TEST Engineering & Management

May – June 2020
ISSN: 0193-4120 Page No. 7399 - 7406

Hydraulic Analysis of the Kabacan Water District Pipe Network

[1]M. P. Esberto, [2]J. N. Orejudos
[1]University of Southern Mindanao, Kabacan, Cotabato, Philippines
[2]Mindanao State University – Iligan Institute of Technology, Iligan City, Philippines
[1]mdpesberto@usm.edu.ph, [2]jerson.orejudos@g.msuiit.edu.ph

Article Info
Volume 83
Page Number: 7399 - 7406
Publication Issue:
May-June 2020

Article History
Article Received: *19 November 2019*
Revised: *27 January 2020*
Accepted: *24 February 2020*
Publication: *18 May 2020*

***Abstract*:**
This study intends to analyze the hydraulic characteristics of the Kabacan Water District (KWD) pipe network in Bgy. Bannawag, Kabacan, Cotabato, Philippines, to assess the vulnerability of the water system in terms of the hydraulic integrity. Establishment of the network topology entails actual site survey to measure the distances between junctions, the ground elevations, and the pipe sizes. The sketch of the network topology is used as backdrop overlain on the EPANET to create the network map [11]. Simulation results indicate that there are excessive head losses beyond the allowable limit which is not sufficient to maintain the minimum operating pressure of 7.0 mwc (10 psi) in most of the nodes. This event signifies depletion of positive line pressures or a cutback in hydraulic capacity that cause unsatisfactory water supply in the network during peak hours. The situation could be worsened by failure of a pump or valve that can cause surges in pressure. The KWD network is vulnerable to pressure surges as evidenced by fluctuations caused by rapid pump start up or shut down. The vulnerability of the KWD network can be described using the EPANET simulation results, thus, mitigating measures can be done to address such and improve the capacity of the water system.

Keywords: *Water Distribution System, Looped Pipe Network, Hydraulic Analysis, Head Loss*

I. INTRODUCTION

Most water service providers in developing countries are confined to poorly designed water distribution infrastructure and poor operation and maintenance practices due to limited budget [9]. In the Philippines, the agency's budgetary constraints for new investment hinders the local water utilities' ability to expand and improve services towards attaining operational and technical efficiencies. In a report disclosed by the Local Water Utilities Administration (LWUA), majority of the water districts that are operating at the end of 2011 have service connection deficiencies [1]. As a matter of fact, some households have no equitable access to potable water. This phenomenon is also true in other parts of the globe as indicated in the study of reference [2]. These services caused a heightened inconvenience particularly to those experiencing intermittent water supply during peak demands.

The Kabacan Water District is confronted with challenges in sustaining its water service delivery at Bgy. Bannawag, Kabacan, Cotabato, as the community reported a low pressure in taps some time of the day [8]. These maybe caused by leaks or excessive head losses that compel the system to generate larger pressures within the network to compensate the loss. It is in this context that this

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study is initiated to assess the hydraulic integrity of the KWD pipe network in Zone B Water System, in order to properly address deficiencies.

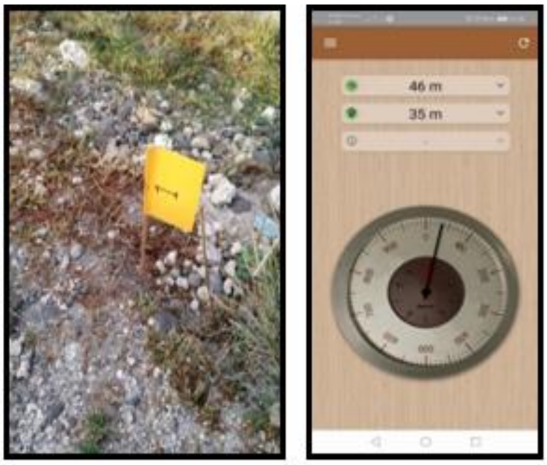
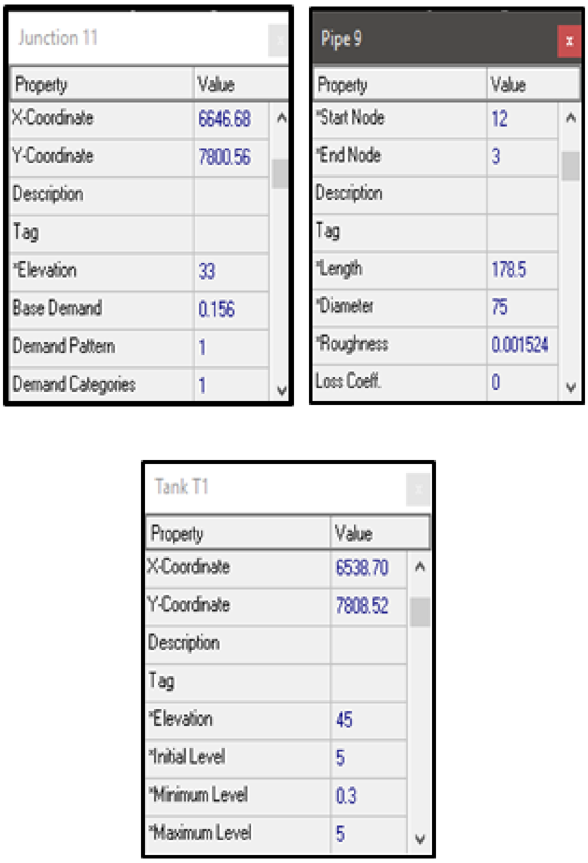


Fig. 1. Stake(left) and elevation reading (right) at *junction no. 1*

II. METHODS

A route survey is done using the Global Positioning System for the distance measurement, the Altimeter (Fig. 1) to measure the local position of ground elevations, and the Geo Tracker to track the network path. The data on the hourly mean flow and household consumption provided by the KWD are used to calculate the average daily demand (ADD), base demand, as well as the time pattern of the pump, water source and the network nodes. The inputs on the pump curve is based on the KWD data while the information obtained from the field survey are entered as input properties of the network nodes, links and tanks.

As an example, the property inputs for the network node is shown in Fig. 2a. The information supplied for the node includes the elevation (m asl) based on the route survey, the calculated base demand in liters per second (lps), and the time pattern for that node. This study assumed only two types of time pattern; 1) domestic demand and 2) pump and source. The link property inputs consist of the pipe length, diameter and roughness coefficient (Fig. 2b).



Junction 11

Property	Value
X-Coordinate	6646.68
Y-Coordinate	7800.56
Description	
Tag	
*Elevation	33
Base Demand	0.156
Demand Pattern	1
Demand Categories	1

Pipe 9

Property	Value
*Start Node	12
*End Node	3
Description	
Tag	
*Length	178.5
*Diameter	75
*Roughness	0.001524
Loss Coeff.	0

Tank T1

Property	Value
X-Coordinate	6538.70
Y-Coordinate	7808.52
Description	
Tag	
*Elevation	45
*Initial Level	5
*Minimum Level	0.3
*Maximum Level	5

Fig. 2. Input properties of junction J11(a), pipe 9 (b) and Tank T1 (c)

The required properties for the tank (T1) shown in Fig. 2c are the tank's bottom elevation (in meters); the tank's initial level equivalent to the height of water (in meters) at the start of the simulation; and the minimum level which is the minimum height (in meters) of the water level that must be maintained. The tank is constrained at this level.

The base demand for junctions near the tank is assigned a zero value, in this case, the junctions connecting pipes leading to the main distribution line. On the other hand, the principal input properties for a pump are its time pattern, head, flowrate and pump curve (the combination of head and flow produced by the pump), labelled as Curve ID "1" , as shown in Fig. 3.

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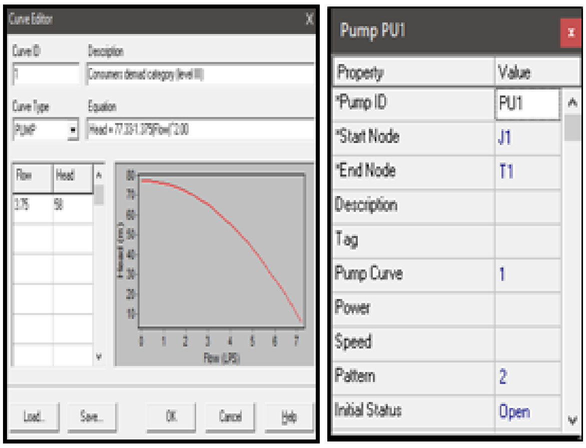

Fig.3. Pump curve and property

The input parameters for the pump which are flow and head gain and the curve's equation are shown along with its shape (Fig. 3, left) and the entry for the pump curve is described by a time pattern of 2 (Fig. 3, right). The hydraulic head is calculated as 58 m consist of the maximum height of the water level in the tank (5.0 m); the tank bottom height (13.0 m) and the depth of the suction pipe (40.0 m). The system is designed for level III connections having an average daily flowrate of 3.75 lps (Table 1) although the pump's actual capacity is 6 lps.

Table 1: Calculated 24-hour flow pattern for the pump and water source

Table 1 continued

Time	Weekly Mean Flow			Pump Flowrate Pattern		
	Mini-mum (lps)	Ave-rage (lps)	Maxi-mum (lps)	Abso-lute Diffe-rence	Square of differ-rence	Pump Pattern (%)
12:00 AM	0	0.14	1	-2.75	7.58	0.27
1:00 AM	0	0.03	0.24	-3.51	12.35	0.06
2:00 AM	0	0.86	3.44	-0.31	0.1	0.92
3:00 AM	0	0.32	1.76	-1.99	3.98	0.47
4:00 AM	0	1.52	3.22	-0.53	0.28	0.86
5:00 AM	2.11	3.83	4.97	1.22	1.48	1.32
6:00 AM	4.54	4.57	4.64	0.89	0.79	1.24
7:00 AM	4.49	4.51	4.53	0.78	0.6	1.21
8:00 AM	4.47	4.49	4.5	0.75	0.56	1.2
9:00 AM	3.43	4.32	4.49	0.74	0.54	1.2
10:00 AM	0	2.93	4.47	0.72	0.51	1.19
11:00 AM	0	2.09	4.49	0.74	0.54	1.2
12:00 AM	0	1.96	4.47	0.72	0.51	1.19
1:00 PM	0	1.97	4.47	0.72	0.51	1.19
2:00 PM	0	2.28	4.44	0.69	0.47	1.18
3:00 PM	0	3.02	4.52	0.77	0.59	1.2
4:00 PM	1.48	3.37	4.54	0.79	0.62	1.21
5:00 PM	0	3.1	4.52	0.77	0.59	1.2
6:00 PM	0	2.89	4.52	0.77	0.59	1.2
7:00 PM	0	2.22	4.36	0.61	0.37	1.16
8:00 PM	0	0.91	3.79	0.04	0	1.01

<table>
<tr><td rowspan="2">Time</td><td colspan="3">Weekly Mean Flow</td><td colspan="3">Pump Flowrate Pattern</td></tr>
<tr><td>Mini-mum (lps)</td><td>Ave-rage (lps)</td><td>Maxi-mum (lps)</td><td>Abso-lute Diffe-rence</td><td>Square of differ-rence</td><td>Pump Pattern (%)</td></tr>
<tr><td>9:00 PM</td><td>0.00</td><td>0.21</td><td>1.46</td><td>-2.29</td><td>5.26</td><td>0.39</td></tr>
<tr><td>10:00 PM</td><td>0.00</td><td>0.51</td><td>3.57</td><td>-0.18</td><td>0.03</td><td>0.95</td></tr>
<tr><td>11:00 PM</td><td>0.00</td><td>0.55</td><td>3.68</td><td>-0.07</td><td>0.01</td><td>0.98</td></tr>
<tr><td colspan="3">Average</td><td>3.75</td><td colspan="3"></td></tr>
</table>

2.1. Construction of the Network Model in EPANET

In setting up the project, the hydraulic options are set first considering a relative viscosity at 27° C of 1.0 centistokes (D-W), and the Darcy Weisbach (D-W) is selected as the head loss formula (Fig. 4). For solving the nonlinear equations, the maximum number of trials is set to 40, while the convergence criterion is fixed to 0.001. The KWD water distribution network model constructed in EPANET is shown in Figure 5 consists of a source junction (J1) connected to a pump (PU1) leading to a storage tank (T1) supplying water to the looped pipe network.

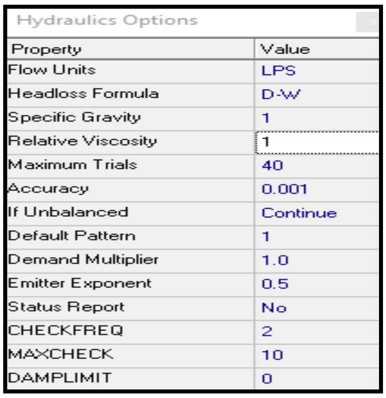

Fig. 4. Network hydraulic options

The connection details of pump, tank and source junction is clearly sketched where the pipe sizes are given in millimeters (red font) and the node index in black font (Fig. 5). When all the required input information is completed, the hydraulic analysis is run. The simulation is not successful without the faucet icon with a flowing water displayed on the EPANET task bar.

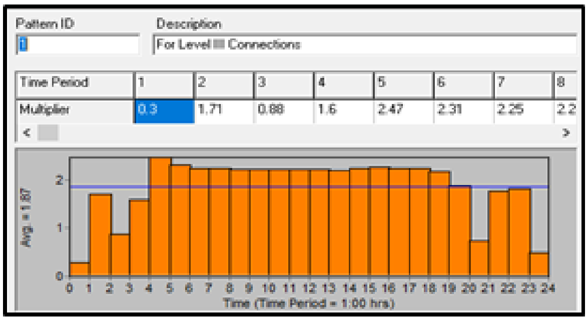

Fig. 6. Demand time pattern for nodes

2.2. Time Pattern

The 24-hour time pattern calculated based on the actual domestic consumption describes the time variation in this demand category at each junction (Fig. 6). The values are applied as multipliers to the base demand characterizing moderate to peak water demands, while Fig. 7 illustrates the time pattern for the pump (labelled as PU1) and the source (node J1). The calculated multipliers are based on actual pump flowrate. In the nodes and links data entry, the same time pattern is used for the pump and the water source which is assigned a Patten ID of "2" (referring to Fig. 3), while for the rest of the nodes, a Pattern ID of "1" is assigned. Incorrect data entered to the EPANET render invalid property entries, thus, the simulation run would not be successful.

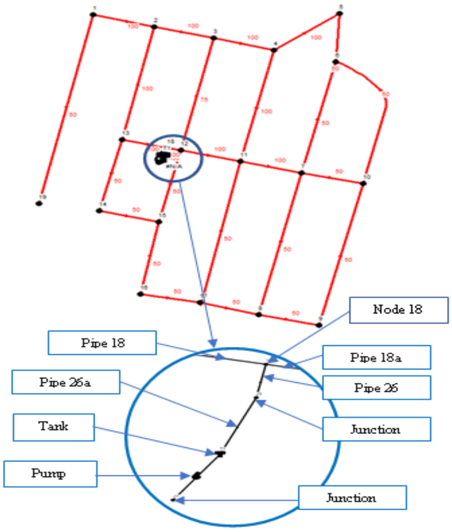

Fig. 5. The existing KWD pipe network modelled in EPANET

The node and link properties of the network obtained from the route survey is shown in Table 2. For the Darcy-Weisbachheadloss formula, the roughness coefficient (ε) for the HDPE SDR 32.5 pipe material is 1.524 x 10^{-3} mm, based on an open source software. The calculation of the base demand is based on the actual household consumption reported by the KWD.

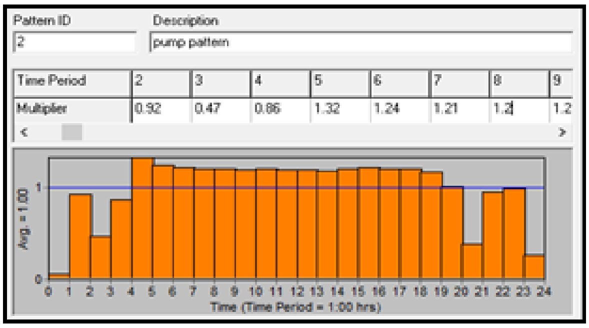

Table 2. KWD Zone B network property profile

NODAL PROPERTIES			LINK PROPERTIES			
Node ID	Ele-vation (m)	Base demand (lps)	Link ID	Dia-meter (mm)	Length (m)	D-W Roughness coef (mm)
1	35	0.2100	1	100	60.0	1.524E-03
2	36	0.0780	2	100	66.0	1.524E-03
3	36	0.0960	3	100	58.0	1.524E-03
4	37	0.3480	4	100	79.0	1.524E-03
5	38	0.3720	5	50	67.0	1.524E-03
6	37	0.0480	6	50	279.0	1.524E-03
7	34	0.1500	7	100	173.0	1.524E-03
8	30	0.0300	8	50	94.0	1.524E-03
9	30	0.0180	9	75	178.5	1.524E-03
10	34	0.0540	10	50	109.0	1.524E-03
11	33	0.1560	11	50	109.0	1.524E-03
12	32	0.0780	12	50	182.0	1.524E-03
13	31	0.0540	13	50	212.0	1.524E-03
14	31	0.0600	14	50	171.5	1.524E-03
15	30	0.0720	15	50	210.0	1.524E-03
16	29	0.0480	16	50	185.0	1.524E-03
17	29	0.0360	17	50	208.0	1.524E-03
18	32	0.0000	18	100	63.0	1.524E-03
19	31	0.1020	18a	100.0	2.0	1.524E-03
T1	45.3		19	100	70.0	1.524E-03
J2	32	0.0000	20	100	64.0	1.524E-03
			21	50	68.0	1.524E-03
			22	50	66.0	1.524E-03
			23	50	63.0	1.524E-03
			24	50	71.0	1.524E-03
			25	50	60.0	1.524E-03
			26	75	8.0	1.524E-03
			26a	75	13.5	1.524E-03

III. RESULTS AND DISCUSSION

Computer based mathematical models such as EPANET 2.0, provide a practical means of analyzing the hydraulic conditions of a water distribution system and can provide hydraulic calculations including nodal outflow, pressure, and other hydraulic parameters in the pressurized pipe network [4, 12]. The use of a hydraulic simulator provides a wide range of predictive capabilities necessary to determine whether the hydraulic integrity is compromised or not.

The researcher used the actual base demands over a series of points in time with a hydraulic time step set at 1 hour for the extended period simulation (EPS), which is also set at 24 hours. The hydraulic analysis provides reports on the calculated junction heads, actual demands, pressures and link flows, velocities and unit head losses that can be accessed and generated both in graphs and tabular forms. This information provides a heads up on critical scenarios and be able to manage the risk.

3.1. System Flow

Referring to the EPANET simulation result, the graph in Fig. 8 describes a balanced water supply (input) over water demand (output) which suggests a satisfactory replenishment from 0 to 23 hours. However, the demand is not met at the 24th hour. The maximum flowrate (5 lps) in Fig. 8 is used as the basis for calculating the pump's time pattern under a controlled condition where peak water consumption is deduced. This is to ensure the reliability and stability of the system given the supervised data measurements that characterize the tendencies of domestic consumption.

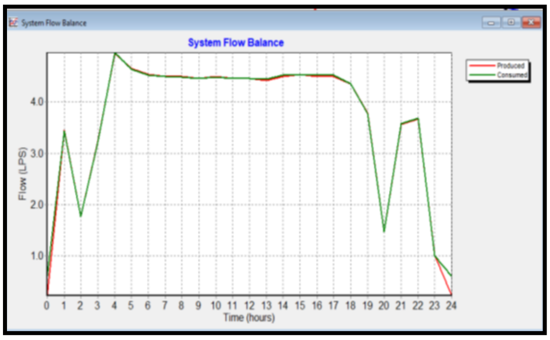

Fig. 8. System flow generated in

Comparing the graph of the EPANET result (Fig. 8) with the actual KWD data (Fig. 9), it can be observed that the maximum weekly mean flowrate in Fig 9 compensates the system peak flow rate in Fig. 8. The data series for both figures are consistent except for the minimum flow rate which is reflected in 2h and 20 h in EPANET (Fig. 8), while it landed at 3h and 21h in the existing data (Fig. 9). Nevertheless, the minimal inconsistencies maybe attributed to the calculated multipliers in the demand pattern assigned to the nodes, thus, the discrepancies.

The observed data in Fig. 10 followed similar configurations with the two discussed figures (Figures 8&9). This is the actual daily mean flowrate of the KWD pump station. Examining the figures (Figures 8& 10), it can be noted that the system flowrate in EPANET (Fig. 8) satisfies the actual flowrate illustrated in Fig. 10. Overall, the EPANET simulation result is representative of the actual, hence,

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reliable to describe the hydraulic conditions of the existing KWD pipe network.

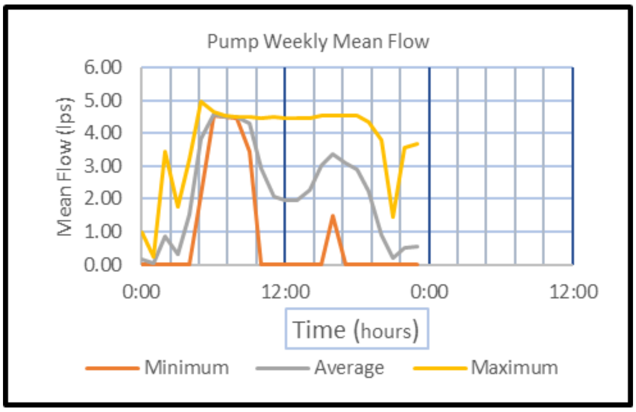

Fig. 9. Maximum, average and minimum weekly mean flowrate

3.1. Pressure Profile

Based on the local engineering standards and conditions, the KWD specification for the operational pressure ranges between 20 psi and 40 psi to be maintained throughout the network under all flow conditions.Based on Fig. 10, the network experiences pressure fluctuations caused by sudden pump shutdown during the day time. This causes pressure transients in the distribution system which is most severe at nodes near the pump station and in elevated areas. The nodes with low static pressuresand specifically in the remotestreachalso suffer negative pressures [6, 7].

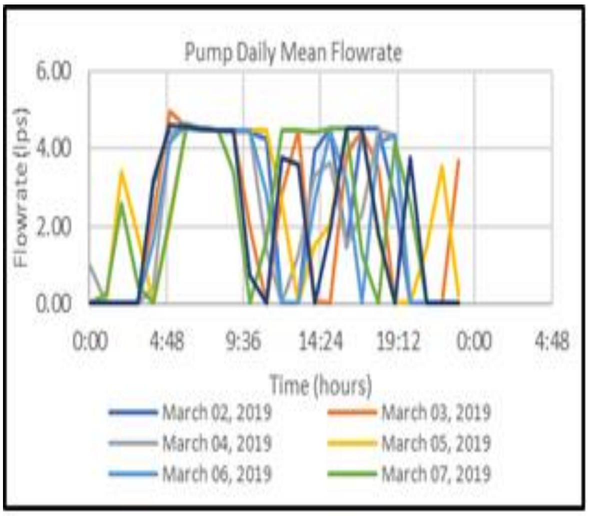

Fig. 10. KWD Pump daily mean flowrate

Fig. 11 illustrates that the operating differential pressure (static) present in the system is ranging from 11 meters of water column (15 psi) to 21 meters of water column (30 psi).

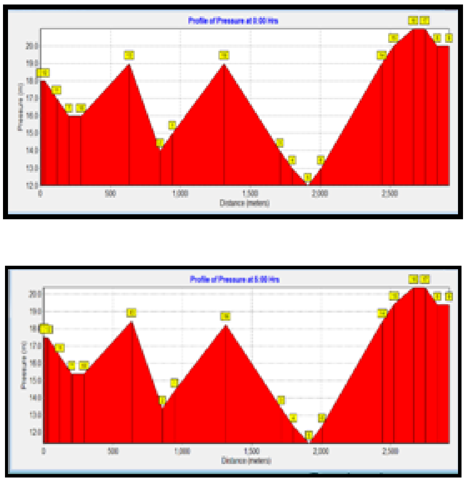

Fig. 11. Pressure profile of the network at (a) 0:00 hrs and (b) 5:00 hrs

It can be construed that the end-users are served with adequate water supply under an ideal hydrostatic pressure, where those household connections tapping at the highest elevation at node 5 (with ground elevation of 38.0 masl) are still served with a reliable quantity under acceptable hydraulic provisions. But still, there are other hydraulic parameters to consider satisfying the stability of the system.

The lowest ground elevations (e.g., nodes 14-17) in the network have the highest pressuresalthough the reported pressures are below the maximum level as shown in Fig. 11. The minimum pressure of 11 mwc (Fig. 11a) and 12 mwc (Fig. 11b) for node 5 establishes this junction, which is the highest elevation, that can be supplied by the system, provided there are no excessive head losses and pressure surges that occur. This junction is one among the other spots with critical hydraulic condition that needs monitoring as the differential pressure approaches the minimum boundary. It is also noted that the area frequently experiences power interruption [8] that causes sudden pump arrests. This case creates pressure surges of excessive magnitude, not to mention the reversal of flows due to swift closing of valves as well as periodic flushing. These transient pressures can be somewhat greater than the operating pressure.They can even be greater than the magnitude of static pressure in the water mains, thus, increasing the chances for possible leakages and pipe rupture. This anomaly will decrease the system reliability or even cause equipment failure in extreme cases [12]. Besides, the network is at risk for another hydraulic issue, when the velocity is increased due to water bumps, such as filling or draining of the reservoir[10]. Further, the situation calls for an in-depth analysis of the system to evaluate its hydraulic integrity in response to the accounts for low pressure in taps as reported by most residents in

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the barangay, usually encountered during the pump shutdown due to power outage.

3.2. Head Losses

The EPANET simulation result identified areas of low and excessive head losses. When the water surface in the storage tank is at its maximum level (50.0 m), during peak hours, most pipes are subjected to excessive head losses. Limited supply connections is evident as shown in Table 3. The hydraulic capacity is critical at 5:00 AM at the maximum water level where only about 12.84% of the total household end-users are supplied with water.

Table 3: Network links and nodes with water supply at 5:00 AM

Link ID No.	Supply Node ID No.	No. of
2	2	13
5	6	8
17	9	3
23	16	8
24	17	6
25	8	5
Total No. HHs supplied		43
Total No. of HHs in the system		335
Percentage of HHs with adequate supply		12.84%

There are six (6) links in the main distribution line that can still sustain the water delivery to the service pipes as a result of lesser head losses which are below the boundary limit, though only one link (pipe 5) is supplying water to node 5, all the rest of the households (62 HHs) connected to this node experience difficulty in water supply at this particular time of the day (5:00 am). The supply source nodes in the network are also indicated in the table. Large head losses cause a substantial declinein the network'shydraulic capacity leading to deficient line pressures as discussed in the paper of reference [5]. This loss in the system pressure compromises the hydraulic integrity of the water system.

In the day time specifically at 5:00 hours, the system pressure is below 25 m (Fig. 12b), which is consistent with the actual daily mean pressure in Fig. 13. The peak hours begin at 5:00 hours until 9:00 hours as reflected in Fig. 13. This result demonstrates similar trends in Fig. 10, where the water flowrate is highest during this period (5-9 hrs). Indeed, the system pressure showed relationship with the nodal outflow. The pressure is lower when the water demand is higher, otherwise, it is excessive during off peak hours.

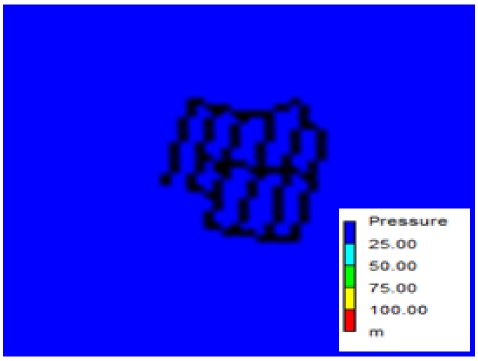

Fig. 12b. KWD network contour plot pressure at 0:00 hours

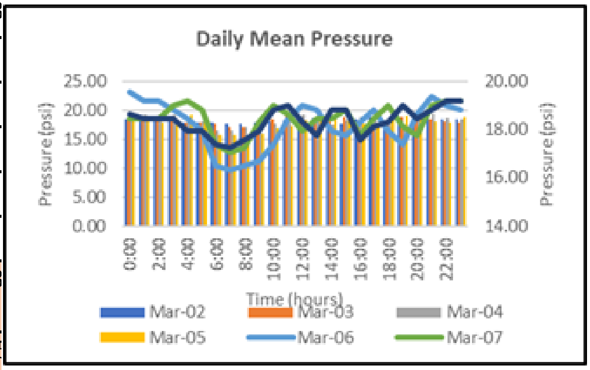

Fig.13: KWD daily pressure log monitor inPump A

IV. CONCLUSIONS

The most critical factorof losing the hydraulic integrity is the failure to maintain the desired operating pressure supplied to customers with varying demand conditions. In the specific case of the KWD Zone B pipe network, earlier findings indicate excessive head losses beyond the allowable limit [5] which is not sufficient to maintain the minimum operating pressure of 7.0 mwc (10 psi) in most of the nodes. This situation signifies decline in positive line pressures or a reduction in hydraulic capacity leading to unsatisfactory water supply. The KWD network is also vulnerable to pressure surges as evidenced by pressure fluctuations (Fig. 13). The vulnerability of the KWD network is further aggravated when the reservoir levels drop suddenly resulting to a large main break with potential reversal of flows [3]. Such a hydraulic drawback in the network becomes more resolute as conditions contribute to system inefficiency and equipment damages. Nonetheless, the occurrence of pressure surges may be minimized by knowing the causes of its origin [6].

The vulnerability of the KWD pipe network is indeed evident and needs immediate action to save the system from impending failure.

V. ACKNOWLEDGMENTS

My appreciation to the KWDpersonnel for the logistics and technical supportprovided for the success of this endeavor. Myheartfelt thanks to my husbandfor his benevolence and above all, a million thanks to God Almighty for His provisions.

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MAIN AUTHOR PROFILE

Name: Ma. Dely P. Esberto
Employment: Faculty, University of Southern Mindanao,
Kabacan, Cotabato, Philippines
Graduate Studies: Mindanao State University- Iligan Institute of Technology, Iligan City, Philippines