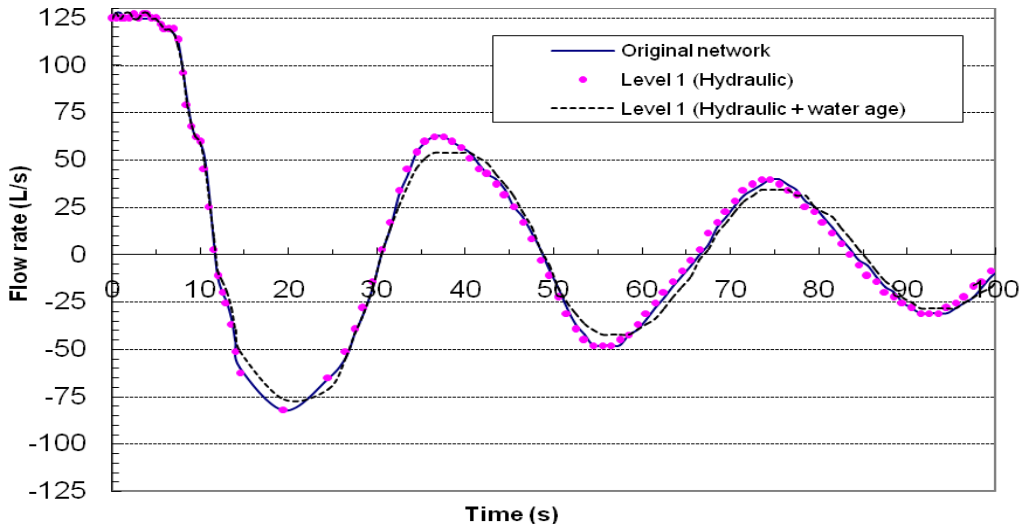
Fig. 7. Transient flow rate at node J2 due to suddenly demand shut-off at node 10 for the simplified (level 1) and original networks.
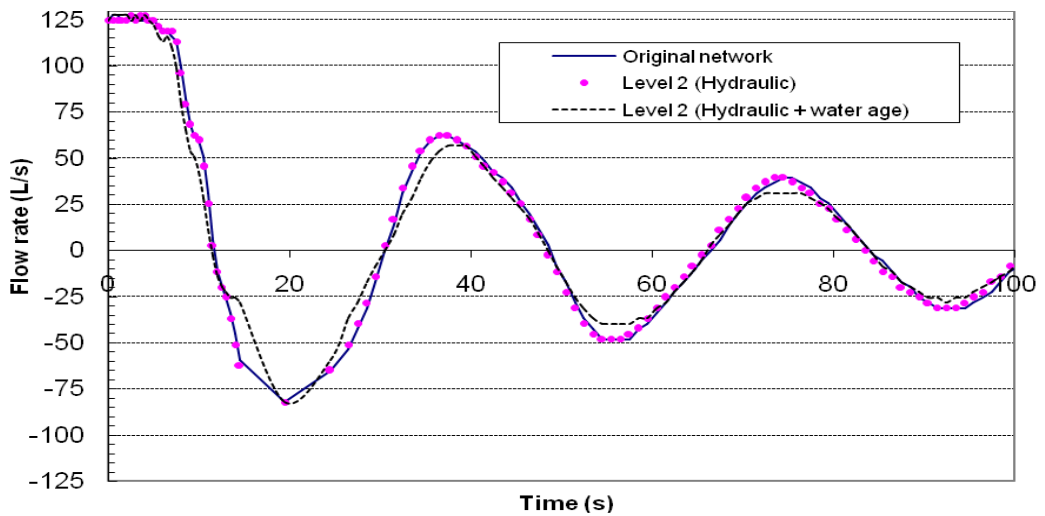


Fig. 8. Transient flow rate at node J2 due to suddenly demand shut-off at node 10 for the simplified (level 2) and original networks.
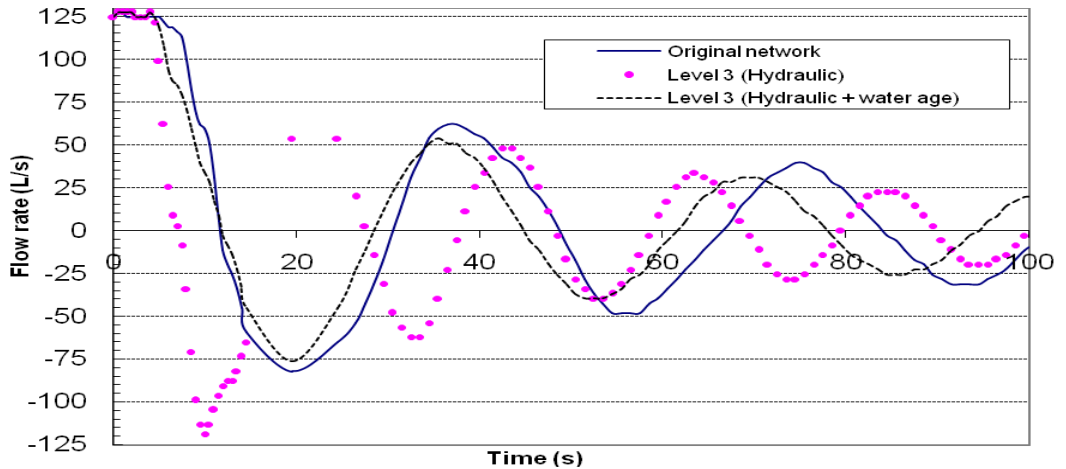
Fig. 9. Transient flow rate at node J2 due to suddenly demand shut-off at node 10 for the simplified (level 3) and original networks.

As shown in **Fig. (9)**, the third level of hydraulic equivalence simplification produces a transient flow rate with waves that have high frequency compared with those of hydraulic plus water quality equivalence simplification, original network, and even others hydraulic equivalence simplification levels.

## 5.2 Effect of Suddenly Concentrated Demand Release (2ⁿᵈscenario)

To show the effect of network simplifications on water hammer phenomenon when a concentrated boundary demand is released suddenly, the valve at node J10 was assumed to be linearly opened in a short period of 2 seconds. When the valve at node J10 is completely closed, there is no-flow in the network; consequently the pressure head through the network equals the water boundary level in the reservoir (59 m). As the valve at node J10 is suddenly opened and the demand is released from 0 to a boundary value of 126 L/s a negative pressure wave travels along the pipes network from node J10 towards the reservoir. A cycle of pressure waves (negative – unloading - positive – unloading) starts to travel up from node J10 towards the reservoir and down from the reservoir to node J10 through the pipes in a successive manner.

**Figures (10 to 12)** illustrate the transient pressure head at node J10 after its suddenly demand release for the three levels of simplifications compared with that of the original network. As shown in each figure, the peaks of the transient pressure waves are gradually damped due to the friction effect. From these figures, it is clear that the transient pressure conditions are damped fast within 50 seconds and the steady-state conditions prevail. From each figure, it is noticeable that the hydraulic equivalence simplification increases the peak values of the transient pressure heads compared with those of the original network, however, the simplification according to hydraulic plus water quality equivalence reduces the peak values. In comparison between these figures, it is clear that as the level of simplification increases, the transient pressure head of the simplified network deviates more from that of the original network.
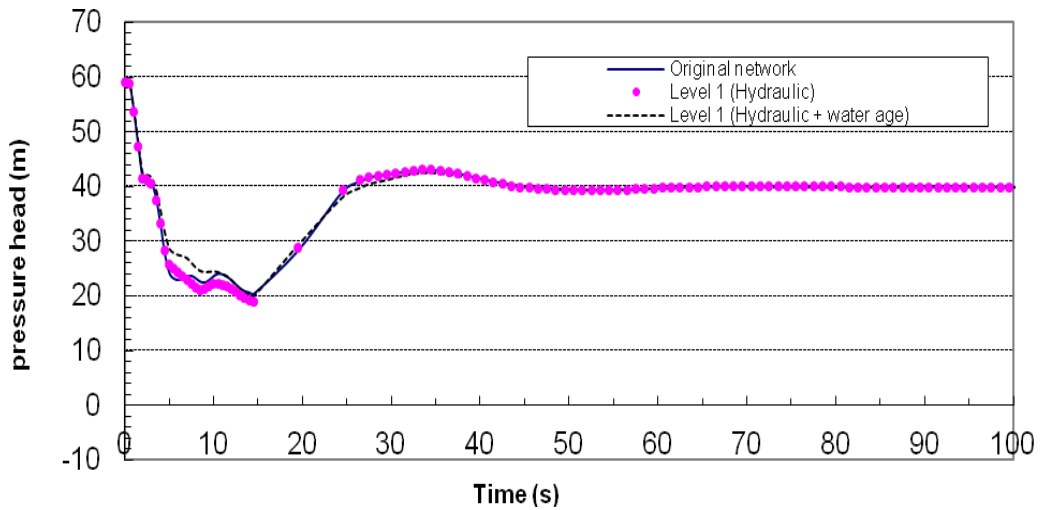
Fig. 10. Transient pressure head at node J10 due to its suddenly demand release for the simplified (level 1) and original networks.
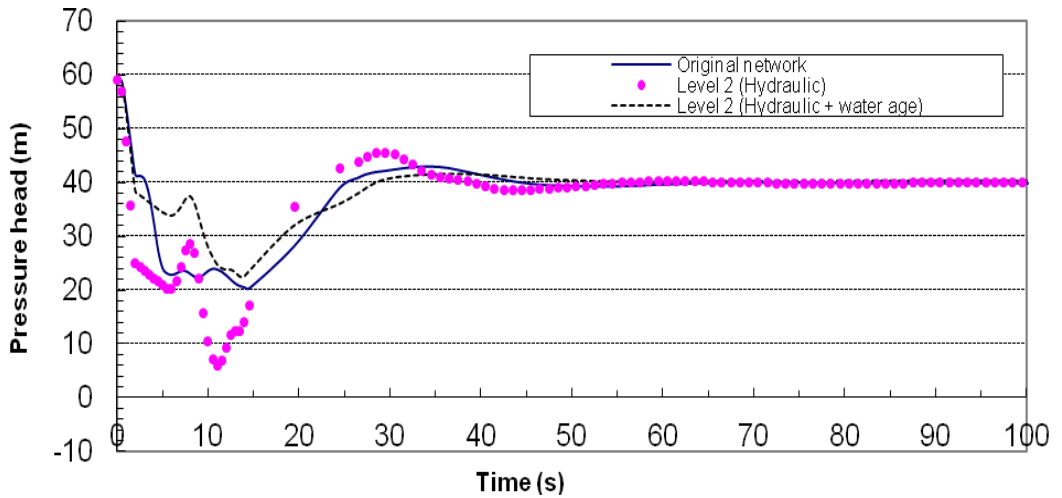


Fig. 11. Transient pressure head at node J10 due to its suddenly demand release for the simplified (level 2) and original networks.
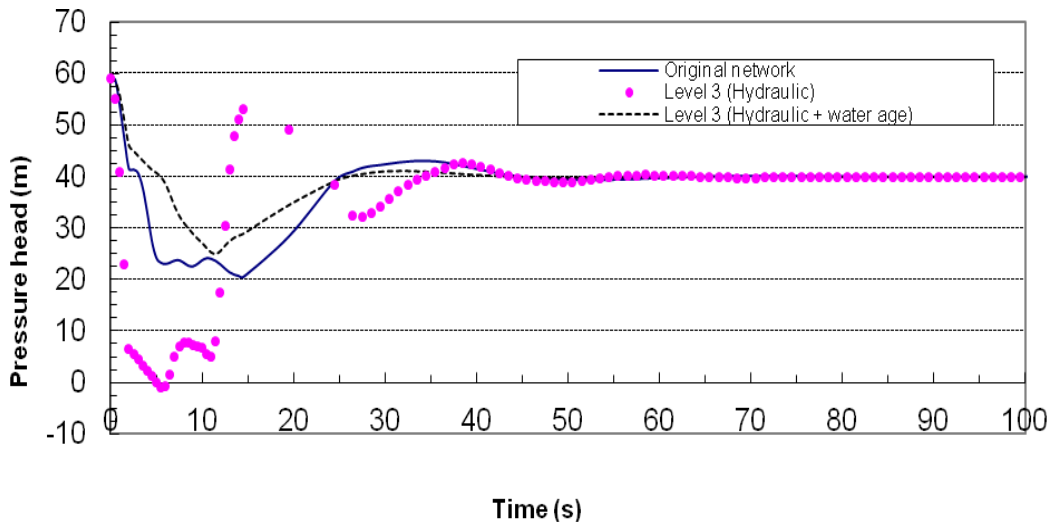
Fig. 12. Transient pressure head at node J10 due to its suddenly demand release for the simplified (level 3) and original networks.

**Figure (12)** illustrates that the frequency of the transient pressure waves for the third level of the hydraulic equivalence simplification is higher than those of the hydraulic plus water quality simplification, original network, and even others hydraulic simplification levels.

**Figures (13 to 15)** show the transient flow rate at node J2, as an example, after releasing the demand at node J10 from zero to a boundary value of 126 L/s linearly in a short period of two seconds for the different three levels of simplifications compared with that of the original network. In comparison between these figures, it is noticeable that as the level of simplification increases the deviation of the transient flow rate from the original case increases. Also, it can be seen that the hydraulic equivalence simplification increases the peak values of the transient flow rate compared with those of the original network, however, the simplification according to hydraulic plus water quality equivalence reduces the peak values. Also, from the figures, it is clear that the transient flow conditions are damped fast within 50 seconds and the steady-state conditions prevail.

In general, **Figs. (10 to 15)** demonstrate that the transient flow rate at node J2 for the different levels of simplifications has an inverse trend to that of the transient pressure head at node J10 with a short time lag which could be attributed to the location's difference between the concerned nodes.
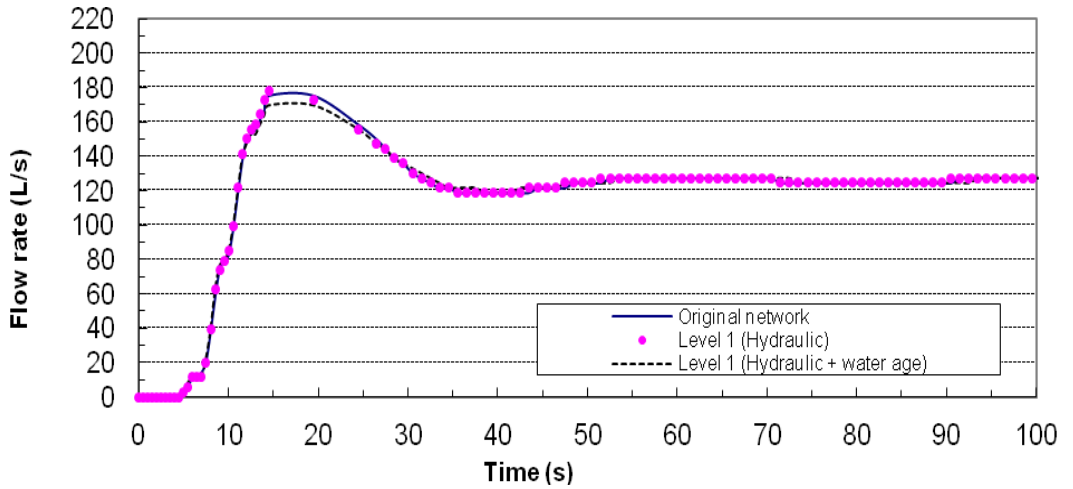
Fig. 13. Transient flow rate at node J2 due to suddenly demand releases at node J10 for the simplified (level 1) and original networks.
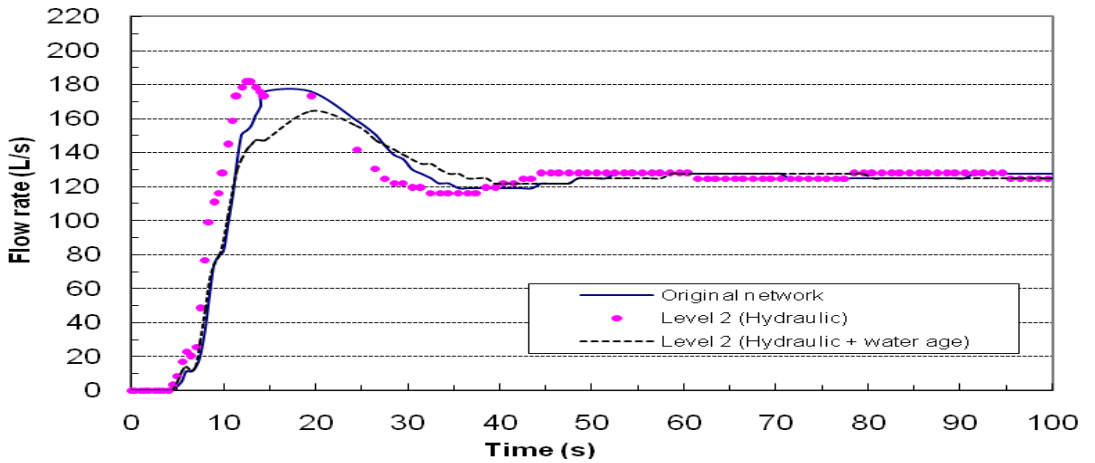


Fig. 14. Transient flow rate at node J2 due to suddenly demand releases at node J10 for the simplified (level 2) and original networks.
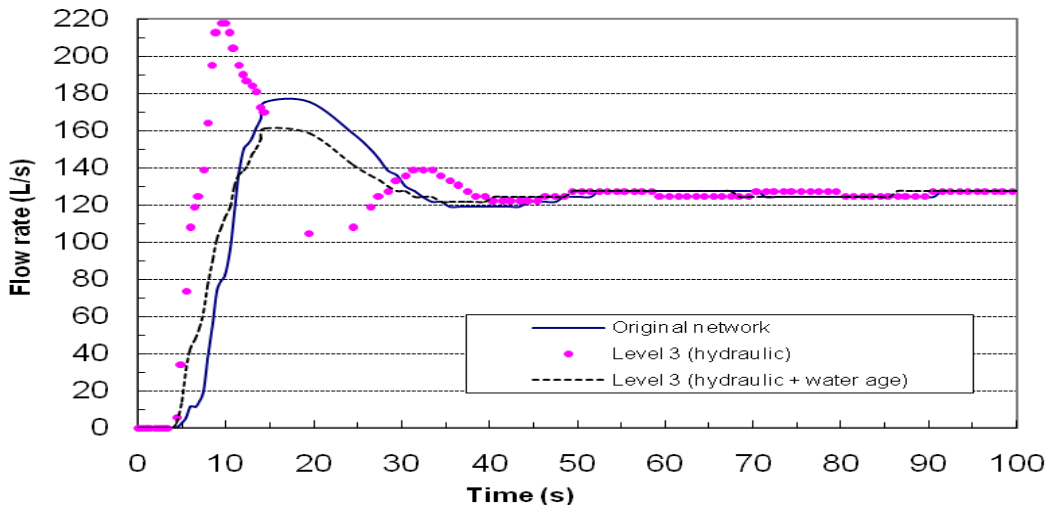
Fig. 15. Transient flow rate at node J2 due to suddenly demand releases at node J10 for the simplified (level 3) and original networks.

## 5.3 Effect of Water Demands' Concentration (3<sup>rd</sup> scenario)

To show the effect of water demands' concentration on water hammer phenomenon, a boundary distributed demands loaded on nine nodes of the original network (J3~J11) with a value of 14 L/s at each node were concentrated at the end node (J10) with an equivalent boundary value of 126 L/s and both distributed and concentrated demands were suddenly and linearly shut-off and released through a short period of 2 seconds. **Figures (16 and 17)** show the simulated transient pressure head at node J10 and flow rate at node J2 for both distributed and concentrated demands for two cases of suddenly shut-off and release, respectively. It is clear from these figures that concentrating the demands produces bigger transient pressure head and flow rate compared with the distributed one in case of demands shut-off as well as demands release. It is observed from the figures that 100 seconds period after shutting-off the concentrated or distributed demands is not sufficient to reach to the steady-state flow while it takes only around 50 seconds after releasing the demands to reach to the steady-state conditions. Also, **Figs. (16 and 17)** illustrate that the transient pressure head at node J10 and flow rate at node J2 for the original network loaded by either distributed or concentrated demands have an inverse trend with a short time lag which may be attributed to the location's difference between the two joints.
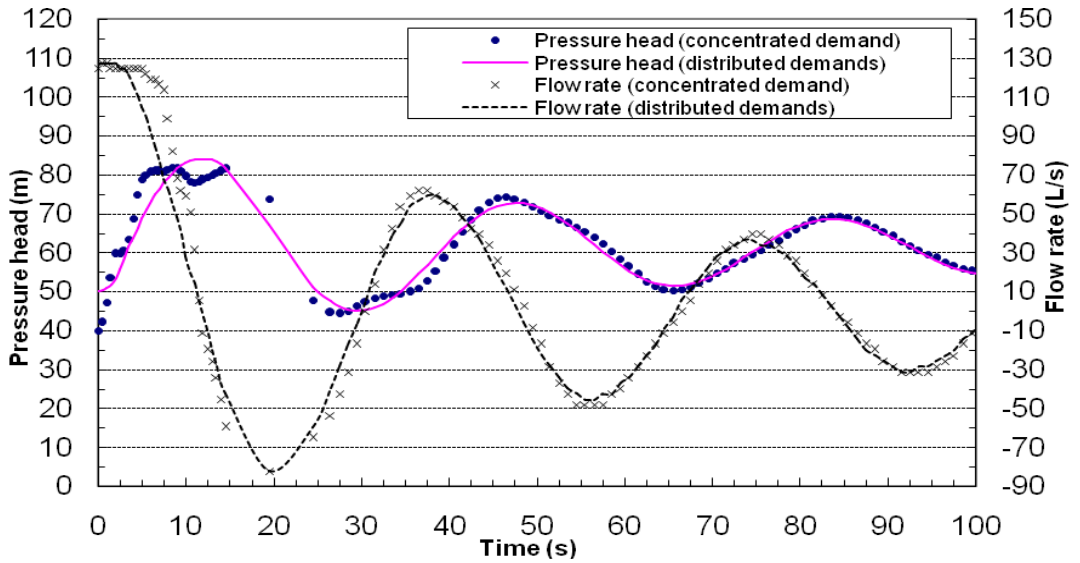
Fig. 16. Transient pressure head at node J10 and flow rate at node J2 due to concentrated and distributed demands sudden shut-off.
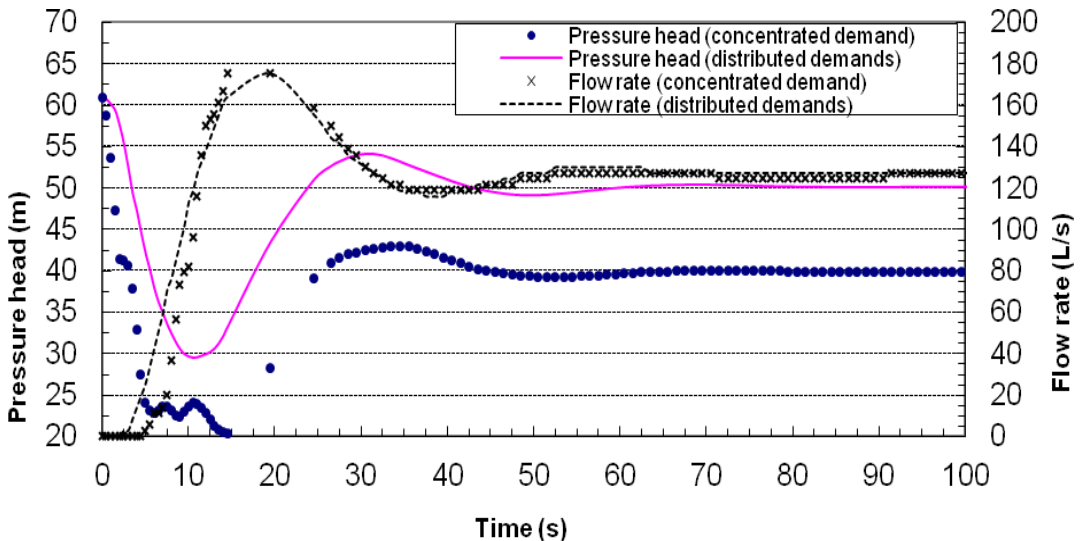


Fig. 17. Transient pressure head at node J10 and flow rate at node J2 due to sudden release of concentrated and distributed demands.

## 6. CONCLUSIONS

Simplification is an indispensible design step for water supply and irrigation pipes networks. Three types of simplifications may be performed on distribution networks as; hydraulic equivalence, hydraulic plus water quality equivalence, and demands' concentration. Variable water demands and usage patterns in water distribution systems may create transient flow that could make so many undesirable consequences. The effect of pipes networks' simplification on the transient flow must be accurately

determined so that they can be properly designed. Three scenarios producing transient flow in both simplified and original networks were investigated in this research. The transient flow was initialized by linearly and suddenly shutting-off or releasing of either distributed or concentrated boundary demands in a short period of 2 seconds. WHAMO software which uses the implicit finite difference scheme for solving the momentum and continuity equations at unsteady-state case was used in the simulation. The major findings of this study can be summarized as:

1- In all cases, for both simplified and original networks the peaks of the transient pressure and flow rate occur in the first cycle thus demonstrate the effect of friction on damping the transient flow.

2- Hydraulic equivalence and demands' concentration simplifications increase the peak values for the transient pressure and flow rate in the simplified network compared with the original one. However, hydraulic plus water quality equivalence simplification results in an adverse effect.

3- As the degree of simplification increases the transient pressure head and flow rate of the simplified network deviate more from those of the original network.

4- In case of the hydraulic equivalence, the frequency of the transient waves increases as the simplification level increases. However, the frequencies of the original and hydraulic plus water quality simplified networks are found to be the same. This result is quite clear in the $3^{rd}$ level of hydraulic simplification which converts the pipes network from looped to a single line.

5- For the transient flow results from boundary demands' shutting-off, the simulation period (100 seconds) is not sufficient to achieve the steady-state flow conditions in both simplified and original networks simulated in this study. However, it takes only around 50 seconds after releasing the boundary demands to reach to the steady-state flow conditions.

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## تأثير عمليات تبسيط شبكات المياه على ظاهرة المطرقة المائية

تعتبر المطرقة المائية التي تتولد في الشبكات من الظواهر الهيدروليكية الخطرة التي قد تؤثر على سلامتها. ففي كل شبكات المياه يقوم المصمم بعمليات تبسيط للشبكة الحقيقية بشبكة أخرى مكافئة من الناحية الهيدروليكية أو من الناحية الهيدروليكية والنوعية (من حيث جودة المياه) معا حتى يسهل حلها. كما تتضمن عمليات التبسيط تركيز استهلاكات المياه عند نقاط محددة بالشبكة. تقوم هذه الدراسة بتوضيح تأثير عمليات التبسيط المختلفة على موجات الضغط والسريان المصاحبة للمطرقة المائية. تمت الدراسة على شبكة مياه تتكون من 12 ماسورة من البولي إيثيلين عالي الكثافة ذات ضغط تشغيل 10 جو على شكل حلقتين وتتصل بخزان أرضي مرتفع المنسوب. تم تبسيط الشبكة إلى 3 مستويات بشبكات أخرى مكافئة هيدروليكياً وشبكات مكافئة هيدروليكياً ونوعياً كما تم تحميلها باستهلاكات موزعة ومركزة. تم في هذا البحث دراسة 3 سيناريوهات للسريان غير المستقر في الشبكة ففي الأول والثاني تم محاكاة عملية القفل والفتح الفجائي لمحبس خروج الاستهلاكات المركزة من الشبكة. وفي السيناريو الثالث تم دراسة تأثير تركيز استهلاكات المياه على موجات الضغط والسريان التي تصاحب السريان غير المستقر الذي يتولد من عمليات القفل والفتح الفجائي للمحابس. تمت المحاكاة الرياضية الممتدة باستخدام برنامج WHAMO لحل المعادلات التفاضلية غير الخطية التي تمثل السريان غير المستقر للمطرقة المائية مع أخذ الظروف الابتدائية والمحيطة في الاعتبار. وقد وجد أنه في المكافئة الهيدروليكية وكذلك عند تركيز استهلاكات المياه فإن موجات الضغط والسريان الناتجة عن المطرقة المائية تزيد في الشبكة المبسطة عنها في الشبكة الحقيقية بينما وجد عكس ذلك في المكافئة الهيدروليكية المصحوبة بالمكافئة النوعية وتزداد درجة الحيود كلما زادت درجة تبسيط الشبكة. كما وجد أن عملية الفتح الفجائي للمحابس في الشبكات ذات الاستهلاكات الموزعة أو المركزة تؤدي إلى سريان غير مستقر يصل خلال فترة قصيرة إلى حالة السريان المستقر عنها في حالة القفل الفجائي. وقد وجد كذلك أن شبكة المياه المكافئة هيدروليكياً والتي بُسِّطَت إلى خط واحد بدلا من الحلقات ينتج عنها موجات ضغط وسريان ذات ترددات عالية بالمقارنة بالشبكة الأصلية والشبكات المكافئة هيدروليكياً ونوعياً وحتى الشبكات المكافئة هيدروليكيا ذات درجات التبسيط الأقل. ومن هنا فإن عمليات تبسيط الشبكات يجب أن تتم في أضيق الحدود حيث أن موجات الضغط والسريان الناتجة عن المطرقة المائية تحيد عن الضغوط الحقيقية التي تحدث في الشبكة الأصلية كلما زادت درجة التبسيط.