THESIS

MODELING OF HYDRAULIC TRANSIENTS IN CLOSED CONDUITS

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ABSTRACT

MODELING OF HYDRAULIC TRANSIENTS IN CLOSED CONDUITS

Hydraulic transients (often known as 'water hammer') occur as a direct result of rapid variations in the flow field in pressurized (closed-conduit) systems. For example, changes in velocity from valve closures or pump operations cause pressure surges that are propagated away from the source throughout the pipeline. The elasticity of the pipe boundaries and the compressibility of the fluid prevent these sudden changes in pressure from taking place instantaneously throughout the fluid. The associated pressure changes during a transient period are often very large and occur very rapidly (within a few seconds). If the maximum pressures exceed the bar ratings (mechanical strength) of the piping material, different types of failure such as pipe bursts can occur. Similarly, if the minimum pressure drops below the vapor pressure of the fluid, cavitation can occur and can be detrimental to the pipeline system.

The purpose of this research is to model and simulate hydraulic transients in a closed conduit water system using different numerical methods. First, a numerical model was implemented to simulate the water level oscillations in a surge tank caused by the rapid closure of the outlet valve. The water surface oscillation results from the numerical model were compared with experimental results obtained from a surge tank experiment and found to be in good agreement. Furthermore, the stability and accuracy properties of the first-order explicit Euler time discretization scheme and the fourth-order Runge-Kutta (RK) time advancement scheme are highlighted using this example. It is found that using a higher-order scheme (such as the 4^{th} order RK scheme) not only ensures a greater degree of numerical stability, but permits the use of larger time steps to achieve a similar degree of accuracy as the less stable first-order

scheme. This is followed by a field test case study to investigate a pipe burst that occurred on a pipeline system in the Man-Made River in Libya. The Bentley HAMMER V8i software was employed to study this problem. A total of 28 scenarios were simulated using different combinations of the operating levels in the upstream Ajdabiya Reservoir and the downstream Gran Al-Gardabiya Reservoir and different time to closure of the valve. The simulation results show that the transient pressures in the pipeline exceeded the bar rating of the pipe where the burst occurred for most of the simulated scenarios.

The range of results from the idealized simulations to the field test case study of hydraulic transients presented in this research highlights the importance of accurate prediction of the pressure fluctuations in order to ensure that a pipeline's integrity is not compromised.

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CHAPTER 1. INTRODUCTION

1.1 Introduction

Under steady state conditions in a pipeline system, flow variables like discharge remain constant. However, if a sudden change occurs in the system through a change in control operations such as the closure of an outlet valve or the sudden shutdown of a pump due to power failure, a transient state is initiated, and it takes a finite amount of time before another (new) steady-state condition is established in the pipeline system. The flow phenomenon associated with such rapid changes is called a hydraulic (or fluid) transient. The main concern during a hydraulic transient in a system is the rapid fluctuations in the pressure since dramatic changes in the pressure can result in catastrophic damage to pipelines and hydraulic machinery.

Hydraulic transients have been studied by researchers for more than a century. Due to the devastating effects of hydraulic transients, their analyses is very important in order to determine the rapid pressure variations that result from flow control operations and therefore establish operational guidelines for hydraulic systems so as to ensure an acceptable level of protection against system failure.

Numerical models are widely used to study hydraulic transients since analytical solutions to the nonlinear governing equations for transient flows are difficult if not impossible to obtain. An effective numerical model should allow a hydraulic engineer to analyze a potential hydraulic transient event '*a priori'* in order to identify and evaluate alternative solutions for controlling the extreme pressures that may occur in the system.

A variety of commercial software is available for simulating hydraulic transients and can be used for the design of sophisticated pipeline networks and for research studies. Regardless of the availability of such software, it is imperative for hydraulic engineers to understand the hydraulic transient phenomena in order for them to be able to use sound engineering judgment in evaluating the output from simulations.

1.2 Objectives

This thesis focuses on modeling of hydraulic transient phenomena. The main objective of this research is to study the hydraulic transient phenomena in detail and simulate the resulting transient pressures due to sudden valve closure by using different numerical methods. In particular, water surface oscillations in a surge tank are simulated as part of this research using different numerical methods. The results from numerical simulation of water level oscillations in the surge tank are compared with results from an experiment that was carried out by Professor Karan Venayagamoorthy in South Africa. This is followed by a field test case study to investigate a pipe burst that occurred on a pipeline system in the Man-Made River in Libya.

1.3 Thesis layout

Chapter 2 provides a literature review of hydraulic transients in closed conduit flows. In addition, different control devices are presented. Chapter 3 consists of several parts. First, a simplified version of the governing equations for describing unsteady flow in a surge tank is derived. A numerical simulation of an experimental study of water surface oscillations in a surge tank is then presented. Two different numerical discretization methods are used to highlight the stability and accuracy properties of numerical schemes. Finally, a valve closure problem in a closed conduit flow is presented as a second example. Chapter 4 focuses on a case study of a pipe burst most likely caused by hydraulic transients in a transmission system in the Man-Made River in Libya. Different operational and valve closure scenarios are simulated using Bentley HAMMER

V8i software with an eye to use reverse engineering to explain the cause of the burst. Chapter 5 gives a brief summary of the main conclusions and suggestions for further research.

CHAPTER 2. LITERATURE REVIEW

2.1 Introduction

When water flows under pressure in a closed conduit (pipeline), the laws governing the changes of pressure and velocity along the pipe depend upon the conditions under which the flow occurs. If the water is considered to be incompressible and the discharge remains constant, the steady flow energy equation can be used to analyze the energetics of the flow at any given two cross-sections in the conduit. However, when the motion is unsteady, that is, when it varies rapidly from one instant to the next at any given location in the conduit, rapid pressure changes can occur and the steady flow energy equation is no longer applicable. Such rapid fluctuations in pressure are referred to as "hydraulic transients", commonly known as "water hammer" because of the hammering sound that often accompanies the phenomenon (Parmakian 1963).

Hydraulic transients in closed conduits have been a subject of theoretical and practical research for more than a century. A common and simple example is the knocking sound or hammering noise which is often heard when a water faucet in a house is rapidly closed. The transient state of the flow from time of closure until a new steady state condition is established is complex due to pressure surges that propagate away from the valve. By closing the valve rapidly, the valve converts the kinetic energy carried by the fluid particles into strain energy in the pipe walls. This results in a "pulse wave" of abnormal pressure to travel from the disturbance into the pipe system. The hammering sound that is sometimes heard results from the fact that a great portion of the fluid's kinetic energy is converted into pressure waves, causing noise and vibrations in the pipe. Energy losses due to mainly friction cause the transient pressure waves to decay until a new steady state is established (Boulos, Karney, Wood, & Lingireddy, 2005).

Figure 1 illustrates the hydraulic transient phenomenon in a closed conduit flow as a result of rapidly closing a valve. The transient occurs during the time interval T_t, between an initial condition when the valve closure begins and a final condition when the flow in the closed conduit comes to rest. Figure 1 shows the behavior of the pressure transient at a fixed point just upstream of the valve. The pressure (p) is presented as a function of time (t). At the start of the valve closure, the pressure in the system is p_i and, once the transient decays, the final pressure in the system is given by p_f. In between the two steady state conditions, the pressure fluctuates as shown in the figure. Both the maximum and minimum pressures can be significantly higher or lower than the pressure under steady state conditions. It is the accurate prediction of these extreme pressures that is of vital importance during a hydraulic transient event.

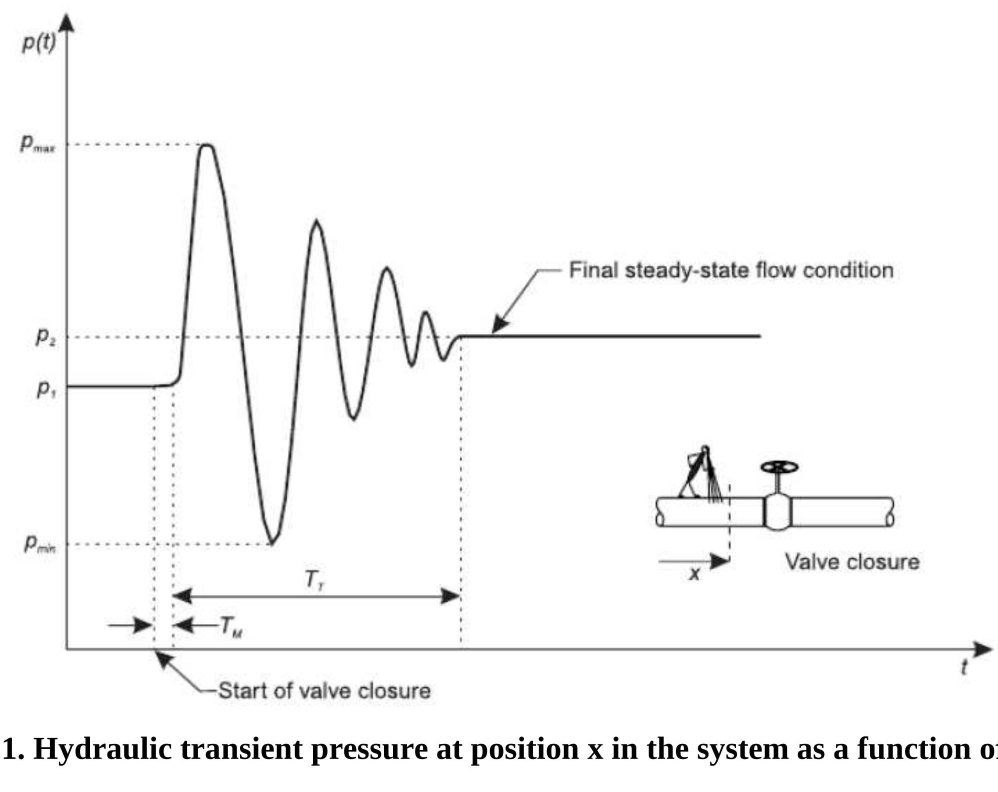

Figure 1. Hydraulic transient pressure at position x in the system as a function of time.

While engineers usually use the steady operating conditions as the basis for system analysis and design, the transient analysis is often at least (if not more) as important as the analysis of the steady-state conditions. The extrema of the transient pressures must be determined in order to properly design a pipeline so that it can withstand these extreme pressures. For design purposes, pipes are usually characterized by their "pressure ratings" that define their mechanical strength. Therefore, pressure ratings have a significant influence on their cost (Boulos, et al, 2005).

In what follows, an overview of hydraulic transients is provided. First, the consequences are discussed in slightly more detail, followed by events that cause hydraulic transients. A brief discussion of the different methods that are used to analyze hydraulic transients is then presented and this is followed by a discussion of control measures that are commonly used to regulate transients in closed conduit systems.

2.2 Consequences of Transients

Hydraulic transient events in water distribution system can cause significant damage, disruption, and expense (Boulos, et al, 2005). In general, transient events are usually most severe at control valves, pump stations, in high-elevation areas, and in remote locations that are far from overhead storage tanks. However, all systems have to start up, switch off, undergo flow changes, and so on. In addition, water systems are not immune from human errors, malfunction and break down of mechanical devices, and other risky events (Wood, 2005).

Hydraulic engineers are most concerned with consequences from transient effect that affect safety, cause equipment damage, or result in operational difficulties. Some of the common consequences are (Boulos, Karney, Wood, & Lingireddy, 2005):

- Maximum pressures in hydraulic systems. This is the most common consequence of hydraulic transients;
- Occurrence of local vacuum conditions at specific locations that may result in cavitation either within specific devices such as pumps or within a pipe;
- Hydraulic vibration of a pipe, its supports, or in specific devices;
- Occurrence of contaminant intrusion at joints and cross-connections.

2.2.1 Maximum pressure in a system

During transient events, the maximum pressures can cause damage in pipelines, tunnels, valves, or other equipment. Sometimes, the damage can result in the loss of human life. On the other hand, high pressures may not necessarily destroy pipelines or other devices, but can cause cracks in internal linings, damage connections between pipes or cause deformations in equipment. This equipment could be valves, air valves, or even hydraulic transient protection devices. Moreover, high pressures may result in leakages in hydraulic systems even though no visible damage can be noticed.

2.2.2 Vacuum conditions and cavitation

Vacuum conditions (i.e., low pressures in the system that are close to the vapor pressure of the fluid) should be avoided because they can cause high stresses and strains in the system. This is due to the fact that, as pressure drops in a system and approaches the vapor pressure of the fluid, the fluids begins to boil, resulting in the formation of air bubbles. When these tiny air bubbles are transported to a high pressure region by advection, they implode, and cause excessive stresses on the pipe walls.

2.2.3 Hydraulic vibrations

Strong hydraulic vibrations may damage pipeline, internal lining, or system equipment. Such long-term moderate surges may gradually lead to fatigue failure. Oscillations of water masses through a pipeline may also cause vibrations and suction of air into the pipeline. Therefore, neglecting such influences during the design phase may lead to system damage.

2.2.4 Water quality and health implications

Hydraulic transients may result in objectionable effects on water quality and have serious health implications. High intensities of fluid shear stresses from hydraulic transients may cause erosion and resuspension of settled particles as well as biofilm detachment. Moreover, low-pressure transients can cause the intrusion of contaminated ground water into a pipe at a leaky joint and/or through cracks in a pipe. Depending on the size of the leaks, the volume of intrusion can vary from a few gallons to hundreds of gallons (Boulos, et al, 2005).

2.2.5 Severity of transient pressures

Urban water delivery network systems, particularly the underground components, can be damaged by various causes such as earthquakes, severe cold weather, heavy traffic loads on ground surfaces, etc. Hydraulic transients have also been known to cause catastrophic damage in urban water delivery systems. Sometimes, it is difficult to predict these transient effects due to uncertainty. Two real examples are presented here to show how the rapid pressure changes from transient events have resulted in catastrophic damage. The first damage occurred in Denver, Colorado on February 7th, 2008. The other one occurred in Libya on February 10th, 2012 in a transmission pipeline linking Ajdabiya Reservoir and Al-Gardabiya Reservoir. A 66-in water main beneath Interstate 25 in Denver burst on February 7th in the afternoon. This pipe burst resulted in a sinkhole, about three lanes wide and 16 feet deep, and forced the closure of all northbound lanes of the freeway (see Figures 2 and 3, Leslie 2008). As mentioned earlier, there are many causes of hydraulic transients, and one of these causes is pump failure. *A posterior* analysis of the system revealed that the burst occurred due to excessive pressure build up in the system as a result of a pump failure.

Figure 2. 66-in water main ruptures as a result of hydraulic initiated by a pump failure beneath the I-25 in Denver (Leslie, 2008)**.**

Figure 3. 66-in water main ruptures, I-25 in Denver (Leslie, 2008)**.**

On February 10th, 2012, the personnel in the control room at the City of Ajdabiya reported huge fluctuations in the flow meter readings at Ajdabiya Reservoir, which is part of the Man-Made River Project in Libya. It was found that the transmission pipeline at station (76+820) had burst, causing a large leakage, as shown in Figure 4. Based on a report prepared by the Man-Made River Project management, the amount of leakage was estimated to be about 200,000 m^3. The cause of this pipe burst is mostly likely due to a hydraulic transient event caused by sudden valve closure. This aspect will be investigated further in Chapter 4 of this thesis.

These examples highlight the severity of the damage that can result from hydraulic transients in pressurized systems. Therefore, it is important for engineers to be cognizant of the various causes of such events and develop appropriate design and operational criteria.

Figure 4. 4-meter diameter pipe burst in Libya in February 2012 (personal communication, August 28, 2012).

2.3 Causes of hydraulic transients

Hydraulic transient events are disturbances in the flow field that occur due to operational or other unforeseen changes in a system. The disturbances, which are due to the rapid changes in pressure, propagate as pressure waves that travel at the speed of sound in the fluid medium. The speed of sound depends on the compressibility of water and the elastic properties of the pipe. Some common operational events that require transient analysis are (Larock, et al, 2000):

- Pump start up or shutdown
- Valve closing and opening
- Rapid changes in demand conditions
- Changes in transmission conditions

- Pipe filling or draining

2.4 Pressure fluctuations during a hydraulic transient

The reason for the rapid changes in pressure can be illustrated using a simple closed conduit of length L with a valve at the downstream end and a tank at the upstream end that is held at a constant head. A momentum impulse analysis shows that the excess pressure head resulting from a rapid valve closure from an initial state V_0 is

$$\Delta h = -\frac{a}{g} V_0 \ , \tag{1}$$

where a is the speed of the pressure wave and g is the gravitational acceleration. The wave speed a can be calculated from the properties of the conduit material and the fluid. The formula for the wave speed for a conduit with slightly deformable walls is given by

$$a = \sqrt{\frac{K_f}{\rho\left(1 + c\left(K_f D\right)/eE\right)}} \tag{2}$$

where K_f is bulk modulus of elasticity of the fluid; ρ is fluid density; D is the inside diameter of the conduit; e is the wall thickness; E is Young's modulus of elasticity of the conduit-wall material; $c = 1 - \nu_p/2$ for a pipe anchored at its upstream end only; ν_p is Poisson's ratio. $c = 1 - {\nu_p}^2$ for a conduit anchored throughout its length in order to restrain the pipe from axial movement; and $c = 1$ for a pipe anchored with expansion joints throughout (Boulos, et al, 2005). Typically, the wave speed is of the order of the speed of propagation of sound in the fluid medium under consideration. For example, $a \approx 1500$ m/s in a closed conduit carrying water. Clearly, from equation (1), it can be seen that the change in pressure head due to a sudden (instantaneous) change in velocity can be at least to 2 to 3 orders of magnitude larger than the change in velocity, indicative of the high transient pressures that occur from such operations.

Once a pressure wave is initiated in a conduit, it propagates back and forth in the conduit until it is eventually dissipated by friction. During the time interval $0 < t < L/a$, the pressure wave propagates toward the tank (upstream) and will reach the tank in L/a seconds. On the tank side of the wave, the flow will be undisturbed (normal conditions), while on the valve side of the wave (behind the wave), the flow will be at rest but the pressure head will have increased by Δh, thereby causing an enlargement of the pipe diameter. In the time period $L/a < t < 2L/a$, the pressure wave would have reflected back from the tank and propagated toward the closed valve and will reach it at $t = 2L/a$. However, since the valve is fully closed, the flow instantaneously comes to rest, causing the pressure head to drop by Δh. For $2L/a < t < 3L/a$, a negative pressure wave now propagates back towards the tank where, upon reaching the tank, it gets re-reflected as a positive wave. For $3L/a < t < 4L/a$, the water starts to flow back from the reservoir into the conduit and the pressure rises back to normal at the valve at time of $4L/a$ seconds after closure. This is the complete pressure wave cycle. In principle, since the valve remains closed, this pressure wave cycle would occur repeatedly if the flow is frictionless. However, in reality, it gets dissipated because by friction and other minor losses throughout the hydraulic system.

2.4.1 Rigid column and elastic column theories

The rigid model assumes that the pipeline is not deformable and the liquid is incompressible. Hence, system flow-control operations affect only the inertial and frictional aspects of the transient flow. Given these considerations, it can be demonstrated using the continuity equation that any system flow-control operations result in instantaneous flow changes throughout the system. In addition, the liquid travels as a single mass inside the pipeline, causing a mass oscillation. If liquid density and pipe cross section are also constant, the instantaneous velocity is the same across all sections the pipeline. In other words, the flow variables are independent of space and are only functions of time. Hence, the governing equations revert to ordinary differential equations that are much easier to solve than the original set of partial differential equations (see Section 2.5).

The rigid model has limited applications in hydraulic transient analysis because the resulting equations do not accurately represent pressure waves caused by rapid flow-control operations. The rigid model applies to slower surge or mass oscillation transients. Generally, the maximum transient head envelope calculated by rigid water column theory is a straight line. Bentley HAMMER software only employs the rigid column theory under certain conditions.

On the other hand, the elastic column model assumes the momentum of the fluid leads to expansion or compression of the pipeline and fluid, both assumed to be linear-elastic. Since the fluid is not completely incompressible, its density can change slightly during the propagation of a transient pressure wave. The transient pressure wave will have a finite velocity that depends on the elasticity of the pipeline and of the fluid as described before using equation (2).

Before the proliferation of computational power, the subject of rigid water column-theory was very popular. Substantial effort was made to improve the accuracy and to determine the range of applicability of the rigid-column theory. Figure 5 shows a plot of the initial pressure head to the transient versus the head valve closure time normalized by one half the characteristic time, (L/a) in a frictionless (or very low friction) system. This graph highlights the different criteria that have been proposed since 1933 to determine when an elastic solution is necessary and when a rigid-column solution is sufficiently accurate.

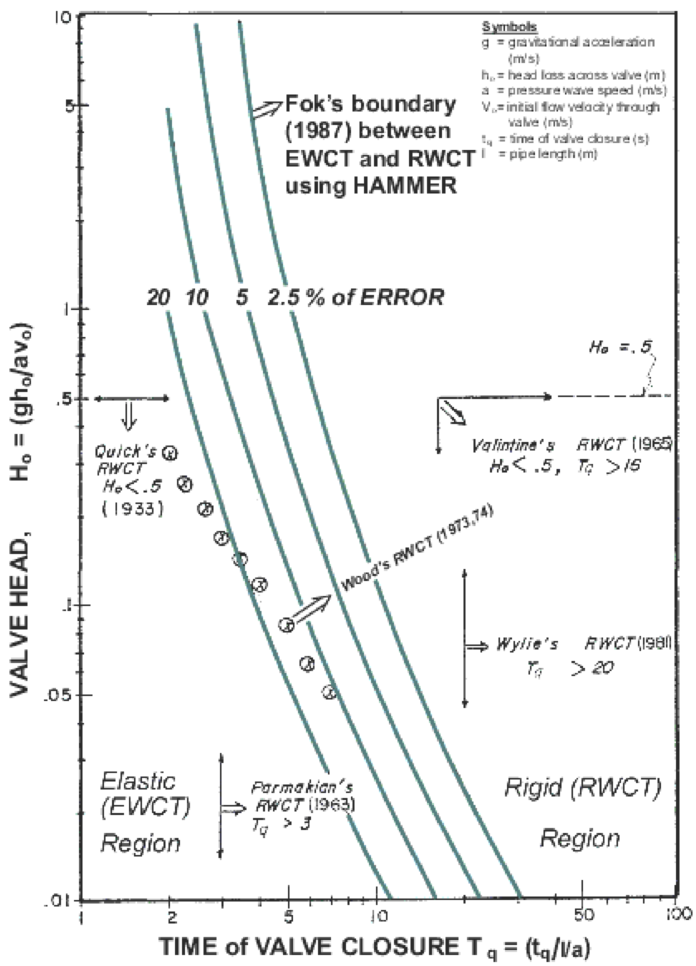

Figure 5. Criteria for determining when an elastic solution is necessary and when a rigid-column solution is sufficiently accurate (Bentley, 2013)

2.5 A brief review of hydraulic transients analysis and methods

The hydraulic transient problem was first studied by Menabrea (although Michaud is generally credited for carrying out the earliest analysis, Anderson 1976). Michaud studied the effect of using air chambers and safety valves for controlling hydraulic transients. By the end of the nineteenth century, attempts to achieve expressions relating pressure and velocity changes in a pipe were carried out by some researchers such as Carpenter and Frizell (Joukowsky 1904). Frizell was successful in his effort to find an expression to relate pressure and velocity changes in a pipe. He also discussed effects of branches in pipelines. Joukowsky (1904) and Allievi (1903) are generally credited for providing the first mathematically correct formulations of the governing equations for hydraulic transients. The best known equation (which has already been discussed – see equation 1) in transient flow theory was derived by Joukowsky and is often called the fundamental equation of hydraulic transients. In addition, Joukowsky studied wave reflections from an open branch, the use of air chambers and surge tanks, and safety valves (Boulos, Karney, Wood, & Lingireddy, 2005).

Allievi (1903, 1913) developed a general theory for hydraulic transients from first principles. He showed that the convective term in the momentum equation was negligible. Allievi also produced charts for quantifying the pressure rise at a valve for uniform closures. Efforts by Jaeger (1933), Wood (1937), Rich (1944), Parmakian (1955), Streeter and Lai (1963), and Streeter and Wylie (1967) led to the classical mass and momentum equations for one-dimensional (1D) hydraulic transient flow as follows

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

$$\frac{\partial V}{\partial t}+g\frac{\partial H}{\partial t}+\frac{4}{\rho D}\tau_w=0 \qquad (4)$$

in which τ_w is shear stress at the pipe wall; *D* is pipe diameter; *x* is the spatial coordinate along the pipeline; and t is temporal coordinate (Ghidaoui, Zhao, Mclnnis, & Axworthy, 2005). These equations were fully established by 1960 and have since been analyzed, discussed, and highlighted in a number of classical texts and research papers on hydraulic transients. In addition, most hydraulic transient software such as Bentley HAMMER used in chapter 4 of this thesis is based on this pair of equations. These are the two fundamental equations that describe 1D hydraulic transient problem. They contain all the physics necessary to model wave propagation in a pipe system (Boulos, et al, 2005). It must be noted that these equations are valid for a uni-directional, axisymmetric low Mach number flow of a compressible fluid in a slightly deformable pipe. Various methods for solving this set of coupled partial differential equations have been developed. They range from approximate analytical approaches to numerical solutions of the nonlinear system. Some of these techniques are discussed next.

2.5.1 Arithmetic mean

The calculation of the hydraulic transient pressure as a result of a sudden change in flow velocity for a time period $t \leq 2L/a$ is simple and can be obtained using equation 1. This has been verified experimentally by Joukowsky (1904). However, it must be noted that the effect of friction has been neglected when using such an approach (Dawson & Kalinske, 1939).

2.5.2 Graphical method

The solution of hydraulic transient problems graphically is similar to the arithmetic method but allows for friction to be considered by assuming that it can be specified at one of the end points of the pipeline. Theoretically, this is not correct but, practically, results are roughly indicative of the effect of hydraulic transient pressures, at least for the first wave cycle. The graphical method is normally used to determine the transient pressures at the beginning and end points of a pipeline and hence the pressure must be determined in steps of $2L/a$ seconds where L is the length of the pipeline under consideration.

In addition, there are other graphical methods that have been developed to calculate hydraulic transient pressures in compound pipes, pump discharge lines, branched pipes, relief valves, and air chambers (Dawson & Kalinske, 1939).

2.5.3 Algebraic method

This method is solving the basic transient flow equations that have been developed (Streeter and Wylie, 1967). The procedure of this method is generally based on the method of characteristics (Wood, 2005).

2.5.4 Method of characteristics (MOC)

The method of characteristics is perhaps among the most popular methods that are used for solving the hydraulic transient equations. The MOC (especially for a constant wave speed), is superior compared to other methods, especially for capturing the location of steep wave fronts, illustration of wave propagation, ease of programming, and efficiency of computations (Chaudhry, 1987). This method is described in slightly more detail in Chapter 3.

2.5.5 Finite-difference methods

There are two broad categories of finite difference methods based on the time discretization schemes, namely: (1) implicit methods and (2) explicit methods. The implicit methods allow for larger time steps to be used in the simulations while preserving numerical stability. Two different explicit schemes are used in the numerical simulations discussed in Chapter 3.

2.5.6 Wave plan method

This method is similar to the method of characteristics because both techniques explicitly combine wave paths in the solution procedure. A fine discretization is required for this method to achieve accurate solutions.

2.6 Control of Hydraulic Transients

During a hydraulic transient state, a pipeline may be subjected to objectionable high and low pressure cycles. The high pressures can damage the pipeline system components, such as valves, pumps, and other pipeline components, as discussed earlier. The change in the fluid velocity (more correctly discharge) in the pipeline systems is the first step that leads to a hydraulic transient. The resulting change in pressure is directly proportional to the change in velocity. Hence, as much as possible, sudden changes in the velocity should be avoided to minimize the occurrence of pressure transients in the system. Most control devices and operating procedures are designed and formulated in such a manner as to avoid sudden velocity changes.

2.6.1 Controlled valve closure schemes

One of the most common causes of hydraulic transients is the sudden closure of valves. The best way to determine the effects of different valve closure protocols is to perform computer simulations of the system's response and evaluate the resulting pressure transients. Based on such simulation studies, a control system must be developed that uses an appropriate valve closure protocol (Larock, et al, 2000).

2.6.2 Check valves

The best check valves close at the moment when forward flow stops, and do not slam shut. When a damped check valve is used, it must be treated in the same manner as a closing valve during the back flow time. The valve must be either closed quickly before reverse flow becomes large, or closed slowly over a time interval greater than the critical time of closing ($2L/a$). Otherwise, excessive high pressure could occur at the time of closure of the check valve. This problem is difficult to analyze because it requires a priori knowledge of the back-flow loss characteristics of the valve which are rarely available.

2.6.3 Surge relief valves

Sometimes, it is necessary to close valves fast to create a reduction in the flow velocity; this results in high transient pressures. In this case, the best solution is to use a surge relief valve. A surge relief valve opens when a prescribed minimum pressure is exceeded in the hydraulic system. The surge relief valve is generally located adjacent to the device that is expected to be closed rapidly ,and provides an escape for the flowing liquid before objectionable pressure transients occur in the system.

2.6.4 Air venting procedures

The procedure of filling an empty line of a pipeline system is important. The liquid must be introduced slowly into the system at a velocity of 1.0 ft/s or less. Air release and air vacuum valves must be placed to remove all air from the system slowly. Usually, these valves are located at the ends of the pipeline so each line can be pressurized and all air can be forced out. Therefore, proper locations and sizing of air-release and air-vacuum valves are an important part of the pipeline design process (Larock, et al, 2000).

2.6.5 Surge Tanks

A surge tank is an open standpipe or a shaft that is connected to the pipeline system or to the closed conduit of a hydroelectric power. The main purpose of a surge tank is to reduce the amplitude of pressure fluctuations by reflecting the incoming pressure waves, to improve the regulating characteristics of a hydraulic turbine, and to store or provide water in order to accelerate or decelerate water slowly. Depending upon its configuration, a surge tank may be classified as a simple tank, an orifice tank, a differentiation tank, a one-way tank or a closed tank. The water level oscillations in an orifice tank will be studied as a part of this thesis in chapter 3.

2.6.6 Air Chambers

An open-end surge tank would be an excellent device to place on the discharge side of a pump station to control both positive and negative hydraulic transient pressure waves. Since the discharge pressure of pumps is usually high, the surge tank would have to be very tall to extend above the hydraulic grade line (HGL). However, having a tall open-end surge tank would be uneconomical. The best solution in such cases is a device which can play the role of an open-end surge tank without needing the excessive height required of an open surge tank. The device is an air chamber, sometimes called a hydro-pneumatic tank, an air bottle or a shock trap. It is a small pressurized vessel, which contains both air and liquid. This device is connected to the discharge line from the pump station. The main purpose of the air chamber is to avoid negative pressures and column separation that may occur during daily operation conditions. On the other hand, this device can suppress excessive positive pressure as well.

2.7 Summary

A brief but broad overview of hydraulic transients in the context of closed conduit flows has been provided to highlight the salient issues of this important phenomenon. In what follows in Chapter 3, the phenomenon of transient flow as applied to the specific case of surge tanks is studied, primarily through numerical simulations based on the finite difference technique. The objectives are to compute the water surface oscillations in the surge tank and compare the simulated results with experimental data.

CHAPTER 3. SIMULATION OF TRANSIENT FLOW IN A SURGE TANK

3.1 Introduction

Numerous protection devices have been invented to protect a hydraulic system from potential detrimental effects of hydraulic transients. In general, control devices are designed to either store water, delay the change in flow, or discharge water from the line. The simple surge tank is one of the commonly used control devices. The surge tank is used to reduce the amplitude of pressure fluctuations by reflecting the pressure waves, and to prevent cavitation during start-up of a system by providing adequate flow to a low-pressure regime.

In what follows, flow through a simple laboratory-scale surge tank is investigated using numerical simulations of simplified forms of the 1-D hydraulic transient flow equations presented in Chapter 2. The simulation results are compared with experimental data obtained from a laboratory experiment conducted by Dr. Karan Venayagamoorthy in South Africa. The stability and accuracy properties of numerical schemes are also highlighted.

3.2 Surge tank schematic and governing equations

In an orifice tank, there is an orifice between the conduit and the tank (Figure 6). If the orifice area is the same as the area of the conduit, then the orifice losses are negligible, and the tank acts like a simple surge tank. On the other hand, if the orifice area is very small, then the inflow or outflow from the tank will be very small compared to flow in the conduit and in such a case the system behaves as if there is no surge tank.

The derivation of the dynamic (momentum) and continuity equations describing the water level oscillations in the surge tank are based on the following assumptions:

- The conduit walls are rigid, and the liquid is incompressible.
- The inertia of the fluid in the surge tanks can be neglected because it is small compared to the inertia of the fluid in the tunnel.
- Head losses in the system during the transient state can be computed using steady-state formulae for the corresponding flow velocities.

3.2.1 Dynamic Equation

Figure 6(a) shows a schematic of a horizontal tunnel having a constant cross-sectional area and figure 6(b) shows the corresponding free-body diagram with all forces acting on a control volume of the fluid.

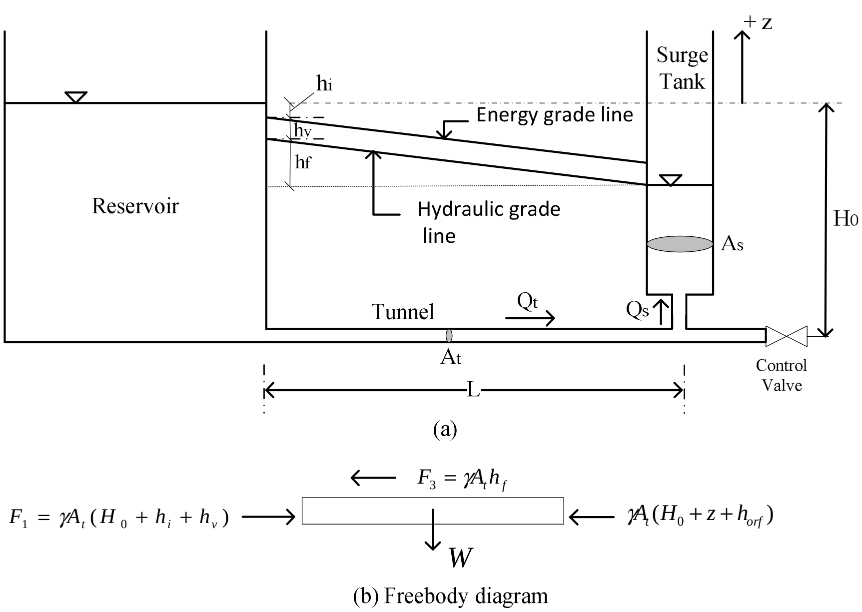

Figure 6. Schematic of a simple surge tank system (adopted from Chaudry 1987)

The forces acting on the fluid are:

$$F_1 = \gamma A_t (H_0 - h_v - h_i) \qquad (5)$$

$$F_2 = \gamma A_t(H_0 + z + h_{orf}) \tag{6}$$

$$F_3 = \gamma A_t h_f \tag{7}$$

where A_t is cross-sectional area of the conduit; H_0 is the static head; γ is specific weight of liquid; h_v is velocity head; h_i is intake head losses; h_f is frictional head and form losses in the conduit between the reservoir and the surge tank; and z is water level in the surge tank measured above the reservoir level (considered positive upward). Considering the downstream flow direction as positive, the resultant forces acting on the fluid element are

$$\sum F = F_1 - F_2 - F_3 \tag{8}$$

Substituting equations 70, 71, and 72 into equation 4, yields

$$\sum F = \gamma A_t\left(-z - h_v - h_i - h_f - h_{orf}\right) \tag{9}$$

In the conduit, the mass of the fluid element is $\gamma A_t L/g$, where L is length of the conduit and g is acceleration due to gravity. Hence, the rate of change of momentum of the fluid element is

$$= \frac{\gamma A_t L}{g}\frac{d}{dt}\left(\frac{Q_t}{A_t}\right)$$

$$= \frac{\gamma L}{g}\frac{dQ_t}{dt} \tag{10}$$

where Q_t is the flow rate in the conduit and t is the time.

According to Newton's second law of motion, the rate of change of momentum is equal to the resultant force. Therefore, from equations 9 and 10, we get

$$\frac{\gamma L}{g}\frac{dQ_t}{dt} = \gamma A_t\left(-z - h_v - h_i - h_f - h_{orf}\right) \tag{11}$$

Total head losses h = $h_v + h_i + h_f$+h_{orf} can be expressed as a function of discharge as $h = cQ_t|Q_t|$, where c is a coefficient. Therefore, equation 11 becomes

$$\frac{dQ_t}{dt} = \frac{gA_t}{L}(-z - cQ_t|Q_t|) \quad (12)$$

In the preceding derivation the conduit is assumed to be horizontal, and the cross-sectional area is constant. For conduits having different cross-sectional areas, the term A_t/L in equation 12 should be replaced with $\sum(A_t/L)_i$. The main head losses considered here are the losses due to pipe friction and head losses due to a sudden enlargement from the pipe to the surge tank. The head losses due to pipe friction can be calculated using the Darcy-Weisbach equation

$$h_f = f\,\frac{L}{D}\frac{V^2}{2g} = f\,\frac{L}{D}\frac{{Q_t}^2}{2gA^2} = c_f{Q_t}^2 \quad (13)$$

where

$$c_f = f\,\frac{L}{2gD{A_t}^2} \quad (14)$$

where f is friction factor; D is diameter of closed conduit.

Losses from a sudden enlargement are usually computed from the equation

$$h_i = K_L\frac{V^2}{2g} \quad (15)$$

where K_L is a empirically determined loss coefficient. Hence, we obtain

$$h_i = K_L\frac{V^2}{2g} = K_L\frac{Q^2}{2g{A_t}^2} = c_s{Q_t}^2 \quad (16)$$

where

$$c_s = \frac{K_L}{2gA_t{}^2} \tag{17}$$

Therefore, equation 12 becomes

$$\frac{dQ_t}{dt} = \frac{gA_t}{L}\left(-z - (c_f + c_s)Q_t|Q_t|\right) \tag{18}$$

3.2.2 Continuity Equation

The continuity equation for the junction of the conduit and the surge tank as shown in figure 6 may be written as

$$Q_t = Q_s + Q_v \tag{19}$$

where Q_s is flow into the surge tank (inflow is positive), and Q_v is the flow through the valve. However, equation 19 is valid for cases where the valve can be replaced by a turbine or pump and the flow through the turbine or the pump is designated as Q_v.

Since $Q_s = A_s(dz/dt)$, equation 19 becomes

$$\frac{dz}{dt} = \frac{1}{A_s}(Q_t - Q_v) \tag{20}$$

Equations 18 and 20 are the simplified (ordinary differential equations - ODEs) governing equations that describe the water-level oscillations located in the surge tank system shown in figure 6. The nonlinearity of these equations (noting that the discharge through the turbine Q_v could also be nonlinear) does not readily permit closed form solutions. Hence, numerical methods are often used to integrate these equations.

3.3 Surge tank experiment and data

A laboratory-scale surge tank model can be used to demonstrate hydraulic transients that arise as a result of rapid closure or opening of a valve. Time histories of the water level oscillations in the surge tank can be recorded and used for comparison with results obtained from numerical simulations. Such an experiment was carried out by Dr. Venayagamoorthy in South Africa while he was at the University of KwaZulu-Natal. The experimental setup is briefly discussed next since the numerical simulations were performed to closely replicate this setup.

3.3.1 Schematic of the surge tank experiment

A schematic of the experiment setup of the surge tank system is shown in figure 7. The system is comprised of a reservoir (feeder tank) at the upstream end to provide the energy head to drive the flow through a 45 mm diameter supply pipe. The pipe connects a large feeder tank (reservoir) with a cylindrical surge tank, which had a diameter of 122 mm. The length of the supply pipe L = 10.4 m. Under normal operating conditions, the water flows from the reservoir to the pipe and is discharged through the control valve into a collection tank. The collection tank was used to measure the flow rate under steady state conditions.

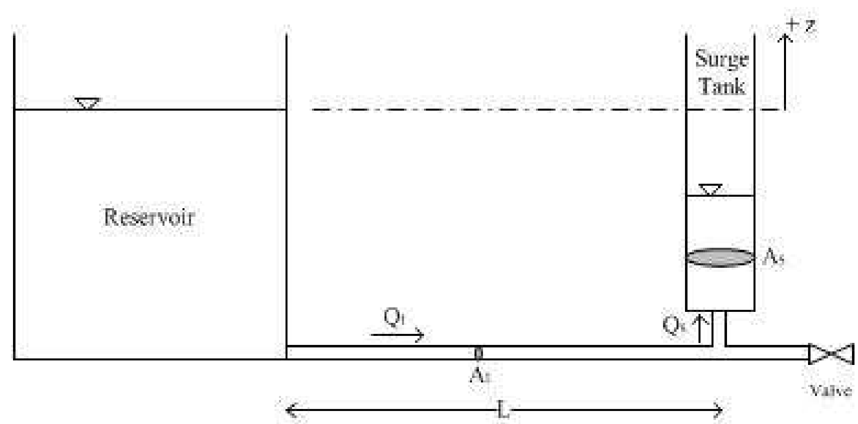

Figure 7. Schematic of the surge tank experiment

3.3.2 Experiment procedure

- In this experiment, the water level in the surge tank with all valves closed, except the surge tank isolator valve, was recorded. This level was used as the reference or initial water level in the surge tank.
- The control valve was then adjusted to give a steady flow rate (Q_0), which was recorded by timing how long it took to collect a known volume of water in the collection tank.
- The initial water level in the surge tank was then recorded with all valves in the open position.
- Then, discharge (outlet) valve downstream of the surge tank was rapidly closed and the time history at which the water level in the surge tank crossed the still water level and reached the extreme (maximum and minimum) levels, together with water surface elevations, were recorded for three full cycles.
- The same procedure was repeated for different values of steady state flow rates.

Table 1 shows the water surface elevation data as a function of time for a flow rate of Q_0 = 0.001723 m3/s. Figure 8 depicts the results shown in Table 1 graphically.

Table 1. Water level oscillations

First set	
Time (sec)	**Level (mm)**
3	4923
8	5075
12	4923
16	4832
20	4923
24	4989
29	4923
34	4877
39	4923
44	4961
49	4923
52	4897
57	4923

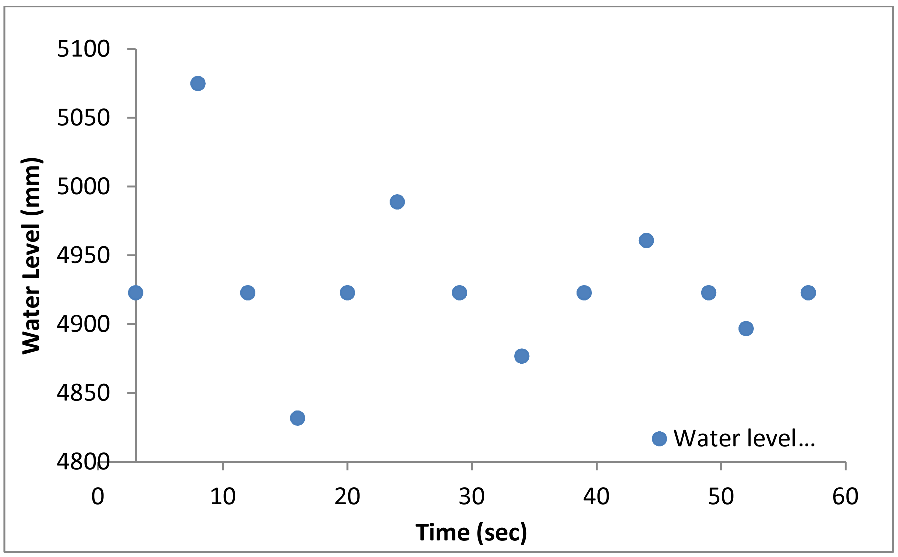

Figure 8. Water level oscillations – experimental results

3.4 Numerical solutions

Today, scientific computing is an important tool for conducting research in engineering. Different numerical methods are employed to solve physical governing equations, which are usually differential equations. There are many different methods that can be used to solve ordinary and partial differential equations, such as method of characteristics, finite difference, and finite element methods. The finite difference technique will be used to solve the surge tank problem presented in section 3.3. In general, for time-dependent (i.e., time marching or initial-value problems) partial differential equations, the finite-difference techniques fall into broad categories of explicit and implicit schemes. Explicit schemes are easier to program and solve but suffer from numerical stability issues that require the use of small time steps. On the other hand, implicit methods permit the use of larger time as they are generally numerically stable schemes, but this comes at a much higher computational cost. Detailed discussions of finite difference techniques and numerical stability can be found in classical texts on numerical methods such as Moin (2010).

Essentially, the ordinary differential equations (e.g., equations 18 and 20) are replaced by finite-difference approximations where the unknown quantities at the end of the time step are expressed as functions of the known conditions at the beginning of the time step. Here, two explicit finite difference schemes will be used to solve equations 18 and 20. These schemes are: the forward Euler method and the fourth-order Runge-Kutta method. A brief overview of these two methods is presented next, followed by the simulation results.

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3.4.1 Numerical methods

There are two key numerical properties of finite difference methods that must be considered. These are the numerical stability and the numerical accuracy of the chosen method. The numerical stability is a measure of whether the errors associated with the numerical approximation are bounded in time. If the errors are bounded, the numerical method is said to be stable. On the other hand, if the errors grow (and usually in an exponential manner), the method is said to be unstable. This is a vital property, as once a scheme becomes unstable, no meaningful results can be obtained from it. However, many numerical schemes can exhibit conditional stability; i.e., the solution remains stable for time steps smaller than a certain critical value. The goal then is to determine the critical time step that will ensure stability and the process to do so is usually called a stability analysis. The numerical accuracy is measure of the error between the numerical solution and the "true" solution. Usually, a comparison with known analytical or experimental data is required to evaluate the accuracy of a particular scheme for a given problem.

To examine the stability property of a given numerical method, consider a first-order ordinary differential equation of the form

$$y' = f(y, t) \tag{21}$$

The two dimensional Taylor series expansion of $f(y, t)$ is given by

$$f(y,t) = f(y_0,t_0) + (t-t_0)\frac{\partial f}{\partial t}(y_0,t_0) + (y-y_0)\frac{\partial f}{\partial y}(y_0,t_0)$$
$$+\frac{1}{2!}\left[(t-t_0)^2\frac{\partial^2 f}{\partial t^2} + 2(t-t_0)(y-y_0)\frac{\partial^2 f}{\partial t \partial y} \right. \qquad (22)$$
$$\left. + (y-y_0)^2\frac{\partial^2 f}{\partial y^2}\right] + \ldots$$

Collecting only the linear terms and substituting in equation 21, we obtain

$$y' = \lambda y + \alpha_1 + \alpha_2 t \qquad (23)$$

where λ, α_1, and α_2 are constants.

Stability analysis is usually performed on the model consisting of only the first term on the right hand side of equation 23 since it is the most explosive part of the solution (i.e. has an exponential solution). Hence equation 23 simplifies to

$$y' = \lambda y \qquad (24)$$

Equation 24 is called the model ODE because stability analysis of numerical schemes are usually performed using this model problem.

3.4.1.1 Euler Method

An expansion using the Taylor series can be used to write the solution at time (t_{n+1}) about the solution at (t_n) as

$$y_{n+1} = y_n + hy_n' + \frac{h^2}{2}y_n'' + \frac{h^3}{6}y_n''' + \cdots \qquad (21)$$

where h is time step (Δt), $y_n' = f(y_n, t_n)$ is the first derivative, and (y_n'' *and* y_n''') are higher order (second and third) derivatives. The Euler method is based on only the first two terms of the Taylor series expansion and hence equation 21 simplifies to

$$y_{n+1} = y_n + hf(y_n, t_n) \tag{22}$$

The solution proceeds sequentially starting from the initial condition (y_0) and progresses (marches) in time using a time step size h. The algorithm is straightforward to program but the accuracy of the method is only first-order.

A stability analysis of the explicit Euler method can be performed by using equation 22 to solve the model ODE (equation 24)

$$y_{n+1} = y_n + \lambda h y_n \tag{23}$$

$$= y_n(1 + \lambda h) \tag{24}$$

Hence, the solution at time step n can be written as

$$y_n = y_0(1 + \lambda h)^n \tag{25}$$

For complex λ, we have

$$y_n = y_0(1 + \lambda_R h + \mathrm{i}\lambda_I h)^n = y_0 \sigma^n \tag{26}$$

where $\sigma = (1 + \lambda_R h + \mathrm{i}\lambda_I h)$ is called the amplification factor. The numerical solution is stable, if $|\sigma| < 1$.

$$|\sigma^2| = (1 + \lambda_R h)^2 + {\lambda_I}^2 h^2 = 1 \tag{27}$$

Equation 27 is the equation of a circle in the $\lambda_R h$ -$\lambda_I h$ plane, as shown in Figure 9. The circle represents the stability region from which we need to pick our λh value in order to obtain the time step size (h) so as to ensure a numerically stable numerical scheme. Thus, the Euler method is conditionally stable. The time step size (h) has to be reduced so that λh falls within the circle. If λ is real and negative, then the maximum step size is $\frac{2}{|\lambda|}$. Hence, to gain a stable solution, the step size has to be less than $\frac{2}{|\lambda|}$. In addition, the stability region circle, as shown in Figure 9, is only tangent to the imaginary axis. Therefore, the explicit Euler method is always unstable for pure imaginary λ. In conclusion, the stability analysis plays a big role in numerical solutions of differential equation.

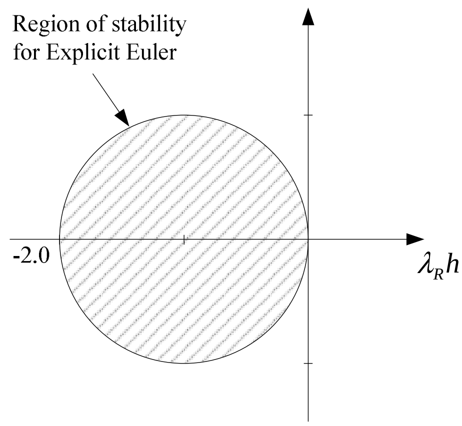

Figure 9. Stability diagram for the explicit Euler method

3.4.1.2 Fourth-order Runge-Kutta method

The order of accuracy of a numerical method can be increased by including more terms in the expansion of the Taylor series. These terms involve more partial derivatives of the function $f(y,t)$, in order to provide more information about the function at $t = t_n$. A well-known method with substantially higher accuracy than the Euler method is the so called Runge-Kutta (RK4) method. The RK4 method uses additional evaluations of the function f at intermediate points between $t_n$and t_{n+1}. The numerical algorithm for the RK4 is shown in equations 28 – 32.

$$y_{n+1} = y_n + \frac{1}{6}k_1 + \frac{1}{3}(k_2 + +k_3) + \frac{1}{6}k_4 \qquad (28)$$

where

$$k_1 = hf(y_n, t_n) \tag{29}$$

$$k_2 = hf\left(y_n + \frac{1}{2}k_1,\ t_n + \frac{h}{2}\right) \tag{30}$$

$$k_3 = hf\left(y_n + \frac{1}{2}k_2,\ t_n + \frac{h}{2}\right) \tag{31}$$

$$k_4 = hf(y_n + k_3,\ t_n + h) \tag{32}$$

A stability analysis of the RK4 method can be carried out in a similar manner to that for the Euler method discussed earlier by applying it to the model ODE problem. The resulting stability region is shown in figure 10. It is clear that there is a significant improvement compared with the stability region obtained for the explicit Euler method shown in figure 9. The RK4 method stability region extends beyond the imaginary axis.

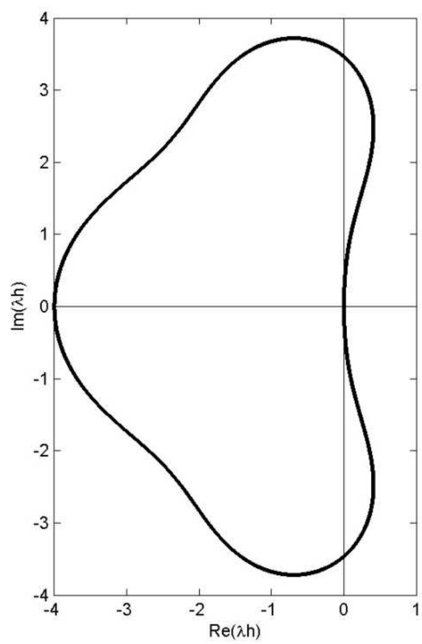

Figure 10. Stability diagrams for fourth-order Runge-Kutta method

3.4.2 Results

Figure 11 shows the numerical solutions to equations 18 and 20 using the explicit method. Results using different time step sizes are shown to highlight the stability as well as the accuracy of the method. It can be seen that the solutions for time step sizes smaller than 1 second are in good experiment with the experimental data. The solution is also stable only for a time step smaller than 1 second and blows up for a time step of 3 seconds.

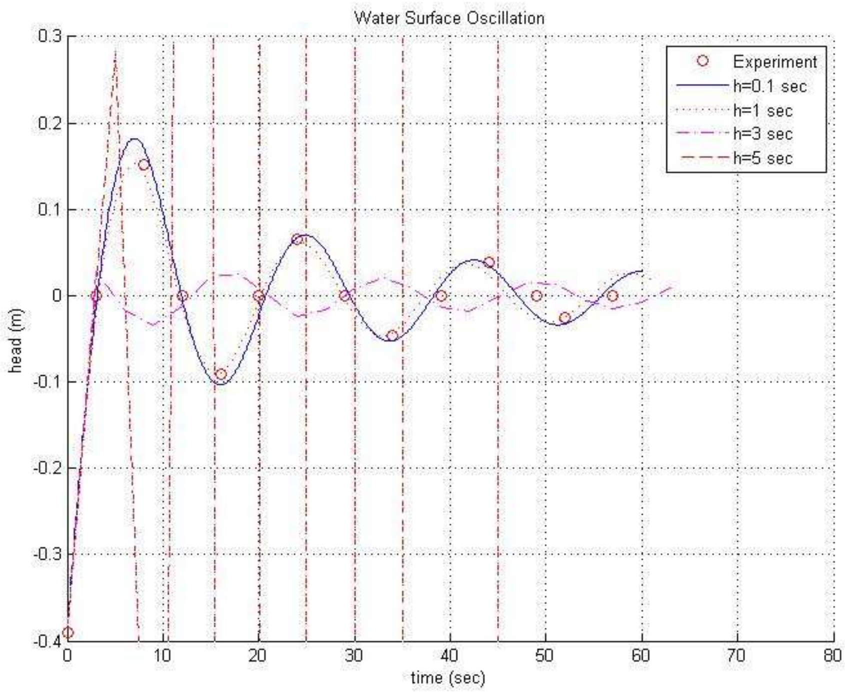

Figure 11. Water level oscillation using explicit Euler method.

Figure 12 shows the solutions obtained using the RK4 method. The solution becomes unstable only at 9 seconds, clearly indicating the superior stability properties of the RK4 method compared to the explicit Euler method. Furthermore, the solution using a time step size of 3 seconds already compares favorably with the experimental data.

Figure 13 shows the water level oscillations obtained using both numerical solution methods using a time step size of 3 seconds. It is clearly evident that the RK4 method is significantly better than the explicit Euler method.

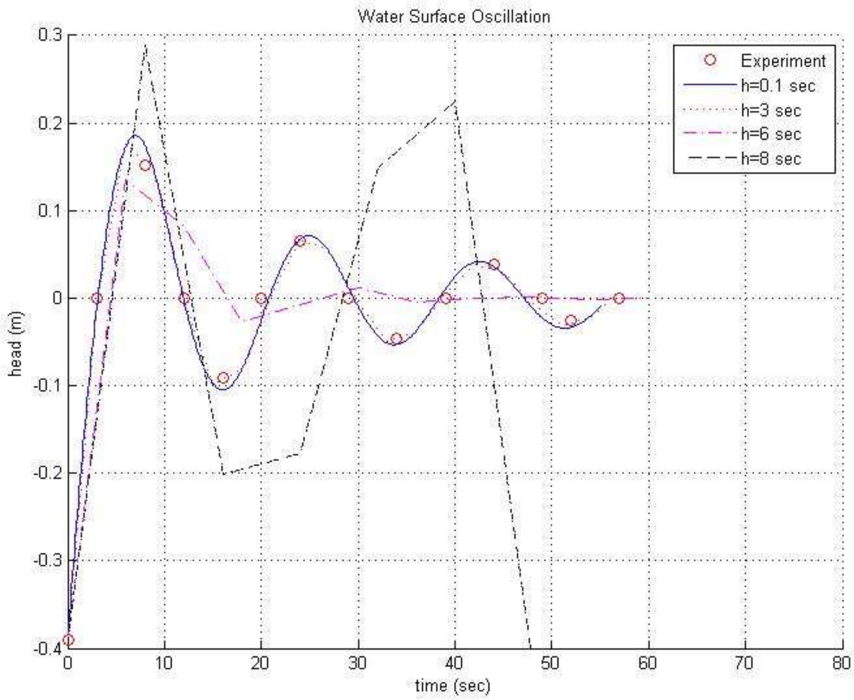

Figure 12. Water level oscillation by using fourth-order Runge-Kutta method.

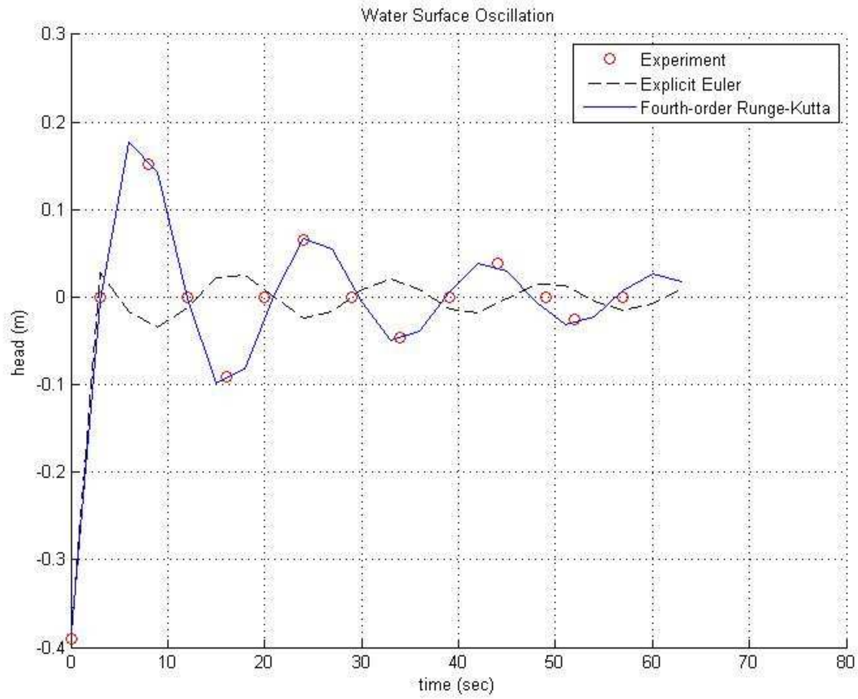

Figure 13. Water level oscillations using explicit Euler and fourth-order Runge-Kutta method

3.5 Summary

A simple surge tank example has been used to demonstrate to a very good degree the validity of using numerical methods to simulate hydraulic transients. However, it also highlights the subtleties associated with numerical methods and the need for care when using such techniques to ensure both stable and accurate results. In chapter 4, a test case problem is investigated using a packaged hydraulic transient software called Bentley HAMMER.

CHAPTER 4. TEST CASE STUDY OF THE AJDABIYA-SIRTE PIPELINE BURST ON THE MAN-MADE RIVER PROJECT

4.1 Introduction

Libya is a dry country (like its close neighbors) in North Africa, with limited water resources. In the early sixties of the last century, the search for oil in the desert of south Libya led to the discovery of large ground water aquifers. It is estimated that most of this fossil water accumulated more than 35,000 years ago. The fossil aquifer from which water is currently being transferred to the northern coast of Libya is the Nubian Sandstone Aquifer System. Some estimates indicate that the aquifer may be depleted within the next 60 to 100 years. Thus, after the discovery of this fresh ground water reserve, a plan for a major water transfer scheme was conceived to pump and transfer water from these aquifers in the southern desert interior to the northern cities on the Mediterranean coast where the majority of the population lives (estimated to be more than 80% of the population of nearly 7 million). This massive project is now known as the Man-Made River. The construction of this massive project of pipes, pumps, reservoirs, wells began in the mid 1980's (Mansor & Toriman, 2011).

In what follows, a brief overview of the Man-Made River project is first provided. This is followed by a discussion of the Ajdabiya-Sirte pipeline burst. The Bentley Hammer transient simulation software is then reviewed since it was used to investigate the possibility of whether transient pressures could have caused this damage. The surge tank simulation discussed in Chapter 3 was repeated with Hammer to provide a benchmark validation before transient simulations of the Ajdabiya-Sirte pipeline system were performed. The results of these simulations and conclusions are then provided.

4.2 Man-Made River Project

According to the Guinness World Records (2008 edition), this project is the largest underground network of pipes. It consists of 4,000 km of pipes, and more than 1,300 wells, most of them more than 500 m deep. The project supplies about 6,500,000 m^3 of fresh water per day to cities located in the north of Libya. The project is owned by the Man-Made River Project Authority and was funded by the Libyan Government. The total cost of the project is more than US$25 billion. In addition, analysts have said that the $25 billion groundwater extraction system is ten times cheaper than an “equivalent” desalination project. Figure 14 shows a schematic drawing of the Man-Made River project.

This project consists of four phases, as shown in Figure 14. The first phase of the project (Phase 1), consisted of the construction of the Tazerbo-Sarir-Sirte-Benghzi system. It is referred to as the SS/TB project. In this phase, the Tazerbo well field is connected to a 170,000 m^3 collection tank, which is linked to the first major 256 km pipeline network that transports water to two 170,000 m^3 tanks in Sarir. From Sarir, two four-meter-diameter pipelines, 380 km in length, transport water north to the Ajdabiya Reservoir. From this reservoir, water is transferred to the eastern city of Benghazi and to the City of Sirte in the west via two transfer pipelines. These pipelines feed the Grand Omar Mukhtar Reservoir in the east and Grand Al-Gardabiya Reservoirs in the west. During phase 2 of the project, 2115 km of pipeline was laid to transport water at a volume flow rate of 2.5 million cubic meters per day from the east, west, and northeast Jabal Hassouna well fields to Tarhouna and eventually to Tripoli. Phase 3 consisted of the construction of a pumping station at Kufra well field, and a 380 km pipeline linking this well field with the Sarir/Tazerbo network along with a 140,000 m^3 regulating tank, and several pump stations. Phase 4 consisted of drilling and construction of well fields and installation of pipelines for the Ghadames Azzawiya-Zuara and Jaghboub-Tobruk systems.

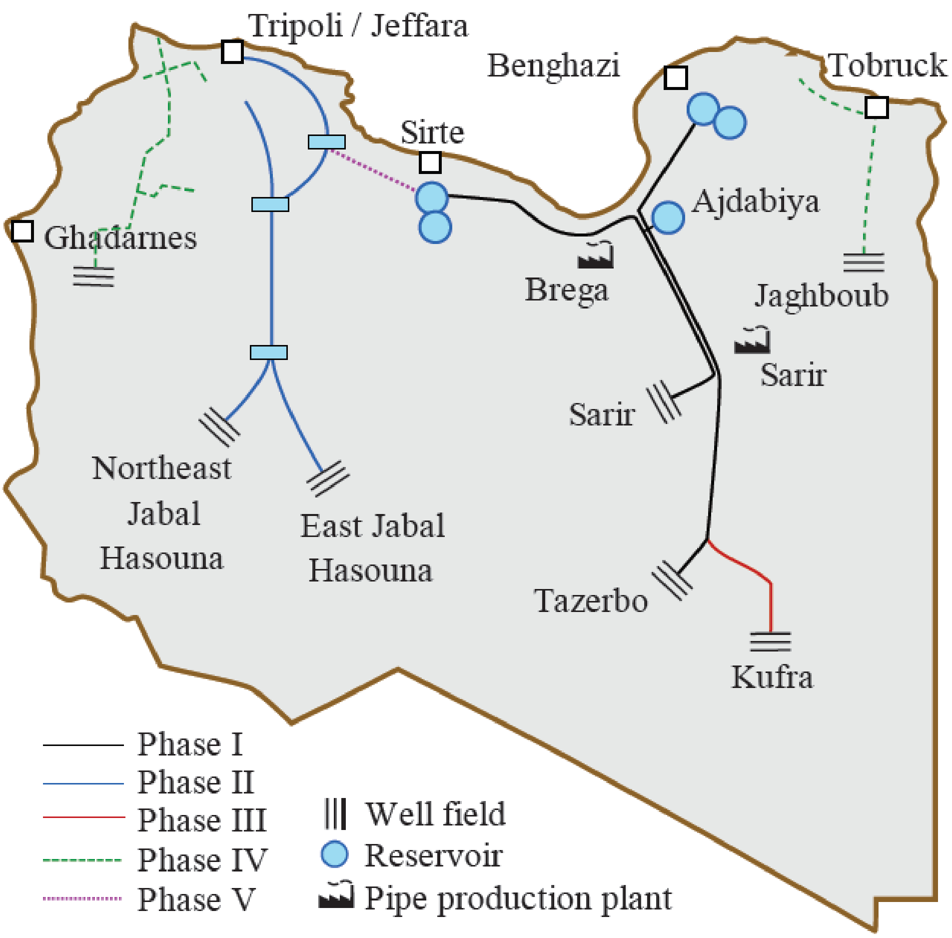

Figure 14. Schematic layout of the Man-Made River project (Wikipedia, 2013).

4.3 Ajdabeya-Sirt Pipeline

As mentioned earlier, water is conveyed from well fields at Sarir and Tazerbo through two four-meter-diameter pipelines north to the Ajdabiya holding reservoir, which can hold a total of 4 million cubic meters of water. From this reservoir, water is transferred to the eastern city of Benghazi and to Sirte in the west via two transfer pipelines. A schematic of the pipeline system linking Ajdabiya holding reservoir to the Al-Gardabiya Reservoir (6.8 million cubic meters) and

Grand Al-Gardabiya Reservoir (15.4 million cubic meters) is shown in Figure 15. The pipeline linking the Ajdabiya holding reservoir to the Al-Gardabiya Reservoir has a total length of 393 km and an inside diameter of 4 m. There is a turn-out at station 386+420, where water is also diverted to the Grand Al-Gardabiya reservoir. The distance between the turn-out and the Grand Al-Gardabiya reservoir is 2.827 km (pipe diameter = 2 m). The bar ratings of this pipeline system range from 6, 8, 10 and 12 bars.

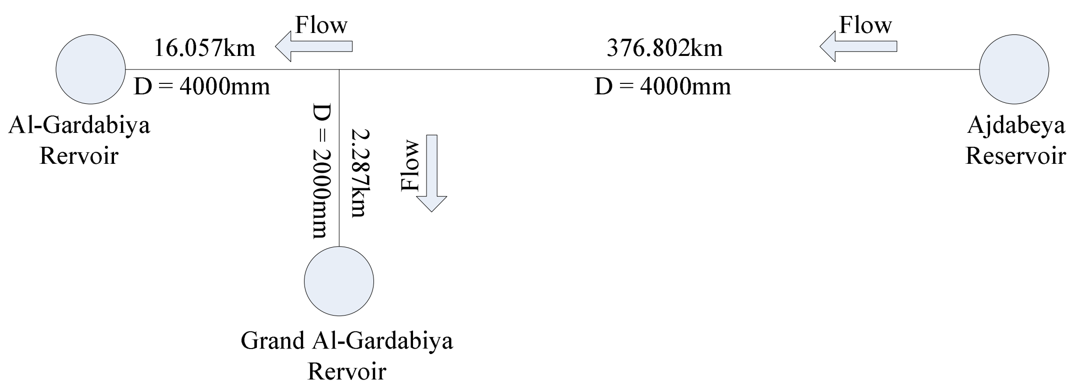

Figure 15. Schematic of the Ajdabiya-Sirt pipeline system.

4.3.1 Ajdabiya –Sirte pipeline burst

The pipeline conveying water from Ajdabiya reservoir to Gardabiya reservoir is a four-meter diameter PCCP pipe. As shown in Figure 15, this pipe ends in Gardabiya reservoir (6,800,000 m^3), but before that there is a turn out at station (386+715), where a two-meter diameter PCCP pipe transfers water to Grand Al-Gardabiya reservoir (15,400,000 m^3).

During daily operation of the transmission pipeline system between Ajdabiya reservoir and Al-Gardabiya reservoirs, the operating and control teams of the Man-Made River project found that there was significant water loss through the system based on inflow and outflow measurements. A decision was taken to close both Al-Gardabiya reservoirs and monitor the system through flow meter readings. After the inlet valve at Al-Gardabiya reservoir was closed, an order was given to the operating staff to close the inlet valve to the Grand Al-Gardabiya reservoir on February 7th, 2012. The flow to this reservoir at that time was about 290,000 m^3/day. On February 10th, 2012, the personnel in the control room at the City of Ajdabiya reported huge fluctuations in the flow meter readings at Ajdabiya Reservoir that is part of the Man-Made River Project in Libya. It was found that the transmission pipeline at station (76+820) had burst, causing a large leakage, as shown in figures 16 and 17. Based on a report prepared by the Man-Made River Project management, the amount of leakage was estimated as 200,000 m^3.

Figure 16. Pipe burst at station 76+860 in the Ajdabiya-Sirte pipeline system (personal communication, August 28, 2012).

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Figure 17. Pipe burst at station 76+860 in the Ajdabiya-Sirte pipeline system (personal communication, August 28, 2012).

The cause of this pipe burst is most likely due to a hydraulic transient event caused by the sudden valve closure. This aspect will be investigated using the Bentley Hammer software, which is reviewed next.

4.4 Bentley HAMMER V8i transient analysis software

Bentley HAMMER is a powerful, easy-to-use program which helps engineers and researchers analyze hydraulic transients in complex pumping systems and piping networks. In addition, Bentley HAMMER helps engineers to understand their water system by making it easy to evaluate different operating scenarios in order to assess the corresponding transients that occur. Bentley HAMMER is based on technology first created by GENIVAR (Formerly Environmental Hydraulics Group Inc.).

Bentley HAMMER is a graphical interface software which makes it easy to quickly lay out the schematic of a complex network of pipes, tanks, pumps and surge control devices. Steady state models from other software such as WaterCad or WaterGEMS can be directly used in Bentley

HAMMER, saving user time and eliminating transcription errors. Bentley HAMMER V8i uses the Method of Characteristic (MOC) to solve the governing equations (discussed in chapter 2) that describe the hydraulic transient phenomenon. Essentially, the MOC is based on the concept that solutions to the governing equations can be expressed graphically in space-time plots as characteristic lines representing wave propagation directions. In Bentley HAMMER V8i, the following capabilities are included:

- Boundary conditions are expressed as algebraic and/or differential equations based on their physical properties. This is carried out for every hydraulic element in the model and solved along with the characteristic equations.
- Bentley HAMMER V8i is capable of modeling cavitation whereby the fluid can flash into vapor at low pressures.
- The length of computational reaches are set to achieve sufficient accuracy without resulting in too small a time step, which leads to excessively long simulation times. The Bentley HAMMER V8i automatically sets an optimal time step based on pipe length, wave speeds, and overall system size in order to achieve model results faster.
- Friction losses are assumed to be concentrated at solutions points. In addition, Bentley HAMMER contains different models that can be implemented, ranging from steady-state to quasi-steady to unsteady friction formulations.

4.4.1 Transient analysis friction method

In HAMMER, a hydraulic transient analysis begins with initial conditions that are based on steady-state calculations. In a steady-state calculation, the heads and flows are computed for every time step in the system. Before the transient analysis is carried out, HAMMER automatically determines the friction factor based on the following information: If a pipe has zero flow, HAMMER uses the friction coefficient specified in the pipe physical properties or based on user entry of a Darcy-Weisbach friction factor; if a pipe has a non-zero flow at the initial steady-state, HAMMER automatically calculates a Darcy-Weisbach friction factor, based on the heads at each end of the pipe, the pipe length and diameter, and the flow in the pipe. HAMMER always utilizes the Darcy-Weisbach friction method in performing the hydraulic transient calculations.

Distributed frictional losses are assumed to be concentrated at discrete computational points, which are treated as hypothetical inline orifices. Therefore, at every calculation point, there are two heads: one on the upstream side and one on the downstream side, as indicated in figure 18 (Bergeron, 1961). These differ by the head loss between adjacent calculation points. The addition of the nonlinear Darcy-Weisbach equation to the system of characteristic equations makes the task more complicated to advance the solution forward in time, and leads to an approximation in terms of the friction coefficient, which is typically small.

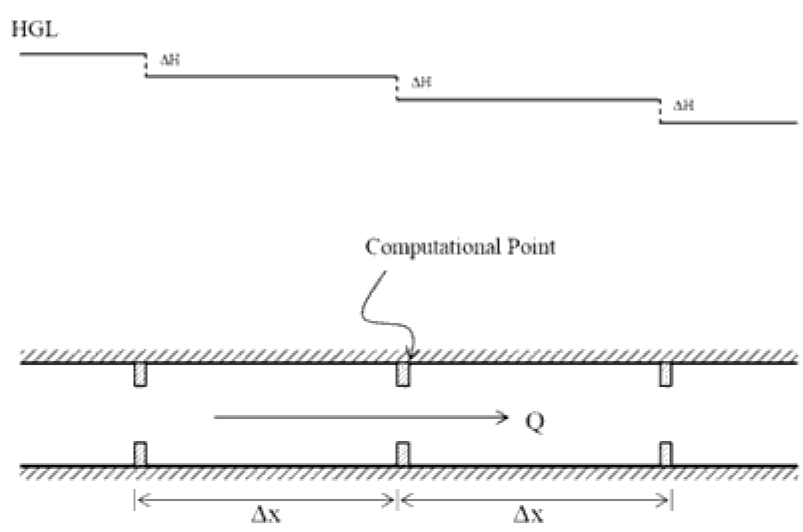

Figure 18. Representation of friction losses (Bentley, 2013)**.**

In the quasi-steady friction approach (Fok 1987), the Darcy-Weisbach coefficient at any point depends on the state of the system at the previous time step. For subsequent time steps, the Reynolds number is computed at each point on the basis of the previous iteration's velocity and then an updated coefficient is ascertained. The quasi-steady friction method is virtually an unsteady method, even though one based on steady-state friction factors. The quasi-steady method is more computationally demanding than the steady-state friction analysis.

The fluid friction increases during hydraulic transient events compared to a steady-state situation. This occurs because rapid changes in transient pressures and flow increase the turbulent shear stresses. Bentley HAMMER V8i is capable of tracking the effect of fluid acceleration in order to estimate the attenuation of transient energy more closely than would be possible with quasi-steady or steady-state friction. This unsteady friction method developed by Vitkovsky et al. (2000) is now the recommended method for computing unsteady frictional losses in HAMMER. Computation effort increases significantly if transient friction must be calculated for each time step. Hence, this results in long model-calculation times for systems with hundreds of pipes or more. However, transient friction has little or no effect on the initial low and high pressures, and these are usually the largest values reached in the system. This can be seen in figure 19 from a HAMMER simulation result comparing steady, quasi-steady and transient friction methods.

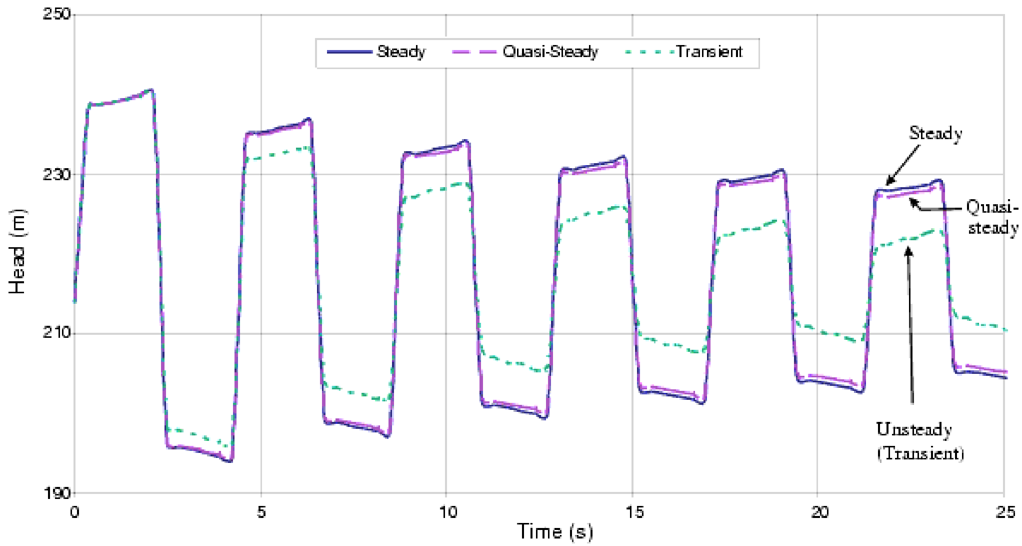

Figure 19. Bentley HAMMER V8i results for steady-state, quasi-state, and transient friction method (Bentley, 2013)**.**

Hence, it seems that the steady-state friction method yields conservative estimates of the extreme high and low pressures that usually govern the selection of pipe class and surge-protection equipment. However, if cyclic loading is an important design consideration, the unsteady friction method can yield less-conservative estimates of recurring and decaying extremes.

Figure 20 shows the Bentley HAMMER V8i interface. Part of the Ajdabiya-Sirte pipeline system schematic setup is shown in the figure.

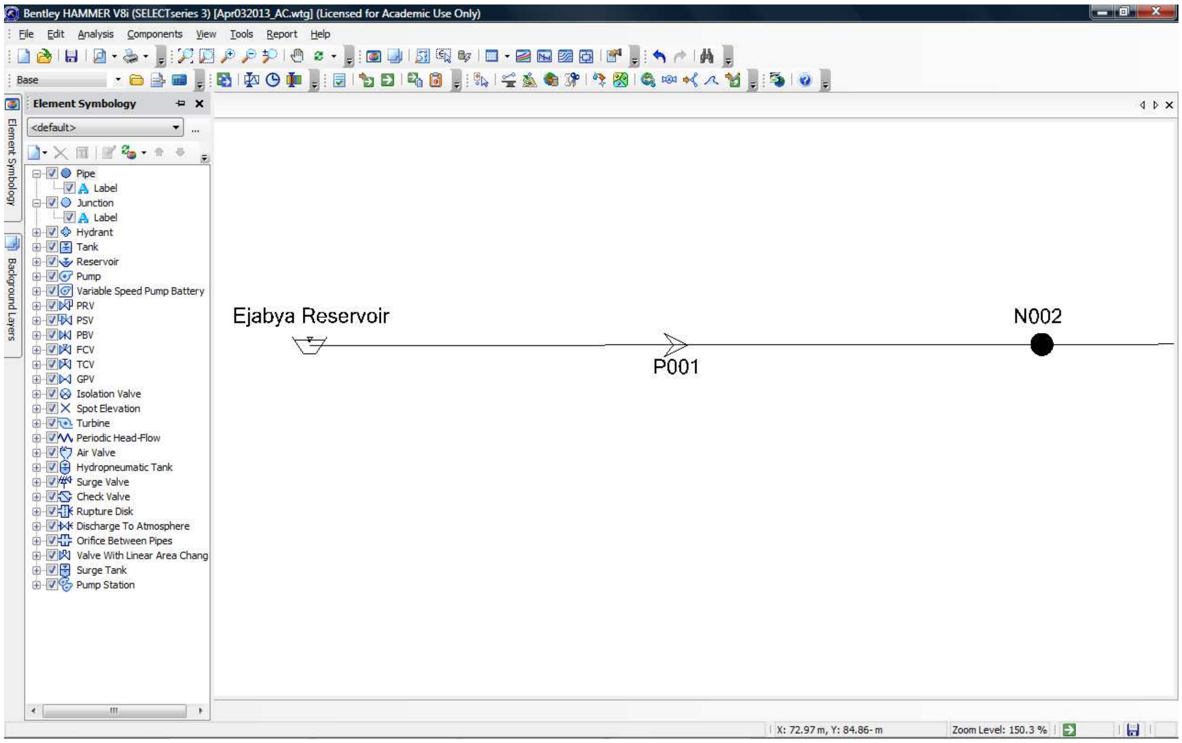

Figure 20. Bentley HAMMER V8i graphical user interface.

4.5 Surge tank simulation

The surge tank example discussed in chapter 3 was modeled using HAMMER. As shown in figure 21, the oscillations of the water surface obtained from Bentley HAMMER V8i software are in good agreement with the experiment results. With this validation in place, this software was used to simulate different operating and valve closure scenarios for the pipe burst problem on the Man-Made River.

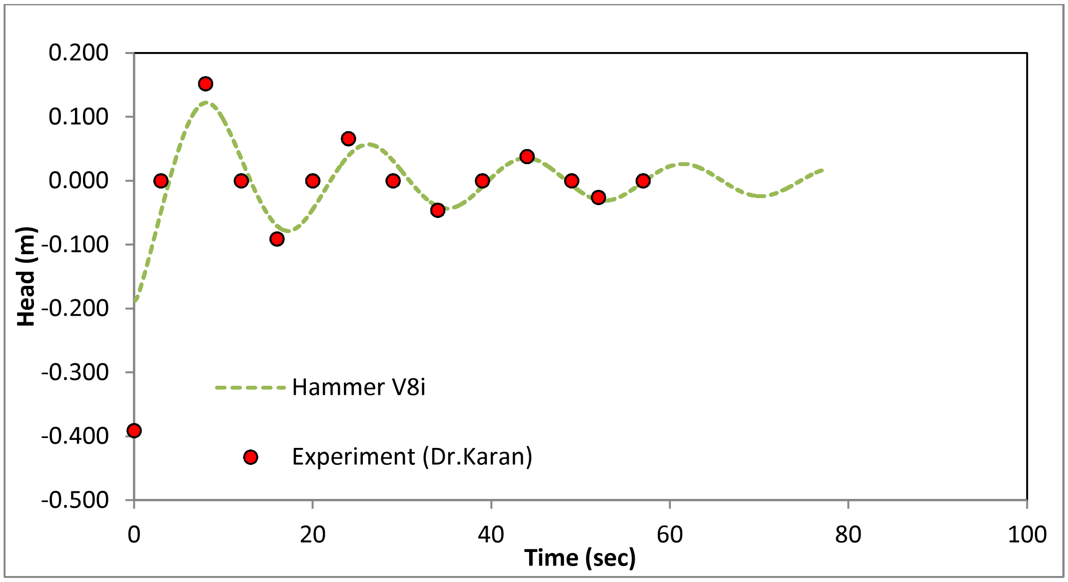

Figure 21. Comparison between water surface results obtained from HAMMER and experimental data (shown as red circles) in a surge tank

4.6 Hydraulic transient simulations of the Ajdabiya –Sirte pipeline

A model was constructed in Bentley HAMMER V8i based on the physical properties of the whole system in order to simulate a number of scenarios based on different water levels in both reservoirs and different times taken to close the downstream valves. The maximum and minimum operating levels in the reservoirs during the period of the pipe burst are shown in Table 2. In addition, simulated times for valve closure at Grand Al-Gardabiya reservoir are shown in Table 3.

Table 2: Reservoir water levels in Ajdabiya and Grand Al-Gardabiya reservoirs

	Ajdabiya reservoir	Grand Al-Gardabiya reservoir
Maximum operating level (m)	98.55 (A)	54.76 (C)
Minimum operating level (m)	91.55 (B)	37.76 (D)

The letters A, B, C, and D are used for convenience to represent the cases that will be simulated based on the corresponding water levels.

Table 3: Simulated times for valve closure

Time to closure (TC) of downstream valve (min)
20
40
80
160
320
640
1280

A scenario with water level (A) in Ajdabiya reservoir with corresponding water level (C) in Grand Al-Gardabiya reservoir will be termed AC. Similarly, other scenarios can be obtained; i.e., AD, BC and BD, respectively. For each case, different valve closing times are used, as shown in Table 3. As a result, there are 4 cases of operating water levels, and 7 valve closing times for each case. Hence, a total of 28 simulations were performed.

4.7 Results

The mechanical strength of this pipe (bar rating) is designed to be 12 bar (120 m head of water). This means the pipe can only withstand a maximum of 120 m of pressure head. Therefore the key results that are presented in the figures that follow are the pressure transients expressed in head of water in meters.

4.7.1 Scenario AC with TC of 20 min, 80 min, 320 min and 1280 min.

This scenario is based on an operating water level at Ajdabiya reservoir of 98.55 m, case (A) as shown in Table 2, and a water level in Grand Al-Gardabiya reservoir of 54.76 m, case (C). The time to close the flow control valve at the inlet of Grand Al-Gardabiya reservoir is varied from 20 min to 1280 min, as shown in Table 3. The simulation results for the pressure fluctuations for 20 min, 80 min, 320 min and 1280 min are shown in Figure 22. The red line represents the maximum pressures; the blue line represents the low pressures; the black line represents the normal operating pressures; and the green line represents the initial pressures. The results show that the maximum pressures in the pipeline are located between 50,000 m to 100,000 m, in which the pipe burst occurred, and which experienced pressures greater than the bar rating of the pipe for all cases except the case with a valve closure time of 1280 min. When the closing time was taken to be 20 min, the maximum pressure in the locality of the pipe burst went above 14 bars (see figure 22 (a)). Such a scenario can cause the pipeline burst that occurred.

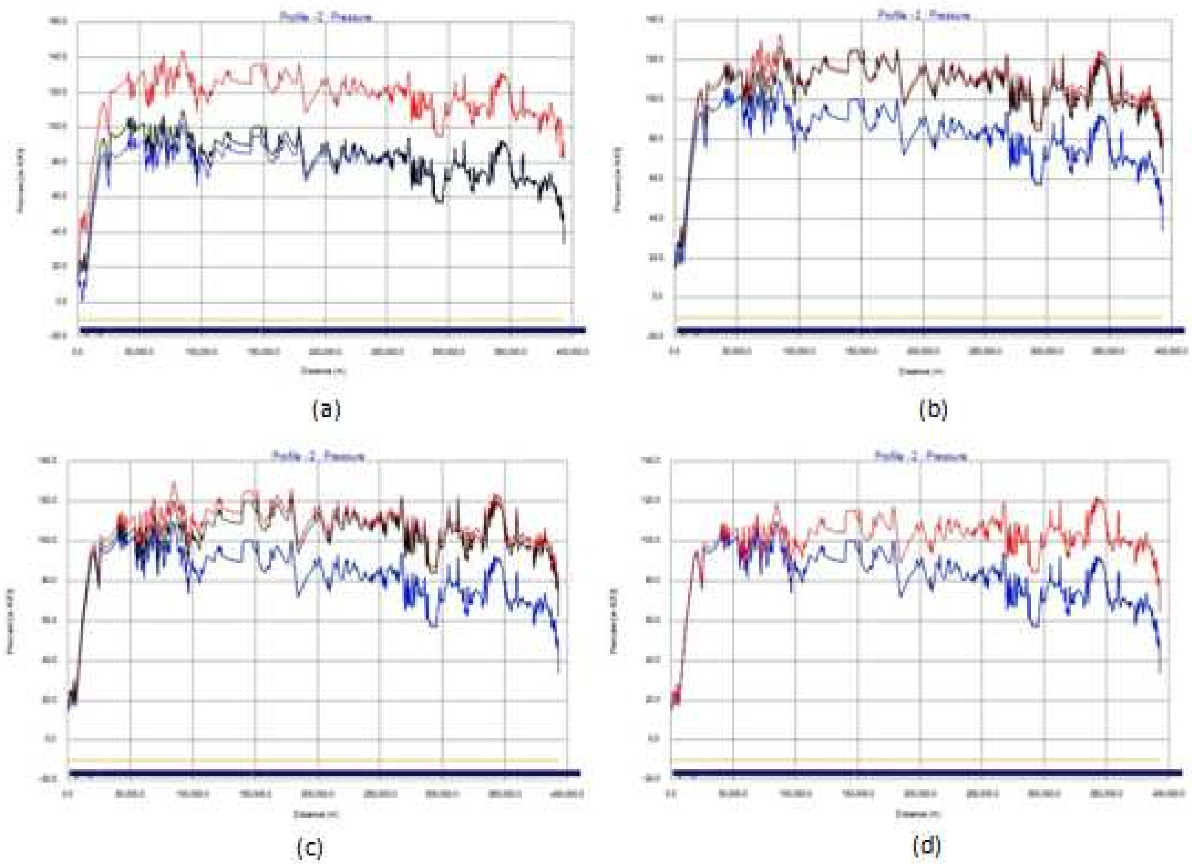

Figure 22. Pressure profiles along the pipeline from Ajdabiya reservoir Grand Al-Gardabiya reservoir for case AC with valve closure times of: (a) 20 min, (b) 80 min, (c) 320 min and (d) 1280 min, respectively.

4.7.2 Scenario AD with TC = 1280 min (21 hr and 20 min)

Similar to the results for case AC with different valve closure times, simulation results of the transient pressure for case AD with a TC of 1280 min were found to fall below the pressure rating of the pipeline, as shown in figure 23.

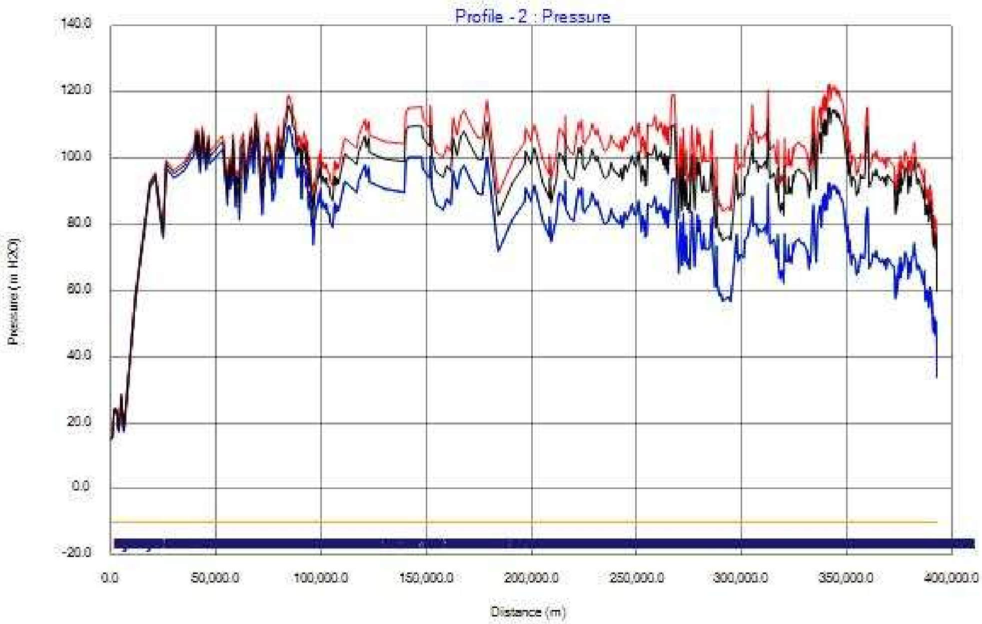

Figure 23. Pressure profiles along the pipeline from Ajdabiya reservoir to Grand Al-Gardabiya reservoir for case AD with a TC of 1280 min.

4.7.3 Scenarios BC and BD with TC = 640 min.

Similarly, for scenarios BC and BD with TC = 640 min, the maximum pressures are less than the bar rating in the region of burst pipe, as shown in figure 24.

4.8 Summary

The pipeline system connecting the Ajdabiya reservoir to Grand Al-Gardabiya reservoir as well as Al-Gardabiya reservoir is a very long pipeline network. It is clear that closure times required for shutting down such systems are in the range of hours to nearly a day. Based on the simulation results, it is plausible that the control valve was shut rather rapidly (on the order of minutes), triggering a transient event that must have caused the maximum pressures to rise well above the pressure rating of the pipeline material in the region where the burst occurred. In hind sight, a simple hydraulic study such as that presented in this study could have prevented this failure.

Again, this example highlights why it is important to be fully aware of hydraulic transient phenomena in closed conduit systems.

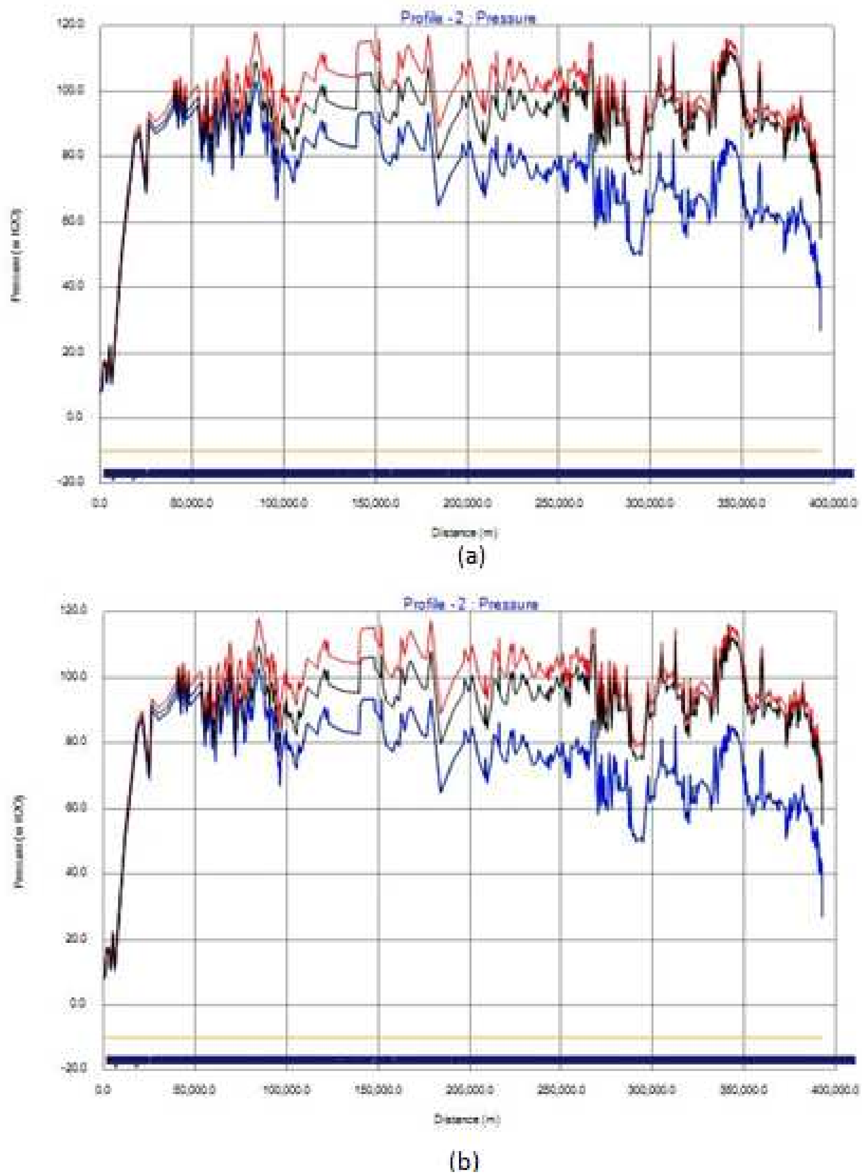

Figure 24. Pressure profiles along the pipeline from Ajdabiya reservoir to Grand Al-Gardabiya reservoir for (a) case BC and (b) case BD, with a TC of 640 min.

CHAPTER 5. CONCLUSIONS

5.1 Summary

Hydraulic transients in closed conduit flows and in other types of fluid flows are important to study. They occur as a direct result of rapid variations in the flow field initiated by changes in operational conditions such as the sudden closure of valves in pressurized (closed-conduit) systems. In this study, an overview of the hydraulic transient problem was presented to highlight the many salient issues associated with this phenomenon, ranging from causes to consequences and control measures. The governing equations for analyzing transient flow were presented for a closed-conduit flow.

Two example problems were studied via two different numerical simulation techniques. The first problem focused on simulating the water surface oscillations in a surge tank due to the sudden closure of a downstream valve. The purpose of this exercise was to demonstrate the usefulness of computational techniques to simulate hydraulic transients and at the same time expose some of the subtleties associated with numerical methods - specifically the stability and accuracy of the chosen numerical scheme, and hence the need for care when using such techniques to ensure both stable and accurate results. Comparisons of numerical results with experimental data showed very good agreement.

The second example problem was more of a case study of a pipeline burst that occurred in the world’s largest network of underground pipes and aqueducts – the Man-Made River Project. A widely used commercial software called HAMMER was employed to model the flow in one of major legs of this huge water transfer system in order to investigate the possibility of the failure as a result of extreme pressure build up in the pipeline from transients. Using different operating water levels in the upstream and downstream reservoirs as well as different time durations for valve closures, the pressure fluctuations in the locale of the pipe burst were investigated. It was found that the maximum pressure easily exceeded the bar rating of the pipeline by 2 bars (or nearly 20 %). This *a posterior* study reinforces the need for hydraulic engineers to systematically investigate the effects of hydraulic transients, especially for very large water systems, in order to prevent catastrophic damages.

5.2. Suggestions for further work

There is always more that can be done in this very challenging and important field of research. Clearly, the analyses presented in this thesis are based on many simplifying assumptions that preclude the investigation of flow features such as helical vortices which can influence the pressure, shear stress and velocity distributions in closed-conduit flows. Current models do not account for such effects and, in order to do so, further research must be done to gain more insights into the physical mechanisms for the creation of vortices and other flow structures that are often observed during transient events. Of course, modeling turbulence is yet another very important problem in transient flows as it is directly related to shear stresses. It is evident that research on hydraulic transients will continue despite the significant advances that have been made since the seminal work of Joukowsky over 100 years ago. Moreover, the procedure carried out in this thesis to investigate the pipe burst can be used for more complex pipe networks in order to study the effect of hydraulic transients.

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