



Investigation of Surge and Transient Flow Conditions in the Water Transmission Line at Ahmadu Bello University

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ABSTRACT

Surges and water hammer, a flow condition characterized by rapid changes in velocity and pressure over time, have the potential to damage a water transmission pipeline if it lacks adequate water hammer protection devices. The hydraulic behavior of a redesigned portable water transmission line that flows from ABU Waterworks to Senate Tower was analyzed. There are many surges and water hammer analysis methods, but this dissertation work employed the graphical method (Parmakian charts), and the Allievi equations were used for the program using the MATLAB and Bentley Hammer V8i software to investigate water hammer in a pipeline. The research revealed that during pump shutdown or power failure at the motors, the system could experience a water hammer pressure rise of 36.428 m above the normal pumping head of 52.04 m at a reverse speed of 4025.6 rpm within 17.71 seconds of power failure. Instantaneous check valve closure consequent to power failure or sluice valve closure within a period of less than 2.53 seconds could equally lead to a pressure rise of 61.64 m above the normal pumping head. These conditions correspond to 70% and 118% of the normal pumping head, respectively, contrary to the pump manufacturer's recommended maximum head rise due to a 12.4% shut-off.

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INTRODUCTION

The vast majority of people have immediate or circuitous thoughts regarding the concept of a water hammer. The sound of a slamming clamor reverberates throughout the channels in our various homes when we take a shower or turn off the water tap. The water hammer phenomenon is a critical issue in pipeline designs and operations; it is as essential as the normal steady-state pipeline plan itself.

Customarily, the analysis of water hammers has not been given the due attention it deserves in our engineering curricula and what it requires for effective, safe, and economic design and operation of pipelines. The transient analysis itself has fundamentally been presented to engineers as difficult and complicated to include in

pipe system hydraulic design (Al-Zaidi & Ismaeel, 2022; Wood, 2005).

From the mid-1900s to the present, various methods have evolved, starting with a graphical technique and culminating in fully intelligent computer-based programming capable of modeling transients in real-time (Ferrante, 2018). According to Haghiabi (2017), safe and efficient operation of pipeline systems is a challenging dynamic task. This is due to the difficulty in anticipating or predicting the effects of changes in the operation of the pump, valve, tank, or other facility states before the ultimate catastrophic effect. The design and operation of a water system's transmission lines differ from system to system due to so many factors (i.e., pipe characteristics, elevation, among others).

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Therefore, the water hammer of this transmission line, as shown in Figure 1, will be peculiar to the pipeline configuration.

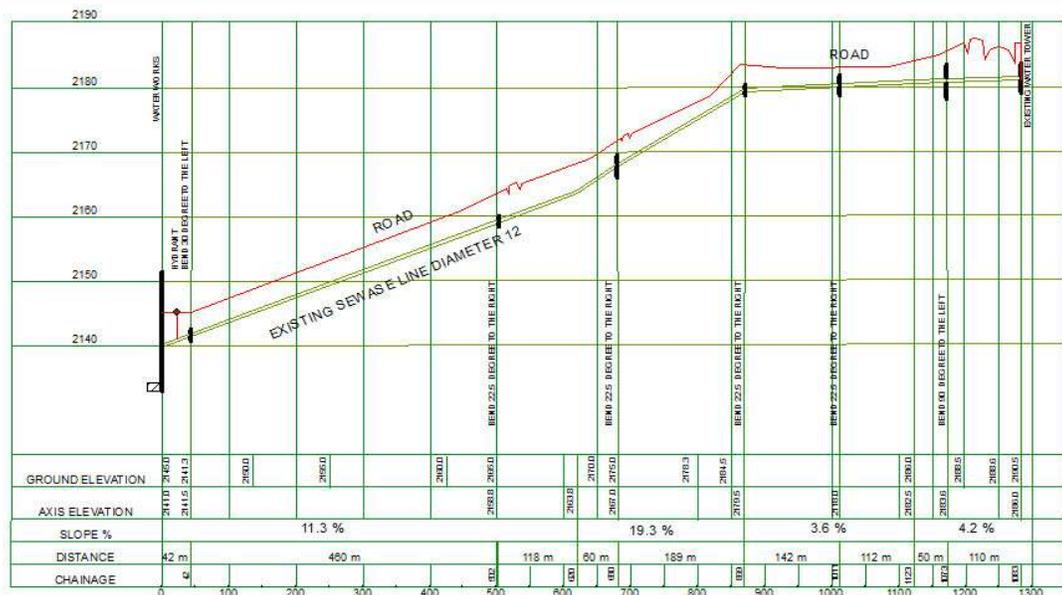


Figure 1: pipeline elevation profile of ABU Transmission line to the Senate tower.

Since 1977, the ABU community has installed water transmission lines. The ABU community's water demand is increasing as the pipeline ages, necessitating continuous pumping and frequent valve operations, making the impact of demand variations irreversible. To prevent or reduce the possibility of failure associated with these pressure variations, it is important to analyze and, if required, design an operational procedure for the pipeline system (Essaidi & Triki, 2021; Triki, 2016). Therefore, the need for guidance on water hammers is not only due to their possible effect on water losses, but also to their contribution to the safe, cost-effective, and energy-saving operation of water conveyance systems, as well as the potential life-threatening dangers associated with transmission line failure (Essaidi & Triki, 2021; Ostad-Ali-Askari, 2021). In 2011, a redesign of the entire system took place because all the water transmission lines, with the exception of the one leading to the Senate building, were not fulfilling their primary function of

conveying water to reservoirs. However, the redesigned system's transient conditions remain unexplored, leading to an unknown level of pressure oscillation.

MATERIALS AND METHODS

System of Data Collection and Analysis

Water hammer problems are more often than not solved using graphical design charts and analytical and arithmetic methods. All these may involve the use of computer models. Each water hammer condition is a response of the fluid to deviations in the hydraulic equipment that controls and conveys the fluid, or in the surrounding environment that influences the flow. GivAny of the methods can be used to find out things about the fluid like its speed, discharge Q , and pressure head H as long as details about the system and the event that caused the water hammer are given. se parameters defined the magnitude of the transient event and helped the hydraulic engineer

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establish the design criteria for the system under study.

The hydraulic behavior of ABU water transmission lines that flows from High Lift Pump to Senate Tower Reservoir will be analyzed using design equations and charts by Allievi and Parmakian respectively in conjunction with a computer program. We wrote the computer program using the MATLAB software interface, and conducted the steady-state analysis with the Bentley Hammer V8i Series 4 device. We will consider both steady-state and transient conditions in the analysis, and gather all the necessary input data from the design drawings, ABU Water Works, and manufacturer's standards,

including pump characteristics, pipeline profiles, reservoir operating data, fitting types, and locations, among others.

The Ahmadu Bello University (ABU) Water Supply Scheme

The inability of the Zaria Water Works to provide its growing population with potable water led to the commissioning of Ahmadu Bello University Waterworks in 1977. The Kubani dam headworks, located approximately 810 m from the water treatment plant, currently supplies water to ABU. The associated characteristics of Kubani Dam headworks are depicted in Table 1.

Table 1: Characteristics of the Kubani Head-Works and Accessories

| Characteristics | present scheme state | new as re-designed |
|-----------------------------|-------------------------|-------------------------|
| Water source | | |
| River | Kubani | Kubani |
| Intake facility | | |
| Capacity of intake | 190m ³ /h | 230m ³ /h |
| Pump head | 45m | 60m |
| Pump output | 30kw – 45kw | 45 - 55kw |
| Conveyance facility | | |
| Pipeline diameter | 375mm | 400mm |
| Pipeline material | Asbestos | Ductile Iron |
| Service reservoir | | |
| Elevated tank major storage | 1500m ³ | 1500m ³ |
| Total head | | |
| Total population | | |
| Planned serviced student | 35,000 | 35,000 |
| Planned serviced staff | 65,000 | 65,000 |
| Present water supply | | |
| Daily | 4,560m ³ | 13,680m ³ |
| Annual | 1,664,400m ³ | 4,993,200m ³ |
| Water demand | | |
| Planned student | 75lpcd | 100lpcd |
| Planned staff | 100lpcd | 150lpcd |
| Lawn and fighters | | 360,000lpcd |

(Source: ABU Water Work)

The Ahmadu Bello University Water Supply Rising Mains

The new design's ABU rising main consists of a 400-meter-diameter ductile iron pipe spanning from the high lift to the senate tower.

Twelve other major tanks also receive water from this rising main with various reduced diameters. The newly designed rising main will transmit water from the clear water reservoir through two sets of KSB eternun pumps to discharge 230 m³/h of

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water at an operating head of 60 m, a rated power between 45 and 55 kw, and an operating efficiency of 75%. The 120-m³ tank is standing on 18-meter steel stanchions in front of the Ahmadu

Bello University Senate building, about 200 meters away. Figure 2 shows the profile configuration of ABU mains from the clear water tank to the senate tower.

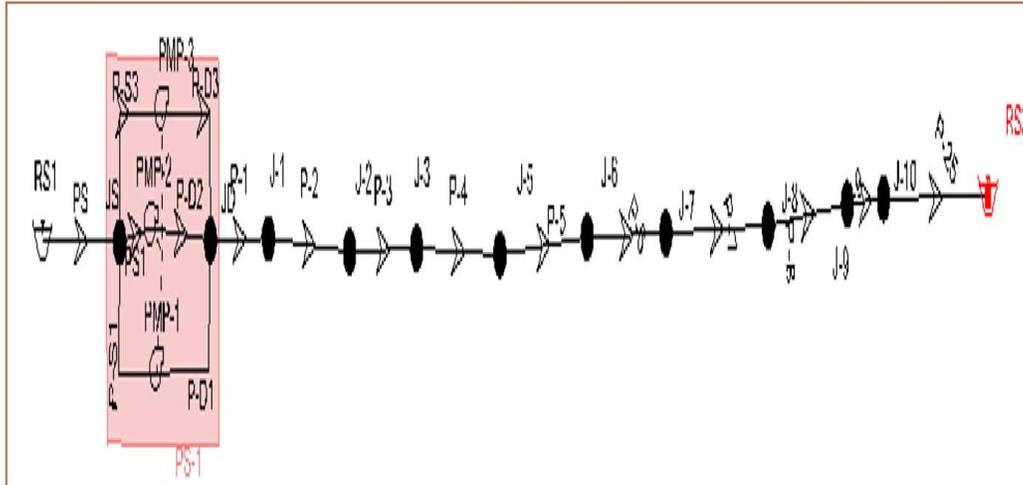


Figure 2: ABU Water Transmission Line configuration

DESIGN VELOCITIES

Maximum Velocities

The choice of design maximum velocity results from a compromise between the choice of reduced pipe diameter and, hence, a reduced pipe cost with higher velocity and the higher cost of water hammer protective devices associated with higher velocities. The engineer's expertise determines whether to choose a larger diameter pipe, which would result in a higher pipe cost at lower velocities and a cheaper cost for water hammer protective devices.

The design of the pipeline under consideration already accommodates 400 mm of Ductile Iron (D.I.) pipe material. Ductile iron pipe material allows for higher operating pressure (greater than 20 m head of water) and tolerates velocities as high as 3 m/s.

The Ahmadu Bello University, Zaria Water Works pumping system operates on two pumps at 230 m³/h, or a maximum initial velocity of 0.508 m/s. Although the pipeline slopes at various segments of the rising main are in excess of those required to overcome pipe friction losses, the maximum velocity of 0.508 m/s is still well

below the recommended pipe velocity of 1.5 m/s (TCH, 1998).

Minimum Velocities

The need to flush out any sediment or debris entering the pipe system during normal operation informs the recommended minimum velocities. Under particular operating conditions, flow velocities will drop below these guidelines, but deposited material should, in most cases, be adequately secured when normal flow resumes (EL-Turki, 2013).

Standard texts indicate scouring velocities for non-cohesive materials in the range of 0.3 m/s for silts and 0.5 m/s for fine sands, which are likely to be the most troublesome material entering the pipeline. A previous study revealed that pipe size plays a significant role in determining the minimum and maximum permissible velocity for pressure pipe network systems (Peretti-Pezzi, 2011). A minimum and maximum velocity of 0.50 m/s and 2.5 m/s, respectively, are acceptable for pipes of a nominal diameter of 400 mm. This corresponds to a pump

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discharge of 226.22 m³/h at a minimum of 75.2% efficiency (Peretti-Pezzi, 2011; Pezzi et al., 2015).

System Design Pressure

The Ahmadu Bello University, Zaria Water Works rising main and fittings have been designed for ductile iron pipes and a mixture of cast iron and ductile iron pipe fittings and appurtenances. PONT-A-MOUSSON, a French manufacturer of ductile iron pipes and accessories, recommended that for a nominal diameter of 400 mm, corresponding to $k = 9$, the maximum pressure, excluding the surge pressure, is 34 bars for the pipes and 35 bars for the fittings (Dattin, 2014). We recommend setting the maximum pressure, including surge pressure, at 20% of the maximum pressure, excluding surge pressure, or 20% of 34 bars + 34 bars, or 40.8 bars (Durham et al., 2001). All appurtenances, such as air valves, washout valves, sluice valves, etc., are recommended to have a nominal pressure (NP) of 8 to 16 bars. Since all pipes, fittings, and appurtenances must work as a unit, the least of these components' operating pressures constitutes the system design pressure. Thus, the system design pressure is 8 to 16 bars, corresponding to the maximum operating pressure for the fittings.

Consequently, operating pressure over 8 bars will certainly cause system failure at those fittings with less than 10 bars of operating pressure (Afshar & Rohani, 2008; Wichowski, 2006). Such a failure could result in damage to the appurtenances' components, their removal from the system, or leaks at the joints. However, the pipes and fittings could withstand a maximum permissible pressure of 40.8 bars, ranging from 42 bars to 48 bars. Hence, the system-designed pressure is between 8 and 16 bars (1 bar = 100 kN/m² = 10.19 m of H₂O) or 101.9 m of H₂O.

Analysis of the Ahmadu Bello University, Zaria Water Works Rising Mains

The rising main under consideration at Ahmadu Bello University, Zaria Water Works is a buried main with an upstream centrifugal pump boundary condition. A steady-state operation of these pumps will not lead to surge conditions. In

such a system, the anticipated frequent causes of water hammer are discussed. We used a Bentley Hammer V8i Series 4 to operate these pumps in a steady-state. We obtained the following data: Pipeline Length, Pipeline Diameter, Pipeline Elevations, Head Losses, Wave Speed, Pump Data, and Reservoir Data.

Starting and Stopping of the Pumps

The system generates shock waves when the pumps start and stop, or when the pump motors experience a power failure. Cutting the power causes the pump motors to slow down, altering the head-discharge and torque-discharge characteristics, which in turn triggers the pump-pipeline system to generate shock waves.

(Rezghi & Riasi, 2016). The kinetic energy of the pump and motor's rotating elements maintains the flow against the discharge head. Because this energy is so small, pump speed and discharge line flow decrease rapidly. Equations 1 and 2 give the discharge at various times between the power cut-off and the complete pump shut-off.

$$Q_{i+1} = Q_0 \frac{N_{i+1}}{N_0} \quad (1)$$

$$H_{i+1} = H_0 \left(\frac{N_{i+1}}{N_0} \right)^2 \quad (2)$$

The head and discharge generated as the pump rundown progress are dependent on the rotational inertia of the rotating parts and the torque exerted by water in the pipeline, as well as pipeline hydraulics. When a power failure occurs at the pump, the motors can lead to four possible flow and rotational configurations through the pump. The occurrence of these configurations depends on the presence of bypass valves, check valves, and regular line valves.

- The flow passes through the pump in a forward direction, causing the pump to rotate forward.
- Flow in the reverse direction occurs while the pump still rotates forward.
- When the pump is rotating backward, flow occurs in the reverse direction.
- Finally, flow in the forward direction occurs while the pump is rotating backward.

The Ahmadu Bello University, Zaria Water Works pumping main has no forward

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bypass around the pump, which could open in the event the head drops to zero under forward flow conditions; rather, along with it are one 300-mm-diameter check valve each near the pumps and a 400-mm-diameter check valve just at the inlet into the rising main. The suction pipe segments also terminate at the foot valve. We expect these protective devices to automatically shut off the flow when the water column reverses. This implies that no reverse flow occurs when the pump shuts down, effectively safeguarding the pump from damage resulting from turbine operation. However, when the gate is closed, the entire water column between the check valve location and the reservoir oscillates back and forth, continuously knocking at the gate disc each time a wave reaches it. This knocking or hammering occurs with great force, subjecting the gate disc to stress and strain. This process gradually erodes the disc, particularly at the hinges. Only adequate inspection and maintenance of the check and foot valves can guarantee their protective role. Experience has demonstrated that fatigue can break and wash off the gate discs of these valves in the flow, leaving the pump susceptible to reverse flow conditions and accelerating damage to the pump impeller and casing (Colón-Díaz, 2009; Mondal, 2010). Consequently, other protective devices must be provided to complement the functions of this valve and better protect the pump-pipeline assembly against surges and water hammer conditions.

Near Instantaneous Valve Closure

Install as many sluice valves as possible at various locations between the foot valve at the base of the suction pipe and the float valve at the reservoir's inlet. These valves are mainly operated to attain partial or complete closure of the line, either instantaneously or slowly. During the closure period, the function in Equation 3 relates the valve area or flow passage area to the elapsed time t and the total time the valve operates to achieve complete closure (T).

$$A_{\text{valve}(t)} = A_{\text{valve}(t_0)} \left(1 - \frac{t}{t_{\text{close}}}\right)^3$$

Parmakian (Parmakian, 1958) in his solution of the water hammer equation by the graphical method indicated that the value of the

head rises or drop at the gate can range between $0.3H_0$ to H_0 . In the case of the Ahmadu Bello University, Zaria Water Works pumping main this translates to 0.3 of 60m to 60m or 18.0m to 60m and when added to the static head of 60m, implying that the surge pressure resulting from valve operations can range between 78.0m to 94.62m of H_2O . This is much closed to the maximum design pressure of 101.9m H_2O and also the overburden pressure is very high since most of this pipeline is beneath the campus access Roads, which indicates that the design pressure cannot continuously cater for this excess.

It is a common practice by the technician in the water supply industry that gate or sluice valves are operated partially closed or completely closed due to forgetfulness after making repairs on the pipeline. If this situation arises pressure build-up behind the gate could be far higher than 94.62m of H_2O . To avoid this situation, additional protective devices are required to assist the pumps.

Pressure of Pockets of Air along the Pipeline

The Ahmadu Bello University Zaria Water Works pumping main traverses steep, rising, and falling ground slopes ranging between 0.1% and 25.0%. This condition promotes the separation of water columns along the pipeline, particularly during pump isolation.

Also, every first-time filling of the pipeline introduces air. The flow may carry the accumulated air to other positions, and an immediate rush of water of equivalent volume compensates for this displacement, causing violent water hammering and cavitation (Jung & Karney, 2020). In addition to the static head and losses along the pipeline, the air pocket generates an extra pressure head for the pump to overcome. The net effects include a decrease in flow rate, energy loss, and an increased risk of water hammering. However, to ensure that the air pockets are collected, transported, and expelled at high points, double air valves were installed at designated points along the rising main to mechanically expunge the air from the system. Despite the malfunctioning of these devices, the

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air problem along the pipeline is adequately taken care of.

The Calculation of the Frictional Head losses

The pipe friction factor for the pipe used was calculated using a K value of 0.1 in the Colebrook formula (as applied by the client and recommended by Pont-A-Mousson)

$$\frac{1}{f} = -4.0 \log_{10} \left(\frac{k}{3.7D} + 2.51 \right) \quad (4)$$

The values of frictional coefficients (k) for various types of fittings were selected from the KSB pipe manual

The pipeline losses were calculated using

$$h_f = \frac{fLV^2}{D2g} \quad (5)$$

and the local losses at fitting locations as

$$h_f = \frac{KV^2}{2g} \quad (6)$$

For the Water Hammer Analysis along the Pipeline,

These valves were analyzed for excessive pressure rise to their location when such valves were closed rapidly (instantaneously) and slowly while the pump was running. The water hammer period is the time taken for a wave generated at a valve location (as a result of its closing or opening movement) to move from the valve to the pump, reservoir, or any other structure, creating the pressure head and returning to the valve. The water hammer period was calculated as

$$T_c = 2\frac{L}{C} \quad (7)$$

Where T is the water hammer period in seconds

L is the length of pipeline upstream of the valve

C is the celerity of wave or speed of propagation of the pressure wave in the pipeline.

The celerity was calculated as

$$C = \sqrt{\frac{1}{\rho \left(\frac{1}{K} + \frac{D}{tE} \right)}} \quad (8)$$

Where ρ = density of water, K = coefficient of compressibility, E = modulus of elasticity

t = pipe thickness, D = pipe diameter

The Allievi Equations

The basic water hammer equations as applied to the element of flow in a pipeline are

Continuity equation:
$$\frac{\delta h}{\delta t} + \frac{v\delta h}{\delta x} + \frac{c^2\delta v}{g\delta x} = 0 \quad (9)$$

Dynamic equation:
$$\frac{\delta h}{\delta x} + \frac{v\delta v}{g\delta x} + \frac{\delta v\lambda V|V|}{g\delta t 2D} = 0 \quad (10)$$

Where,

c = the speed of propagation of the pressure wave, h =hydraulic gradeline location above the point considered, x =distance of the point from the structure reservoir or pump creating the head,

v =velocity of flow, D = pipe diameter, g = acceleration due to gravity

Rieman obtained a solution to these simultaneous differential equations which may be expressed in the form

$$h_{ix} = h_o + F(t - x/c) + f(t + x/c) \quad (11)$$

$$V_{ix} = V_o - g/c(F(t - x)) - f(t - x/c) \quad (12)$$

Where F and f mean `a function and x is positive when measured from the point the wave is generated. Thus the function F(t - x/x) is a pressure (head) wave that propagated upstream at the wave speed c to the pump or reservoir where it will be completely and negatively reflected as a f(t + x/c) at time T/2 = L/c where T is the water hammer period.

In general, the above two equations can be written in the form

$$F_i - F_{i-2} = c/g (V_{i-1} - V_i) \quad (13)$$

$$h_i - h_{i-1} - 2h_o = c/g (V_{i-1} - V_i) \quad (14)$$

Valve in Line Boundary Conditions

The discharge through a valve of area $A_{v,i}$ at time i when the pressure head behind it is h_i is given by

$$Q = C_{d,i} A_{v,i} \sqrt{2gh_i} \quad (15)$$

and the velocity in the pipe immediately upstream is

$$V_i = \frac{C_{d,i} A_{v,i}}{A_p} \sqrt{2gh_i} \quad (16)$$

Where, A_p is the area of the pipe and C_{di} , the discharge coefficient.

The ratio

$$\frac{V_i C_{d,i} A_{v,i} \sqrt{h_i}}{V_o C_{d,o} A_{v,o} \sqrt{h_o}} \quad (17)$$

and

$$\frac{2 h_i}{\varepsilon} = \frac{h_i}{h_o} \quad (18)$$

Substituting into equation 18 yield

$$h_o \varepsilon_i + h_o \varepsilon_{i-1} - 2h_o = \frac{CV_o}{g} (\eta_{i-1} \varepsilon_{i-1} - \eta_i \varepsilon_i) \quad (19)$$

and denoting CV_o/g by the symbol ρ

$$\varepsilon_i + \varepsilon_{i-1} - 2 = 2\rho (\eta_{i-1} \varepsilon_{i-1} - \eta_i \varepsilon_i) \quad (20)$$

Equation 19 represents a series of equations that enables the heads, at a time interval (T) $i = 1, 2, 3 \dots$ at the valve to be calculated from a prescribed closure pattern ($A_{v,i}/V.t$). The equations are known as the Allievi interlocking equations.

For instantaneous valve closure $\eta_1 = 0$ and the equation 20 becomes

$$\varepsilon_i + \varepsilon_{i-1} - 2 = 2\rho (\eta_o \varepsilon_o - 0) \quad (21)$$

Now $\eta_o = 1 : \varepsilon_o = 1$

$$\Delta h = h_1 - h_o = \pm CV_o g \quad (22)$$

A computer program was developed for the solution of the Allievi equation to determine the variation of pressure head at each valve location

within a time interval equal to the water hammer period. Two situations were considered namely rapid or instantaneous closure and slow closure.

The instantaneous or rapid closure gives a maximum pressure rise at each valve locations to be 61.64m. Slow closure analysis was performed for a variable time of closure from 5 seconds to 120 seconds and the iterations were performed for various multiples of the water hammer period. The valve area for different time interval was calculate using equation 23.

$$Area V_1 = Area V_1 * (1 - Elapsetime / T_{close}) \quad (23)$$

The valve is initially assumed fully open. In each of the valve locations at the end of the various time of closure.

RESULTS AND DISCUSSION

Steady-State Analysis of Transmission Line

The steady-state operation of these pumps was carried out using Bentley HAMMER V8i series 4, as shown in figure 3 below.

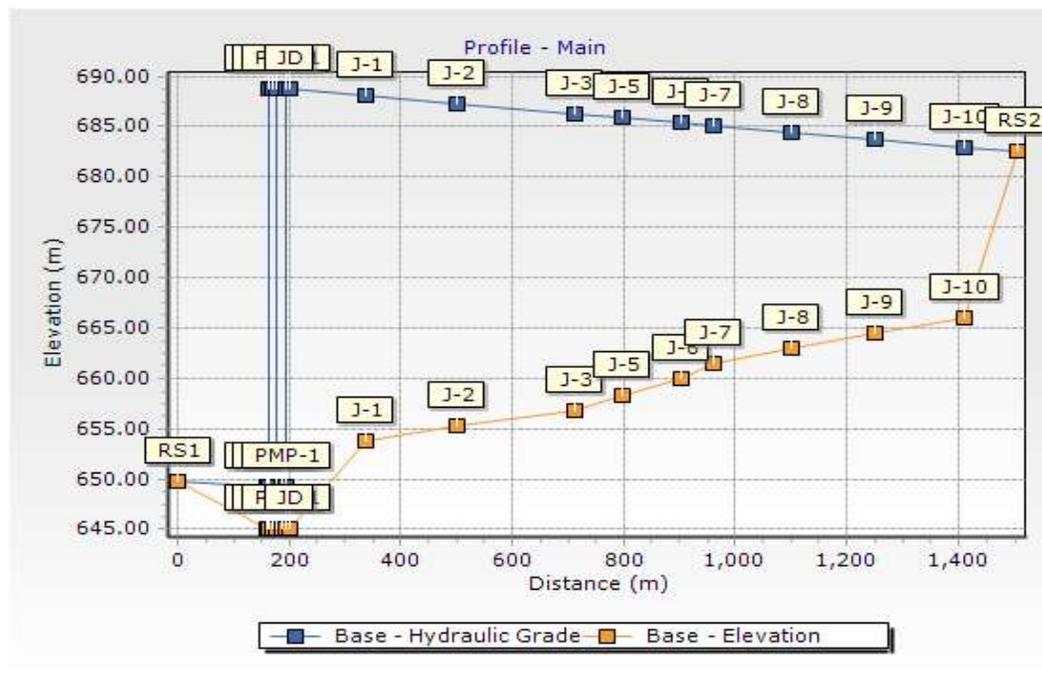


Figure 3: Steady-State hydraulic gradeline of the Transmission Mains

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Analysis of Water Hammer with No Check Valve Is Installed.

These Parmakian solution charts for water hammer at pumping plant was used to investigate the hydraulic transient along with the Ahmadu Bello University, Zaria Water Works pumping main using the pump characteristics and the pipeline constant:

$$2\rho = \frac{CV_o}{gH_o} \quad 4.1$$

$$\rho^* = 2 \frac{KL}{C} \quad 4.2$$

$$K = \frac{91600H_oQ_o}{WR^2\eta N} \quad 4.3$$

Where, WR^2 = moment of inertia, C = wave velocity, V_o = initial velocity, H_o = initial head, L = pipe length, ρ = pipeline characteristics K = constant which incorporate the effect of the pump and motor inertia are required to use the Chart(Parmakian, 1955).

The values of the above parameters are presented in Table 2 for a flywheel effect of inertia.

Table 2: Pipeline characteristics for the Ahmadu Bello University, Zaria Water rising main

| Ho (m) | Vo (m/s) | L (m) | K | C (m/s) | 2ρ | ρ* | Water hammer period (sec) |
|--------|----------|-------|-------|----------|------|------|---------------------------|
| 52.04 | 0.508 | 1506 | 0.107 | 1190.304 | 1.25 | 0.69 | 2.53 |

From Table 2, the analysis indicate that the water hammer period is too short and the implication is that if there is a power cut-off in the pump, the whole column of water in the discharge line will return to the sump near instantaneously. This situation is damaging for the pump and thus must be protected. Using the 2ρ and ρ^* in the Parmakian solution charts presented in appendix B, the following limiting pressure is read for a normal pumping head of 52.04m and Normal pumping speed of 2960rpm(Kinno & Kennedy, 1965):

- Down surge at pump = 70% of Normal pumping head
- Or max drop in head at pump = 36.428m
- Down surge at mid-length = 44% of Normal pumping head
- Max drop in head at pump = 22.898m
- Upsurging at pump = 28% of Normal pumping head
- Max head rise at pump = 14.57m
- Upsurging at mid length = 14% of Normal pumping head
- Max head rise at mid-length = 7.29m
- Maximum reverse speed = 136% of Normal speed = 4025.6rpm

- Time of flow reversal at pump = 3.75L/C after power failure = 7.74seconds
- Time of zero pump speed = 7.5L/C after power failure = 9.49 sec
- Time of maximum reverse pump speed = 14L/C after power failure = 17.71 sec

This water hammer solution yields maximum positive and negative pressure changes, representing the minimum and maximum head envelope curves and the range of head fluctuations for which the discharge line design is necessary. The pump's upsurge, or head rise, exceeds the expected normal pumping head of approximately 70%, or 36.428 m. This translates to an operating pressure of 88.47 m of H₂O, which is within the range of the design pressure of NP 8 to 10 bars, or 81.52 to 101.90 m of H₂O (Kim et al., 2014).

The upsurge at the pump of 28% of the normal pumping head is well above the maximum head rise to shut off of 12.4% recommended by the pump manufacturer. The total head ratio (rate head/mmmaximum head rise) of 60/36.428 (164.71%) exceeds the recommended 72.2% by the pump manufacturer.

The down surge at the pump of 70% of the normal pumping head, or 36.428 m, with friction neglected, translates to a residual pressure of 23.57 m, which is over the rated head of the pump (Caresana, 2011). This residual margin is

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very small when the total head losses due to pipe material lining, fittings, and associated appurtenances are put into considerations (Asli et al., 2010). The implications of this, is that the minimum pressure may approach the vapor pressure of water and when this condition persists for a sufficient period, the vapor pressure of the water may be reached, and the liquid water column may parts and separated by a section of vapor (Arefi et al., 2021). The overall implications of these conditions is that pressure surges and water hammer occurs along the rising main if there is a power cut or failure at pump motors.

Analysis of Water Hammer with Check Valve Installed

When check valves are used along the discharge line, under steady flow conditions, the pump discharge keeps the check valve open. However, when the flow through the pump reverses after a power failure, the check valve closes very rapidly under the action of the reverse flow resulting in a water hammer on the check valve disc. Under this condition, neglecting friction, the check valve can close instantaneously or even the sluice valve can be closed near instantaneously (Carravetta et al., 2017). The head rises due to instantaneous valve closure given by equation 3.4.

$$H = \pm \frac{CV_0}{g} \quad 3.4$$

Where, C = velocity, V_0 = initial velocity, g = acceleration due to gravity

The value of h for instantaneous valve closure for pipe is $H = \pm \frac{0.508 \times 1190.304}{9.81}$

$H = \pm 61.64\text{m}$ which is above the normal operating head of 52.04m

This translates to an operating pressure of 113.68m. This is above the design pressure which is between 81.52 to 101.9m of H₂O. This is a damaging pressure on the pipeline appurtenances having nominal pressure between 8 to 10bars.

The delayed closure of the check valve disc was investigated through a computer program written by the authors based on the Alleivi interlocking equation 19 and the computations for the valve closure are based on

linear function as given in equation 3. The iteration is based on valve closure time of multiples of water hammer period of $\frac{2L}{c}$. Two critical valve locations were considered namely the check disc at the pumping station and the sluice valve at chainage 00+800.10m (Davis, 2004). The analysis showed a pressure of 221.756m at the check valve disc and a negative pressure of -43.998m and -68.115m at the sluice valve and check valve disc respectively.

CONCLUSION

Numerous factors can compromise the hydraulic integrity of a transmission line, potentially impairing the quality and quantity of water. Overpressure and excessive low pressure or negative pressure transients are significant factors that can lead to system failure, including pipe bursts and contamination, among others.

We investigated the occurrence of pressure fluctuations or variations along the ABU transmission line, along with associated surges and water hammer events. We established that either an upsurge or a downsurge along the Main may result from the closure of the valve or the shutdown of the pump.

The Ahmadu Bello University, Zaria Water Supply analysis has revealed the occurrence of pressure surges and water hammers along the Main. The rising Main has revealed the occurrence of pressure surges and water hammers due to power failure at the pump motors, as well as the operation of both the check valves and sluice valves along the Main. If there is a power outage at the pump motors, we expect a maximum pressure rise of 36.428 m above the normal pumping head of 52.04 m. An instantaneous valve closure can also cause a pressure rise of 61.64 m above the normal pumping head of 52.04 m. The excessive reverse speed of 4025.6 rpm occurs at 7.74 seconds from power failure and lasts 17.71 seconds, causing damage to the pump impellers. These conditions are extremely detrimental to the pump assembly, pipeline, and pipeline maintenance. (Jung & Karney, 2020), and the services it rendered, most especially when considering the pump rated head is 60m.

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