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Feasibility of hydropower generation on existing Legedadi water supply scheme, Addis Ababa-Ethiopia

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ABSTRACT

Hydropower can be harnessed by installing in-pipe turbines with the reduced cost compared with hydropower dam construction. Legedadi Water Supply Scheme is found in Addis Ababa and is fed by gravity. This research assesses the hydropower potential of the existing large water transmission pipelines in line with their financial viability. The research required the collection of data from pipe flow (for 29 years from the system record) and pipe layout drawings believed to be useful for estimating the power. The available pipe layout drawing was processed to prepare a profile view with the help of AutoCAD CIVIL 3D. From the profile view, the gross heads and length of the pipelines were obtained. The exploitable power and financial viability of the projects were estimated by RET Screen software. The raw water main (DN1200) was discovered to have a head of 12.46 m and to convey up to 1.47 m^3/s at 90% exceedance over a length of 550 m. The two treated water mains have a head of 19.15 m with a flow of 1.14 m^3/s at 90 % exceedance via DN1200 and 0.29 m^3/s at 90% exceedance via DN900 over a length of 18.4 km. The most suitable sites for the installation of turbines were at the inlet of the treatment plant and the Kotebe Terminal Reservoir. The Toshiba Hydro-eKIDS turbine was selected since it might work efficiently with large flow variation.The annual energy output from the raw water main to be obtained was 1,208 Mwh, with an estimated cost of $461,000 and an annual savings or revenue of $75,946. For the treated water mains, 1,193 Mwh (DN1200) and 344 Mwh (DN900) could be extracted with an estimated cost of 414,500$ (DN1200) and 135,900$ (DN900). The annual revenue for treated water mains is 75,068$ (DN1200) and that for DN900 is 18,842$.

Keywords: Addis Ababa; Hydropower Potential; Legedadi; AutoCAD CIVIL 3D; RETScreen

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1. INTRODUCTION

Energy and water appear to be complicated and strongly interrelated (Muhammad et al., 2016; Muhsen et al., 2019). All sources of energy require water in their fabrication processes; at the same time, energy helps to become water resources available for human use. A water supply scheme comprises civil infrastructures (i.e. reservoirs, pipes), hydro-mechanical and electrical equipment, and services that extract, convey, and distribute water to users (Samora et al., 2016). The clear water delivery from the surface or ground its transport and distributions all need energy which incurs significant operational costs for water providers. The water sector consumes approximately 120 million tons of energy globally each year (Capuano, WEO2018). More than half (850 TWh—around 4% of global energy) is in the form of electricity. There is a lack of a report on energy consumption of water sectors specifically in Africa or Ethiopia. However, the investigation by the author in the study area shows there is an increment in energy consumption even from month to month. Additionally, frequent power cuts are common problems in the study area.

There is an indication in the literature that the energy use in water sectors may constantly grow especially in urban areas because of population growth. Although the energy cost for operating these facilities might be quite high, water companies can significantly benefit from harnessing energy in the system so as to compensate for part or all of that cost. Studies reveal that there exists hydropower potential in the gravity-fed water supply pipelines within closed conduits (Kumar and Shahid, 2017).

Addis Ababa gets its drinking water from surface treatment plants (Gefersa and Legedadi) and sub-surface (well fields) found in different areas of the city. The Legedadi Water Supply Subsystem, whose flow system is via gravity, is one of the principal sources of drinking water in Addis Ababa City Administration. According to the water balance study by the production case team of Addis Ababa Water and Sewerage Authority (AAWSA) in the year 2018, the scheme had an annual production of 60.27 Mm^3. This flow system could provide a hydropower potential benefit using the in-pipe installation.

2. DESCRIPTION OF THE STUDY AREA

The Legedadi Water Supply Scheme is 24 km away from Addis Ababa located in the eastern direction. It is one of the surface sources of drinking water for the city of Addis Ababa and its vicinities. The raw water reservoir catchment area and the treatment plant are situated in Oromia Regional State under the administration of North Shoa Zone in Aleltu Bereh district, Sendafa Town Administration. The scheme consists of two dams: Legedadi (45.9 MCM) and Dire (21.5 MCM) with a modern treatment plant having a capacity of 192,000 m^3/d near the Legedadi dam. The raw water from two retention dams is mixed at a junction and fed into the Legedadi Treatment Works through a single pipe of DN1200 over a distance of 550m. Then, the treated water is carried through two main steel pipes. The first pipe has a diameter of DN900 and discharges water for the whole length of 18.4 km to Kotebe Terminal Reservoirs. The second main pipe has a diameter of DN1400 for the first 6.76 km, and it is reduced to DN1200 because of major off take near the Ayat area. Eventually, it discharges treated water over a length of 11.27 km to the Kotebe Terminal Reservoir. To control high pressure created by gravity flow system, four pressure reducing stations are provided for both pipes on the way to Kotebe Terminal reservoir. Figure 1 shows the general layout of the Legedadi water supply system.

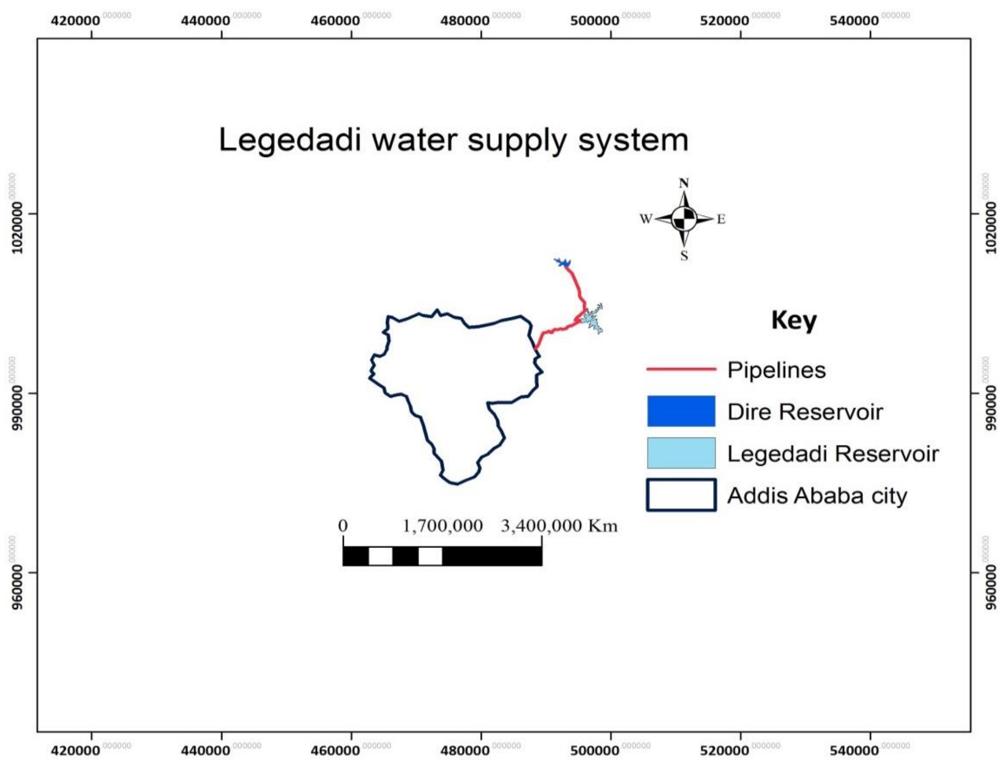

Figure 1 Legedadi Water Supply System

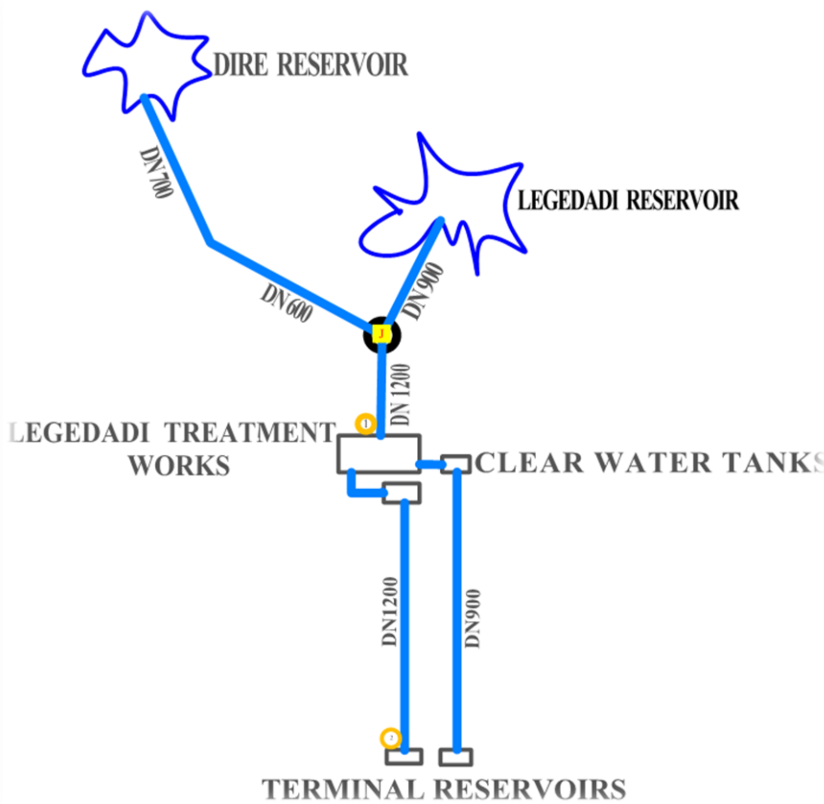

Figure 2. General layout of Legedadi Water Supply System

3. DATA SETS USED IN THE STUDY

In this study, the hydropower potential of the existing water transmission line of the Legedadi Water Supply Scheme was investigated. The necessary data groups to accomplish and achieve the objectives of the study were: pipe flow rate of raw and treated water; construction drawing of pipe layout; locations of intake, treatment plant, and service reservoirs. All the data were secondary and collected from the Addis Ababa Water and Sewerage Authority main office and the project offices. The adequate length of pipe flow (1990–2018) data from system records at the inlet of the treatment plant and service reservoir was obtained. The drawing of the pipe layout in the form of AutoCAD was obtained to process and determine the head.

3.1. Head

Bruno et al. (2010) defined multipurpose systems as those in which electricity generation was not their primary priority. This might suggest the integration of the power plant into the existing infrastructure while ensuring its primary (water supply) and secondary (hydroelectric power generation) functions. An assessment of the site was believed to be a precondition for any hydropower development (Kusre et al., 2010). The head, flow rate, and overall system efficiency were the main factors taken into account during the assessment. These parameters could be worked out through measurement and manufacturer specifications. Head and discharge could be increased or decreased for the same power output. However, the head could not be varied as it was site-dependent. For the initial location evaluation, different scale topographic maps of the study area and additional field investigation would be sufficient to identify the head for a conventional hydropower project.

To select suitable site and head, the drawing of the pipe layout of the water supply scheme was obtained. The analysis initially involved acquiring pipeline designs to prepare the pipe profile view. This was done with the help of AutoCAD Civil 3D. From the design profile, the elevation of the pipeline at regular intervals, length (chainage or stations), and the corresponding gross head along the transmission mains were obtained. If the flow was transported through a long, pressurized conduit, head loss could reduce the power produced and needed to be calculated. The total head loss could be the sum of major and minor head losses. For this study, the frictional head losses in the pipe material were calculated. To estimate the frictional head losses, the material type, length, diameter, flow rate, and average velocity through the pipe were the necessary data required.

This information was obtained based on the site visit during the data collection stage and the pipeline layout data collected from the office. The friction factor was calculated by applying the Excel software and using the Colebrook-White Equation which required relative roughness of the pipe material and Reynolds number. The relative roughness of the pipe material was the ratio of the absolute roughness coefficient to the diameter of the given pipe. Absolute surface roughness coefficient values for the existing pipe material were taken from the Engineering Toolbox. To calculate Reynolds' Number, average velocity in the pipe was calculated by using flow rate (Q_{50}) from FDC and the kinematic viscosity of water was assumed at 20 0c ($1*10^{-6}$). After determining the pipe friction factor, the Darcy-Weisbach Equation (equation 3.3) was applied to determine the total head loss in the pipes.

$$h_l = \frac{f*L*V^2}{2g*D} \tag{2.1}$$

$$\text{while, } \frac{1}{\sqrt{f}} = -2\log\left[\frac{\varepsilon}{3.7D} + \frac{2.51}{Re\sqrt{f}}\right]$$

3.2. Discharge

The next question was how much of the flow could be used for hydropower generation, and how to determine the turbine flow from the previously existing flow of water. Given that the intention was to estimate the maximum potential of the water supply scheme, in this study it was assumed that all of the flow could be passed through the turbine. Hydrological data had to be specified as an FDC in RET Screen, which represented the flow conditions in the site being studied over time. Flow rate values for the raw water transmission pipeline measured at the inlet of the treatment plant and the treated water transmission pipelines at inlet of Terminal for twenty-nine years were obtained from the production department of the authority. The Flow duration curve was plotted for the transmission mains of raw and treated water using flow rate data taken from the production department of the AAWSA. The flow rate variations between supply point and delivery point were determined using the flow rate values upstream and downstream. Microsoft Excel 2016 was used to plot a cluster graph used to determine the transmission mains with the least flow variations. The pipeline with the least flow variation was expected to deliver consistent flow thus giving slight power variations.

Flow rate values falling at 50 and 90 percent exceedance probability on the duration curve were used to determine the power output. The total period method yielded more correct results than the calendar year method which averaged out extreme values. Therefore, for this study total ordered (year) method was adopted since it would give more accurate results.

$$P = \left(\frac{\mathrm{m}}{\mathrm{N}+1}\right) * 100 \tag{2.2}$$

3.3. Financial Viability

Before deciding to invest in a hydropower plant, it is necessary to conduct a financial analysis of the project. The economic analysis is a cost-benefit comparison that allows the investor(s) to make an informed decision about the project. The small hydro cost can be split into three segments: machinery, civil work, and external costs.

As a result, the payback method was used to validate the viability of the project. The payback method might determine the number of years required for the invested capital to be offset by resulting benefits. RET Screen software was used to decide the feasibility of developing the hydropower project on existing pipelines of the Legedadi Water Supply Scheme. The RET Screen software had a cost analysis worksheet that would enable the user to estimate the cost and credits associated with the project. RET Screen provided a tool called the "Hydro formula costing method" to help estimate the project costs. The formula method used Canadian projects as a baseline and then allowed the user to adjust the results for local conditions. The cost of projects outside Canada compared to the cost of projects in Canada might depend to a great extent on the relative cost of equipment, fuel, labor, equipment manufacturing, and the currency of the country. In general, the cost analysis data particularly cost ratios for equipment, fuel, labor, equipment manufacturing, and, exchange with respect to Ethiopia and Canada were provided for the year 2020.

4. RESULTS AND DISCUSSION

4.1. Site Selection and Head Determination (Raw and Treated)

A gross head of 12.46 m was found in the raw water transmission line from the intake to the Treatment Plant (TP). Typically, a hydropower plant with a head of less than 30 meters is regarded as a low head, though there is no definite line of separation for low, medium, and high heads. The profile view of the raw water line from the intake to the treatment plant's inlet is illustrated in Figure 2. The construction material of the pipe was ductile iron (DCI), and its 550-meter length had a head loss of 0.018 m.

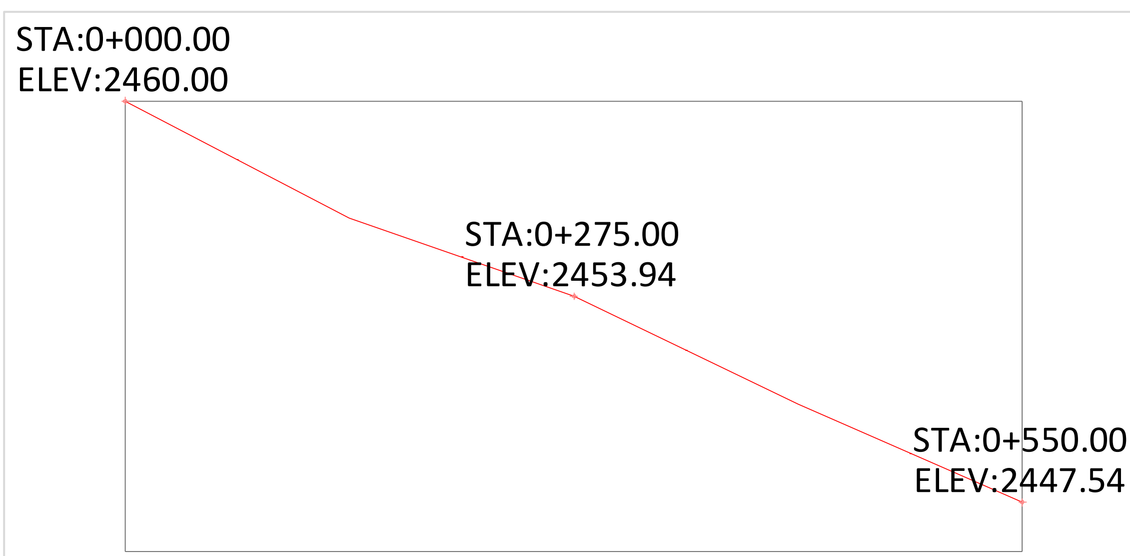

Figure 3. Longitudinal profile of raw water pipe from intake to TP

The 18.4-kilometer pipeline profile shown in Figure 3 transports treated water from the Legedadi clear water tank to the Kotebe Terminal Reservoir in the city, close to the Lamberet Bus Station. The gross head from the off-take point at the Legedadi clear water tank to the Terminal reservoirs was 19.15 m. The Terminal and Legedadi Reservoirs were both built at ground level. The pipeline profile was used to determine the head of in-pipe hydropower installation for the treated water transmission line. The treated water main had eight potential locations as shown in Figure 4 shows the details of the sites described on Table 1). Steel pipe with a tough exterior was recognized as the pipe material type.

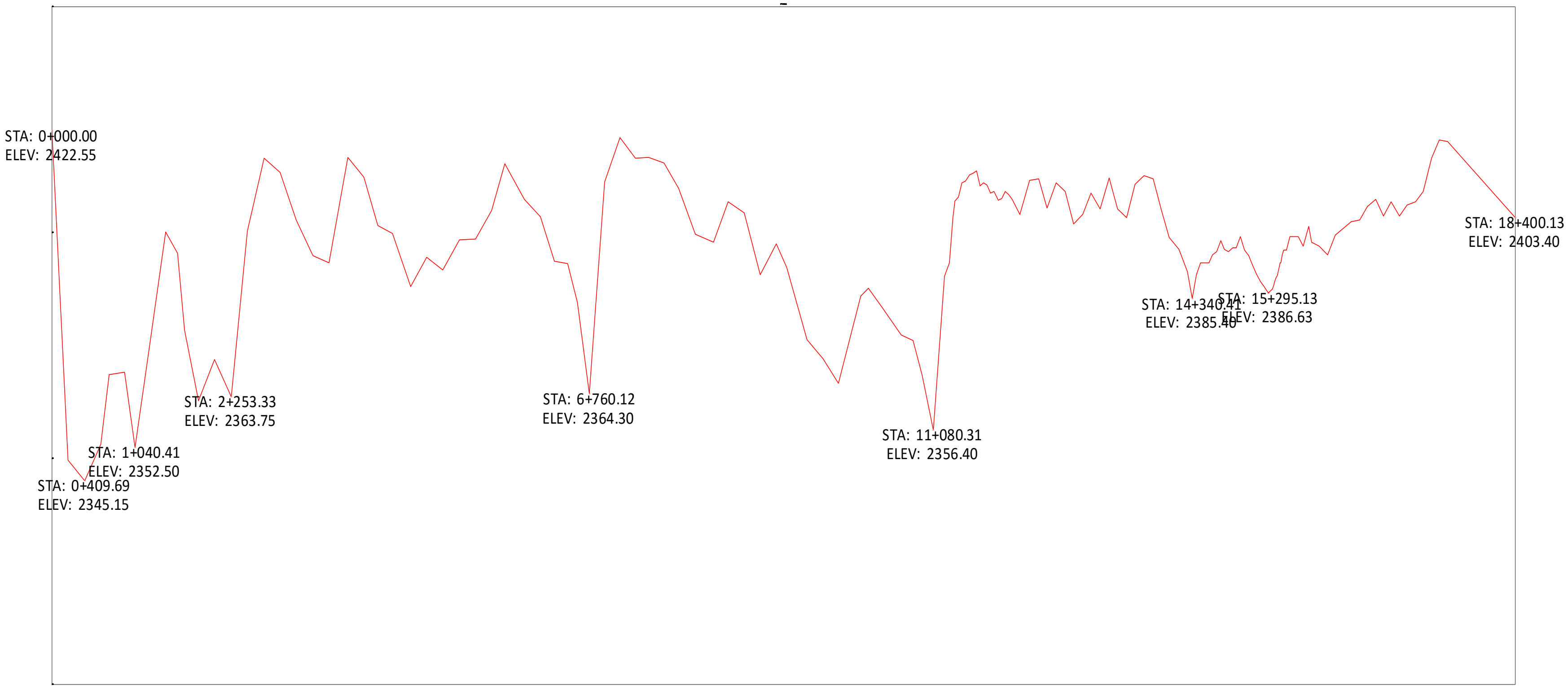

Figure 4. Longitudinal profiles of the LWTP-Terminal Reservoir pipeline

4.2. Discharge Determination (Raw and Treated)

A single value of the flow has no significance in designing a hydroelectric power plant because the flow rate fluctuates considerably in a year, even in a single hour, especially in water supply pipelines. If these fluctuations are not considered in the design stage, the plant may only work efficiently for a short period, resulting in a wasteful investment. The raw water is conveyed from Dire Retention Dam through DN700 and then DN600 and from the Legedadi retention Dam via DN900 pipe. Then after, the raw water from two retention dams are combined at a junction and fed into the treatment plant via a single pipe of DN1200. The flow rates are measured at the inlet of the treatment plant with the help of an electromagnetic flow meter connected to a pipe. The hourly flow duration curve yields average flow rates of 1.82 m^3/s and 1.47 m^3/s at 50% and 90% probability of exceedance (Figure 5). The flow variation in the raw water main is small because there are no off-takes along the line. The only variations that may exist are due to leakages and transmission losses along the line. The raw water flow can also be theoretically estimated if the head between inlet and outlet is known, as well as the pipe size, type, and total length.

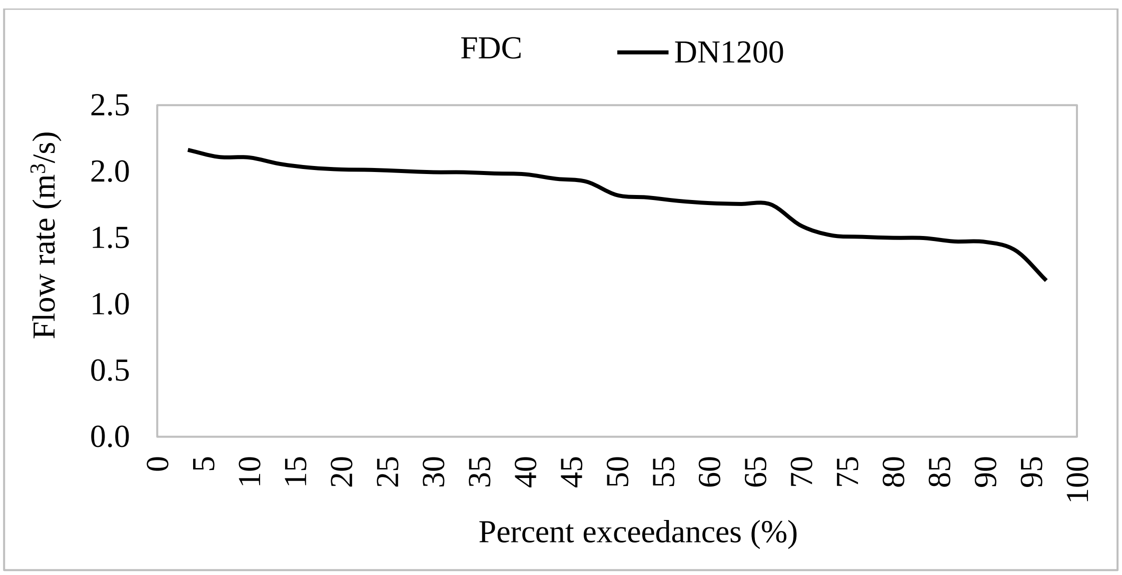

Figure 5. Flow duration curve for raw water main (DN1200).

Using flow information from system records, the flow variations of the treated water mains were determined statistically by calculating the mean (average) and standard deviation. The variations between the outflows (i.e., Legedadi clear water tanks) and inflows (i.e., Terminal service reservoir) points were compared. As illustrated in Figure 6, the flow variations for DN900 were quite larger than DN1200 owing to several off-takes tapped along the transmission mains. For instance, there are four off-take points for the DN900 pipeline near Civil Service University, Saint Michael Church, CMC roundabout, and Ayat roundabout, whereas for the DN1200 pipeline, there are only two pipelines near CMC roundabout and Saint Michael Church. Off-takes create pressure drop along the pipeline thus reducing the flow rates downstream. Because of leakage at fittings along the transmission mains, the flow rate may be reduced. This makes the DN1200 transmission line the most preferable site for integrating an in-pipe turbine system for hydropower generation.

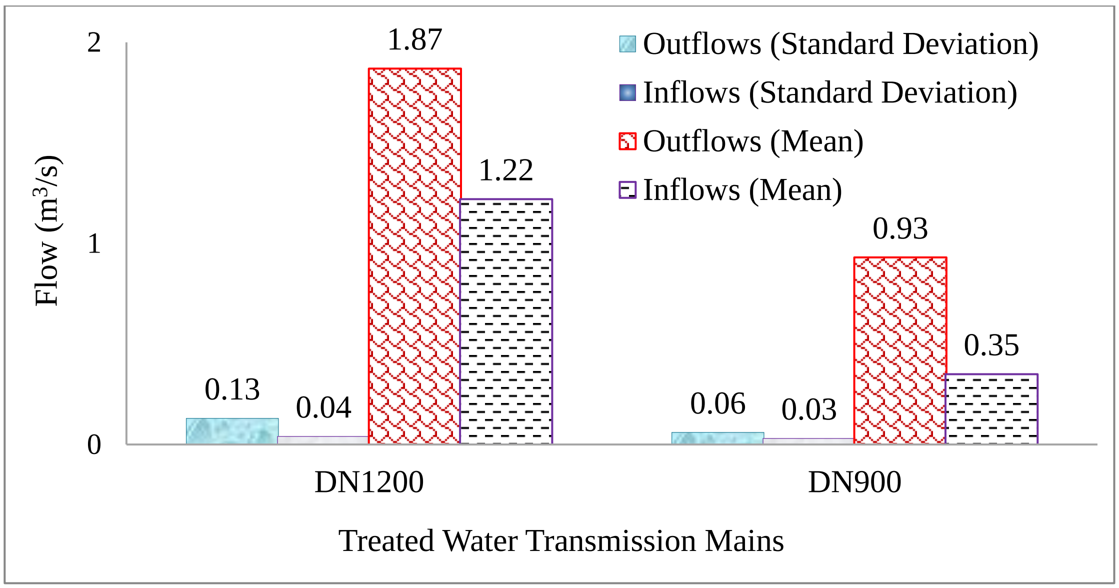

Figure 6. Inflows and Outflows for DN900 and DN1200

Flow duration curves for the inlet point at terminal reservoirs for both DN900 and DN1200 transmission mains are shown in Figure 7. The discharge values for DN1200 treated water main at 50 and 90 percent of exeedience were (1.24m^3/s and 1.14m^3/s, respectively). According to the FDC analysis, the values for DN900 at the same levels of exceedence were 0.34 and 0.29 m^3/s. Because of high flow variations in a pipe, the FDC for treated water mains was nearly horizontal, as shown in the figure. The variations in flow could be ascribed to leakage along the transmission mains and off-take points, as indicated previously. The duration curves showed the expected flow profiles.

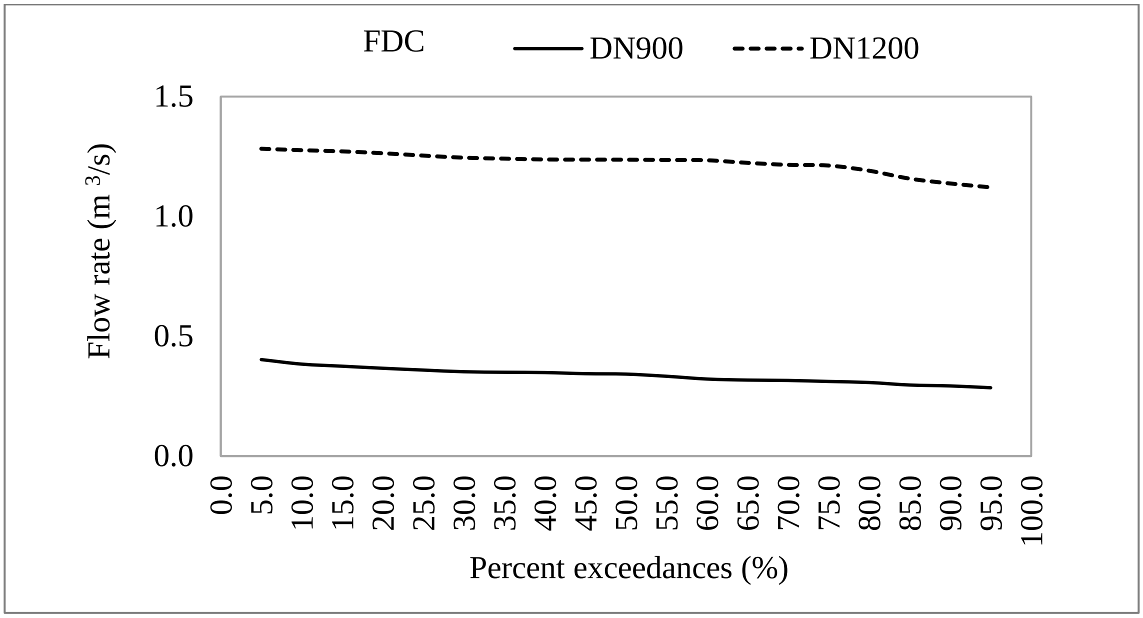

Figure 7. Flow duration curve for treated water mains (DN900 and DN1200)

4.3. Summary of Potential Sites

The identification of potential sites enables the determination of the potential power. This section summarizes the potential sites depending on both discharge and head. There are two main potential areas in the Legedadi Water Supply Scheme: Legedadi Water Treatment Plant and Kotebe Terminal Reservoirs. The location of the site for hydropower generation should be preferably before water treatment works (Kucukali, 2011) because it would be easier to extract more raw water to compensate for the losses. However, in the case of treated water, compensation might be restricted by the capacity of the treatment plant. Another reason was that when the external system was to be installed on treated water mains, they would likely compromise the quality of water. Due to this reason, it is recommendable to locate hydropower sites before the water treatment works or the distribution network (Loots et al., 2015). In this study, potential sites were identified for both raw and treated water mains. For the raw water main, there was only one potential site owing to the short distance between the intake and the treatment plant. This site was identified by its head and discharge. For treated water mains, eight sites were identified using the same criteria at the raw water pipe. Table 1 shows all the potential sites for both raw and treated water mains. Tables 2 and 3 show the RET raw and treated screen output parametres.

Table 1. Potential site for Raw and Treated water mains

Raw Water Main						
Site	Elevation (m)	Distance to intake (m)	Gross head (m)	Diameter (mm)	50% Primary flow (m^3/s)	90% Secondary flow (m^3/s)
TP	2447.54	550	12.46	DN1200	1.82	1.47
Treated Water Mains						
					50% Primary flow (m^3/s)	90% Secondary flow (m^3/s)
Sites	Elevation (m)	Distance to TP (m)	Gross head (m)	Diameter (mm)	DN 1400 and 1200	DN 1400 and 1200
1	2345.2	0+409.69	77.35	1400	1.72	1.42
2	2352.5	1+040.41	70.05	1400	1.72	1.42
3	2363.8	2+253.33	58.75	1400	1.72	1.42
4	2364.3	6+760.12	58.25	1400	1.72	1.42
5	2356.4	11+080.31	66.15	1200	1.24	1.14
6	2385.4	14+340.41	37.15	1200	1.24	1.14
7	2392.1	15+295.13	30.45	1200	1.24	1.14
TR	2403.4	18+400.13	19.15	1200	1.24	1.14

Power and Energy estimation using RET Screen

Table 2. RET Screen output parameters for raw water main

Parameters	
Gross head (m)	12.46
P50 (Kw)	186
P90 (Kw)	150
E50 (Mwh)	1,397
E90 (Mwh)	1,208

Table 3. RET Screen output parameters for treated water mains

DN1200								
Gross heads (m)	77.35	70.05	58.75	58.25	66.15	37.15	30.45	19.15
P50 (kw)	639	579	487	483	548	310	254	159
P90 (kw)	587	532	448	444	503	284	233	146
E50 (MWh)	5,066	4,594	3,862	3,830	4,342	2,454	2,013	1,264
E90 (MWh)	4,790	4,344	3,652	3,621	4,105	2,319	1,903	1,193
DN900								
Gross heads(m)	77.35	70.05	58.75	58.25	66.15	37.15	30.45	19.15
P50 (kw)	189	171	144	143	162	91.5	75.1	47.2
P90 (kw)	168	152	128	127	144	81.6	67	42.2
E50 (MWh)	1463	1327	1116	1106	1254	710	583	366
E90 (MWh)	1371	1244	1046	1038	1176	666	547	344

From power potential considerations for treated water mains, all the sites were most favorable except the inlet of the Terminal Reservoir (site having a head of 19.15 m). However, extracting power at these sites will result in a significant reduction in the flow downstream. These sites are also far from the centers that give power service, so there will be transmission losses along the power lines. Therefore, for the above-mentioned reasons, the inlet of the Terminal Reservoir for the treated water mains was selected as the most suitable site.

Impact Characterization of Generating Power on Exixting System

The impact of generating power from an existing system can be characterized by comparing flow, head, and power produced. Figure 8 shows the relationship between the power output and the percentage reduction in flow for the selected site at a head of 12.46 m. For instance, at 158 kw power output, the percentage flow reduction was 2.04% (0.03 m^3/s), while at 167 kw power output the percentage flow reduction was 6.8% (0.1 m^3/s) for the DN1200 (raw water main).

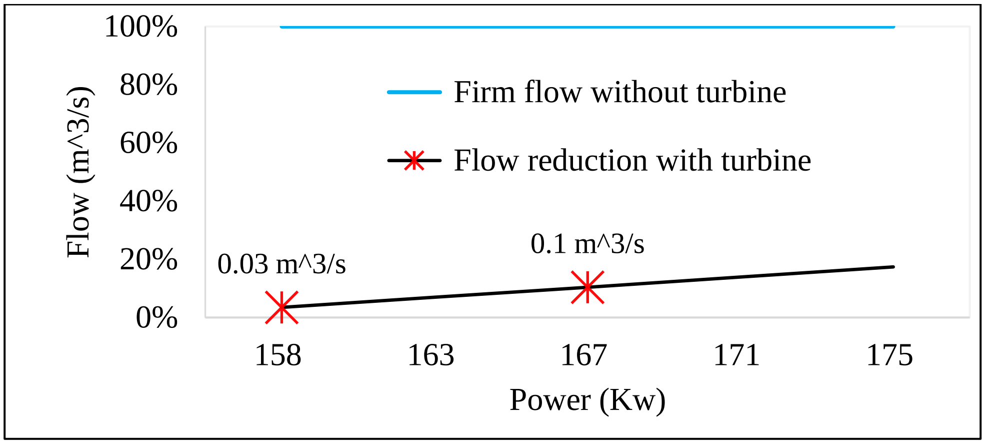

Figure 8. Percentage flow reduction against power at the raw water pipe

The relationship between the power output and the flow reduction for the treated water mains at a selected site of head of 19.5 m is shown in Figure 8. Compared to the raw water mains, the reduction in flow of the treated water mains was two times less per unit power output. In the case of raw water, the reduction was 0.6 liters per kW, while for treated water mains it was 0.3 liters per kW. The reduction of treated water, on the other hand, would be more sensitive than the raw water mains because it could affect the service reservoir level.

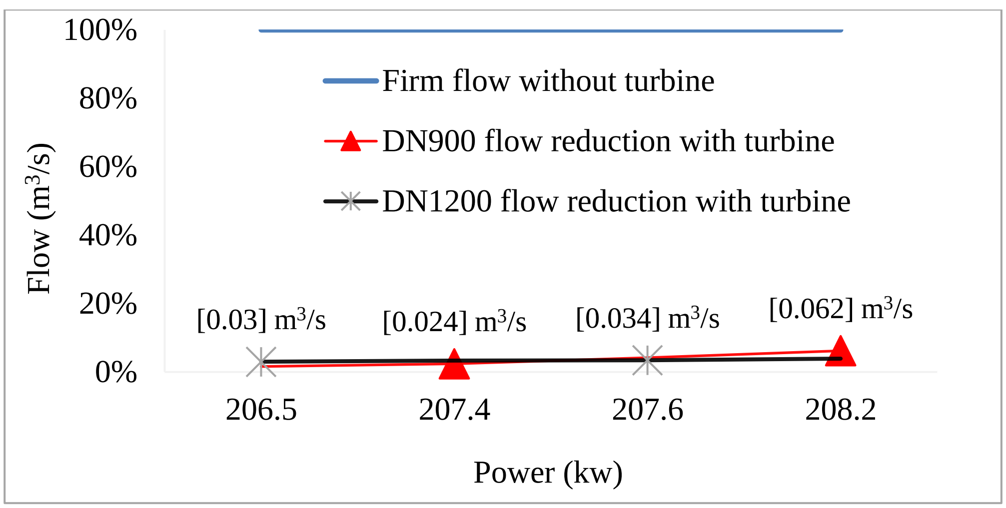

Figure 9. Percentage flow reduction against power at the raw water pipe

Financial viability

The selection of project classification is an important parameter for the correct evaluation of project costing. This is due to larger projects requiring more traditional designs with higher associated risks. The other parameters that affect the financial evaluation were the type of hydro systems selected and the grid type either isolated or connected with central. It is assumed that like other micro projects, the inflation rate of Ethiopia for the year 2020 was considered at 19.5%. The life of the project was 20 years, with a 70% loan of the initial cost, with an interest rate of 7% and a debt term of 15 years. So, 322,700$ was obtained from a loan, and 138,300$ came from the investors, or the equity portion. The total initial cost of the project was 461,000$. The annual cost and debit payment for the 15-year time period was 40,978$ which included the operation and maintenance costs. The annual electricity export revenue was estimated to be 75,946$. Similarly, the hydropower project for the treated water lines (DN1200 and DN900) provided power for 20 years. According to cost analysis, the total initial cost of the project was 414,500$ (for DN1200) and 135,900$ (for DN900). So, 290,150$ (for DN1200) and 95,130$ (for DN900) were obtained from a loan, and 124,350$ (for DN1200) and 40,770$ (for DN900) came from the investors or the equity portion. The annual cost and debit payment for the 15-year time period were 38,074$ (for DN1200) and 11,669$ (for DN900), which included the operation and maintenance costs. The annual electricity export revenue was estimated to be 75,068$ (for DN1200) and 18,842$ (for DN900) (Table 4).

Table 4. Financial Viability

Cases	DN1200 (Raw)	DN1200 (Treated)	DN900 (Treated)
Generated energy (Mwh)	1,208	1,193	344
Estimated total cost ($)	461,000	414,500	135,900
Annual saving ($)	75,946	75,068	18,842
Pre-tax IRR-equity (%)	2.4	1.9	4.9
Pre-tax IRR-assets (%)	positive	positive	Positive
Simple payback (yrs.)	6.5	6	7.7
Equity payback (yrs.)	3.9	3.3	5.5
Benefit-cost (B-C) ratio	1.4	1.4	1.1

The hydropower project in the Legedadi Water Transmission Line was feasible according to RET Screen-small Hydro Model as the Net Present Value (NPV) and Internal Rate of Return (IRR) for both raw and treated water sites were positive and the benefit-cost ratio (B/C) was above one which is shown in (Table 4). The simple payback was 6.5 years and 6 years for raw and treated water sites, respectively (Table 4). According to the analysis, the raw water main and treated water main (DN1200) had the best benefit-cost ratio; hence it can be concluded that it was better to generate power from it.

5. CONCLUSIONS AND RECOMMENDATIONS

Adequate head and flow are requirements for hydropower generation. The Legedadi Water Supply Scheme has the potential to produce electricity. The raw water pipe had a gross head of 12.46m, an average flow of 1.47 m3/s (available 90% of the time), and provided 150 kW for a 1200 mm diameter pipe with a total length of 550 m from the intake. The treated water pipes had a gross head of 19.5m, average flows of 1.14 m3/s (available 90% of the time), and produced 146 kw and 42.2 kW for 1200 mm and 900 mm diameters, respectively, with a total length of 18.4 kilometers. As to the analysis, the hydropower size from the pipes of Legedadi water supply transmission falls in the range of Pico-to Small-scale Projects in general and can be taken as a mini hydropower project in particular. According to economic analysis the project is feasible.

Certainly, more precise results would be realized in the case of considering detailed data considering the quality of pipe flow data and actual survey data of the pipeline that determine the power potential of the water supply scheme, which were not considered in this study. Hence, the results of this study should be taken as an initial basis for further studies of power assessment.

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