University of Southern Queensland

Faculty of Health, Engineering and Science

Hydraulic Modelling of the Water Distribution System of Tavua/Vatukoula, Fiji

Final Dissertation submitted by

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ABSTRACT

This study assessed the performance of the drinking water distribution system of the Tavua/Vatukoula network, Fiji. An important aim and a requisite of the work was the collection and assessment of pertinent data, to enable the development of a detailed hydraulic system model for analyses and simulations.

This study utilized EPANET v2, a popular and freely available software package developed by US Environment Protection Agency. The major features and most important characteristics of EPANET presented and discussed in detail in this report.

The technical focus of this study was mainly on available water pressures in various hydraulic loading scenarios. Pressure conditions were determined for average daily demand, maximum hourly demand, minimum demand, and peak hourly at maximum water plant delivery, as well involved extended (24 h) simulation runs. These examinations revealed various bottlenecks and shortcomings in the network, and provided insight into the underlying causes to suggest effective remediation approaches.

System simulations show that currently the Tavua/Vatukoula system is capable of providing satisfactory supply to the gross majority of customers, and only several distinct locations experience pressure deficiencies. System capacity also would allow the connection and supply of the proposed new sugar mill south of Tavua town (up to 4.63 L/s continuous demand) with not significant impact on existing customers. However, service would be severely limited for increased demand, and widespread pressure problems would arise at maximum water plant deliveries, indicating that network augmentation will be necessary in the coming years.

There was no sufficient data to allow for any disinfectant residual modelling. Nevertheless, water age simulations were carried out over extended (168 hours) periods, which identified various mains and branch pipelines prone to water stagnation. Analysis of operational issues, such as access for maintenance, placement of pressure regulating (PRV) valves, and the promptness with which repairs can be made, among other multidisciplinary concerns, were outside the scope of this study.

While various shortcomings and problems of the water network system were identified in this study, the current hydraulic model has limitations and will benefit from future improvements. The design and planning of remediation and augmentation works will require more and better quality data for model calibration.

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LIMITATIONS OF USE

CERTIFICATION

I certify that the ideas, designs and experimental work, results, analyses and conclusions set out in this dissertation are entirely my own effort, except where otherwise indicated and acknowledged.

I further certify that the work is original and has not been previously submitted for assessment in any other course or institution, except where specifically stated.

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Student Number: 0050105019

Signature

Date**: 29/10/2014**

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Everything I have accomplished is simply a sign of God's grace to me. I have taken a journey that seemed very discouraging as I tried to settle in at the beginning of the first Semester of 2014. Now, looking back at what I have been able to achieve since then, I am glad I kept the faith and the resolve to see the end.

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Contents

List of Figures

1. INTRODUCTION

1.1 PROJECT BACKGROUND

1.1.1 The region

The Tavua/Vatukoula Region is located in the northern central part of Viti Levu as shown in Figure 1. The Region comprises the coastal plain between Qalela and Rabulu in the east-west direction and Vatukoula to the South in the Nasivi Valley.

Figure 1: Map of Fijis' main island (Viti Levu) showing the location of Tavua/Vatukoula (s*ource: Google maps)*

Tavua is a small country town in the province of Ba in the northern part of Viti Levu, and was declared a town in 1992.

Vatukoula is a gold mining centre located approximately 7 km south of Tavua. The economy of the area is based mainly on sugar and gold mining industries. The town centre is comprised of a number of shops, government offices including a small hospital, and is surrounded by numerous village/settlement and schools within close range of each other. Vatukoula is well known for its gold mining and also has a small industrial estate.

The area to be served is shown in Figure 2.In general, it extends from Rabulu to the east and Qalela to the west. Vatukoula settlement, the Industrial and Housing Authority Estate at Vatukoula, and sporadic development between Vatukoula and Tavua are included in the area to be served. Essentially, the area includes development below the 75 m contour level to the east and 60 m contour level to the west. Village and settlement development has generally been in the valleys and lower slopes of valleys, and along the coastal plain. The most heavily populated area in the Nasivi valley.

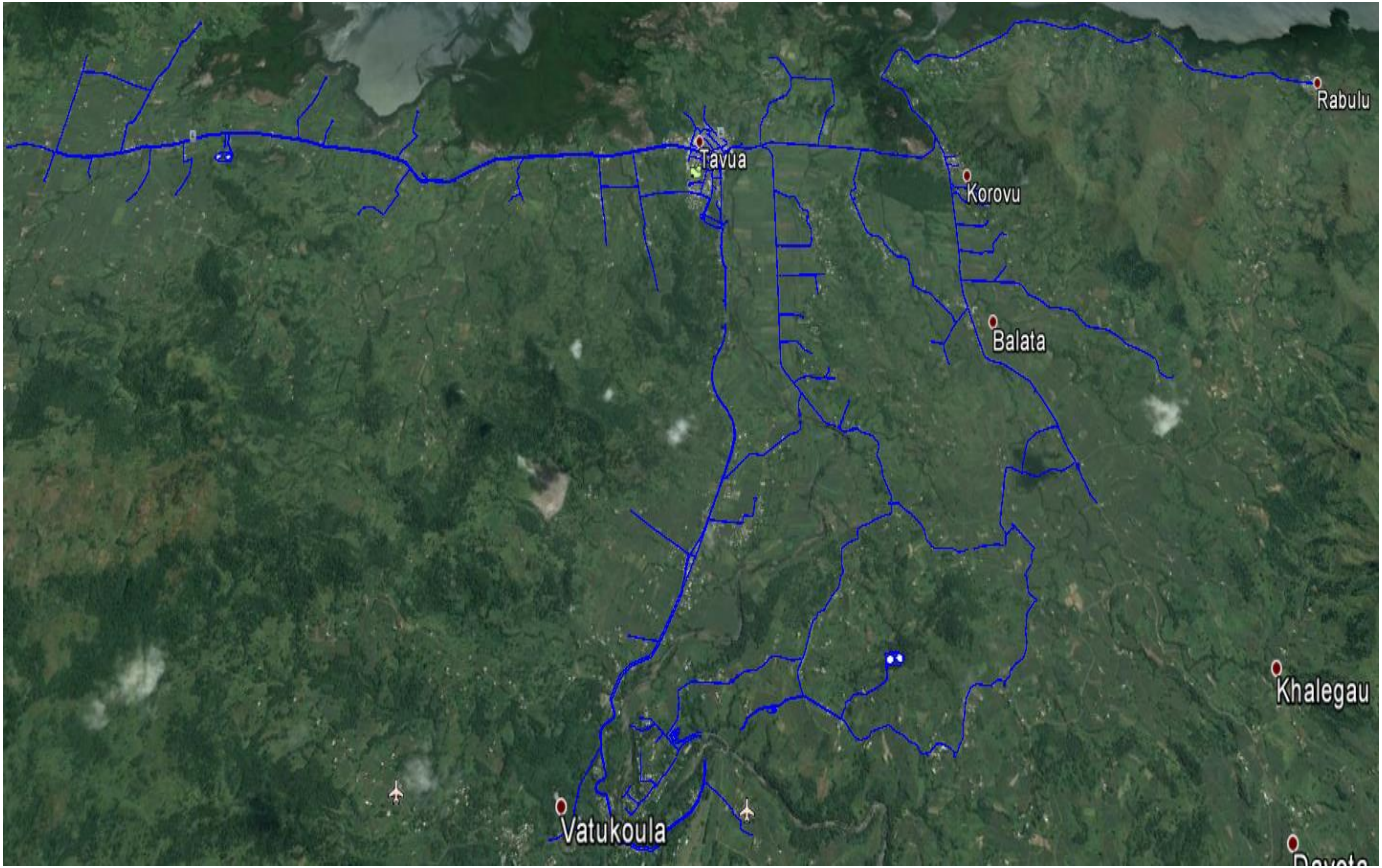

Figure 2: Tavua/Vatukoula water distribution network

The area includes the area of Masimasi around Lakalaka trig station which is currently supplied by tanker, and Vatukoula township to the extent of providing service to the extent of providing service to institutional in Vatukoula.

The whole of the area is volcanic with Vatukoula occupying the caldera of extinct volcano. The geomorphology of the region is summarised in the Harrison Grierson Consultants Ltd reports of 1985. That report identifies the Nasivi River with its catchment of 95 km², as the principal river of the Region, and the only viable water source for a regional water supply. Other water sources are reported as ephemeral, with low or no flow in droughts, and groundwater sources of significant yield within the study area could not be identified.

The Yaqara River, approximately 5 km east of the Study area with a catchment of 76 km² is the closest substantial source of additional surface water potentially available.

1.1.2 Alternative water resources

The hydrological interpretation is difficult given no records of mine abstraction, and there are gaps in the hydrological flow records, which have required synthetic substitution based on the Nakauvadra catchment flow records. From the analysis, a 15 year return low flow of 170 L/s has been calculated (equivalent to 14,700 kL/day). However, the abstractions for the Mine cannot be gauged under such severe drought conditions, and may be significantly greater than normal, given the Mine's need to circulate more than 16,000 kL/d with potential needs up to 25,000 kL/d (300 L/s). Overall, the competition presented by the Mine can potentially overwhelm the yield of the Nasivi River, useless the conditions on its consent are strictly enforced to maintain a minimum residual low flow of 100 L/s.

Investigations have revealed that a source of groundwater exists at the Yaqara River, just outside the study area to the east, and initial reports (so far unconfirmed) claimed that a bore has been tested to 2,200 kL/day. If the supply is proven, and reports are confirmed that the water quality is of potable standard, it presents the most viable sources to supplement supply from the Nasivi River when low flows occur. A bore (or bores) could be constructed and bore pumped installed to supply direct to the distribution system at Rabulu.

The second alternative is to abstract surface water from the Yaqara River, treat it, and pump into the distribution system. The latter presents the least desirable option, requiring both expensive treatment and pumping, and is not favoured; particularly as its 15 year low flow predicted by this master plan study is only 70 L/s.

It is possible that abstractions from the Nasivi River can be improved by construction of a weir (or alterations to the existing weir), so as to capture all low flow into the intake. This would be justified only if some control of the mine abstractions can be exercised. Under the Special Licence issued by MRD, Vatukoula Gold Mine is supposed to forward to MRD their abstraction rates, and when necessary, MRD is to exercise its power to control the abstraction.

1.1.3 Population growth

A previous study took advice from the National Planning Office, Bureau of Statistic, to interpret population figures and predict population growth. The population was expected to grow at the rate of 1.1% per annum, according to the Tavua Local Advisory Plan (prepared by Director of Town and Country Planning, 1982). By 2016, the total population within the area to be served by the regional supply was projected to be 27,000, including re-housed Vatukoula mining residents, to attract an estimated water supply demand of 6,500 kL/d.

The population growth statistic of the area between the since 1986 has greatly been affected and distorted by the political climate, which (especially until 1996) saw an abrupt emigration of a large number of the Indian ethnic population from the area. It is unclear if there will be a significant return to previous population diversity and number.

1.1.4 New development areas

Meetings were arranged with a number of central and local government bodies to determine areas likely to be developed in the planning period. Given the generally lower increase in overall population than historically predicted, not all developments are likely to proceed and the order of development also cannot be accurately defined. The approach in this study has been to identify the areas for development, the population numbers that have been assigned, and thus the ultimate water demand for those areas. The trunk pipeline network has been designed to be able to satisfy the demands of individual developments but only to the aggregate level of the predicted total population. Some judgement has been exercised in satisfying those conflicting requirements. It is expected, however, that much of the population of Vatukoula will be rehoused at Toko, Vatuveleka, Korinisalusalu, and Nabuta early in the planning period, as soon as water is available. This is seen as highly desirable since Vatukoula's water supply that is under the Mine Company's control, is untreated and thus presents a public health concern.

1.2 EXISTING WATER SUPPLY SYSTEM

1.2.1 Overview

The existing Tavua/Vatukoula Regional System consists of an Intake Pumping Station at Vatukoula, which draws water from the Nasivi River, the Water Treatment Plant at

Vatukoula Industrial/Residential Estate, and the Main Storage Reservoir at Yaladro Cemetery; these three sites being linked by 300 mm dia. AC pipelines together with the distribution/reticulation network.

The intake, treatment plant and reservoir were constructed over a period of time from 1986 to 1989 and commissioned in November 1989. These superseded the previous intake and treatment plant built in early 1950's situated at Tavua.

The Treatment Plant has been built to treat 5,500 kL/d, and currently provides water to just over 50% of its design capacity. The plant has been designed for expansion to an ultimate capacity of 7,500 kL/d. From the Treatment Works, the treated water is pumped to the Main Storage Reservoir of 3500 kL capacity at a TWL elevation of 84 m above Mean Sea Level (AMSL) at Yaladro Cemetery, from where it is distributed to consumers via distribution/reticulation mains with sizes ranging from 50 mm to 300 mm in diameter.

The present system layout is shown on Figure 2. It serves areas along Vatukoula Road (from Vatukoula Water Treatment Plant to Tavua Town), Tavua Town proper, along Kings Road (from Qalela in the west to Kavuli in the east), and along Nadarivatu Road (up to Balata).

The Vatukoula Gold Mine has its own water supply system which delivers untreated water for domestic and industrial use in the mine's township and mines. A Commission of Inquiry into Vatukoula Gold Mine conducted in 1994/1995 recommended that water supply to residents working for the company and staying within the Mining area be improved, particularly with regard to its quality. The untreated water for domestic use is becoming a major concern, and the Mine has approached the Government to provide them with treated water.

Water is also abstracted upstream of the WAF's Treatment Plant Intake by the Vatukoula Gold Mine which can exercise its Rights (under a Special Licence issued by the Mineral Resources Department) to abstract 20,000 kL/d over and above any other uses of the Nasivi River water. A condition of the Mine's Right is that residual flow following the Mine's abstraction of the water shall exceed 100 L/s (8,600 kL/d) at all times.

1.2.2 General

Water connections are provided to any person requiring supply. There is no restriction made on customers as to what the water is used for the quantity taken.

A recent review of Quarterly Customer Water Usage reports found some individual consumption above 22 kL/d for connection addresses in rural areas. While these records appear suspect, there are no means of validation, and more vigilant monitoring of usage is recommended.

A master meter reading for water produced at Vatukoula indicated 90,000 kL usage for April 1997. The meter has not been in operation since that time. Moreover, in July 1997 there were 132 connections meters out of order, and that number is reportedly increased each month. A recently initiated programme to replace these defective meters is under way.

Applications for new connections are being received at an average rate of four per month.

1.2.3 Raw water source

Water from the now abandoned Tavua plant was originally pumped from the Nasivi River at a location opposite the WAF Depot, 1.4 km up Vatukoula Road from Kings Road. Water for the Vatukoula plant is taken from Nasivi River at a point approximately 9 km up river from Kings Road. The intake is 100 m downstream of the WAF river flow gauging weir. There is no water abstraction right issued for the Vatukoula plant intake.

Emperor Gold Mine (EGM) takes water from Nasivi River 100 m upstream of the gauging weir. EGM have a water right from abstraction of up to 20,000 kL/d, as long as the flow in the Nasivi River immediately downstream of the intake is greater than or equal to 100 L/s.

1.2.4 Tavua township reticulation

Primary reticulation in Tavua is generally comprised of cast iron and asbestos cement pipes. Cast iron pipe is prone to relatively high leakage at joints with time, and to a decrease in diameter caused by the formation of nodules on internal surfaces. Asbestos cement pipes generally do not suffer from these problems but their use has been reduced worldwide due to health risks associated with asbestos fibre.

Extension to reticulation in recent years have been constructed using plastic (UPVC, and in some cases PE) pipes.

A recent study of water leakage in Suva, where reticulation is considerably older, has conservatively estimated leakage at 50 percent of water production, with 60 percent reported in some areas. According to WAF estimates, water loss due to leakage in Tavua is only 10-15 percent. “Unaccounted for” water, which includes leakage and illegal and non-metered connections is estimated to be 20 to 30 percent of the total production.

Tavua and Tavualevu Village are supplied with water from reservoirs situated at the original Tavua water treatment plant site. The reservoirs have a combined capacity of 795 kL and a top water level of 59.6 meters AMSL. Currently there are no means of recording water delivery from these reservoirs. Because the reservoirs are supplied from the higher Vatukoula reservoir, there is a requirement to isolate them when they are full to prevent overflow. This is achieved by an operator monitoring water level and manually opening and closing the entry valve.

Tavualevu Village is reported to be the fastest growing and largest village in the western area. Because of the condition and small size of pipe network in the area, it is not possible to supply sufficient water to match existing demands, and new connections are no longer installed.

1.2.5 Tavua/Vatukoula reticulation

The general Vatukoula area is served by two water supply systems, namely that of WAF and of Emperor Gold Mine (EGM), noting that the latter is not treated.

The WAF reticulation system has between 193 installed meters (local information) and 178 meters (Water Billing records) to record consumption. Of the 178 meters, six are commercial (showing an average 4,779 L/d meter usage), 171 are domestic (1,271 L/m d) and one is Governmental (2,522 L/d). Domestic supply is to the Nasivi housing area and the Vatukoula High School.

The EGM system is primarily to serve mining operations but also supplies water to Loloma, Korowere, Low Cost, and Matanagata housing areas. According to EGM estimate, 4,600 people live in these four areas. This estimate compares with the 1996 census figure of 6,635 for the total Vatukoula Urban Area. Many cases of gastro-enteritis infection, particularly in young children, are reported from Vatukoula area, and probably are caused by faecal contamination of the Nasivi River source. For an interim measure to alleviate the problem, WAF has installed a hydrant and meter at Vatukoula for residents to collect treated water for drinking and cooking purposes.

EGM reticulation is made of galvanised steel pipe and generally of good external appearance and condition where it is able to be viewed. Internal condition is not known. EGM would prefer that WAF supplied water to all residents, and have indicated that a large quantity of existing reticulation pipework could be made available for this purpose. Some new primary reticulation would need to be constructed.

The decommissioned Filtration Plant, which was connected to the now abandoned EGM scheme at Vatukoula, supplied water to some residences in the Vatukoula area. A small number of those houses (approximately 6) were supplied by a pump/filter system. A proposal to construct another reservoir at a higher elevation to enable gravity supply to these houses did not proceed when this system was decommissioned. These houses are now served by a water truck, regularly delivering water to individual house tanks.

1.2.6 Rural reticulation

Water for rural areas is supplied directly from the WAF Vatukoula headwork. To the west, the main demand areas for water are about 7 km distance from Tavua up to Qalela. The relatively small diameter supply main (predominantly 100 mm diameter) and the small feeder pipes branching off the supply pipe create relatively large friction losses, and the pressure is inadequate in outer areas.

To overcome this lack of supply, the Ministry of Regional Development arranged finance for construction of bores at individual settlements. On commissioning, the works are taken over by a committee of residents. The water is untreated and is not reticulated to individual houses. The largest of these schemes at Vugele serves some 600 people.

The Tagitagi borehole at the junction of Matacawa and Kings Road was abandoned (date unknown) because water became unsuitable for domestic consumption. The reservoir associated with this borehole supply has been connected to the rural reticulation but is presently valved off because the level operated valve requires repairs.

Rural reticulation in the east has large diameter (200 mm) supply main constructed recently along Kings Road, and distances to supply points are shorter than in the west. The higher pressures maintained by the large main and shorter distance to connections would explain why there is a lower number of supply problems in the east than in the west. The Ministry of Regional Development and Multi Ethnic Affairs borehole supply schemes in the east are in areas beyond the extent of the present WAF reticulation network.

1.3 CURRENT AND FUTURE DEMANDS

1.3.1 General

The economy of the area is based mainly on sugar and gold mining industries. Tavua town centre is comprised of a number of shops and government offices, including a small hospital, and surrounded by numerous villages/settlements and schools within close range of each other. Vatukoula is well known for its gold mining activities and also has a small industrial estate.

The extent of the area includes the populated areas of Tavua and Vatukoula and the surrounding rural areas as outlined on Figure 2. The study area boundary in the Vatukoula area has been taken as that area that can be served by gravity supply from the Vatukoula reservoir (TWL 84 m ASML). In outer rural areas the boundary has generally been set at the 60 metre contour. This 60 metre elevation is somewhat arbitrary, but is found to permit gravity supply with economically sized pipelines. The 60 metre contour marks the edge of the gently sloping coastal plain and the steeper sloped hill areas.

It is apparent that the long-term stability of Tavua/Vatukoula region is largely dependent on the continuing operation of EGM. If the mine were to close or scale down its operations, a large number of people would need to be relocated to find alternative employment. It is very unlikely that the general region could establish sufficient industrial or commercial activity to accommodate reluctant mine employees. Continued mine operations are dependent on world gold prices. The study assumes that whilst there will be periodic changes in gold prices, the situation will remain stable enough to allow EGM to continue operations throughout the study period.

1.3.2 Census information

Because of problems experienced by the Department of Statistic in preparation and gathering of census returns, the census returns, the 1996 census detailed information has largely not been available for use in the study. The study has used some preliminary 1996 census information and some provisional results published in Statistical News Nº 8 1997 to adjust 1986 census results to present day.

The 1996 census has shown a dramatic change in growth rate for the period 1986 to 1996 compared with the previous 30 years. The following extract from Statistic News Nº 8 lists the main features of the 1996 census observations:

- Declining growth rates overall and the lowest growth rate between censuses since the 1901 Census

- Smallest increment in population size in recent decades.
- Dramatic change in ethnic composition of total population.
- Decrease in the overall number of the Indian people due to largely to a fall in the rural population
- Intensified urbanisation most especially by the ethnic Fijians. Urban dwellers now comprise 46.4% of the total population.
- Decrease in the proportion, as well as the size, of the overall rural due to heavy Indian losses and a small Fijian increase.

Annual Average growth rates between the last few censuses were as follows:

Table 1 - Annual average growth rate %

	1956-1966	**1966-1976**	**1976-1986**	**1986-1996**
Fijian	3.1	2.5	2.4	1.8
Indian	3.5	2.0	1.8	-0.4
Others	1.7	0.3	0.6	0.9
Total	3.2	2.1	2.0	0.8

The Census Bureau has provided the following Projected Average Annual Population growth rate for High, Medium and Low variants at 10 year intervals for period 1996 to 2016

Table 2 – Projected average annual population growth

Ethnic Origin and Variant	**1996-2001**	**2001-2006**	**2006-2011**	**2011-2016**
Fijian %				
High	1.9	2.0	1.9	1.8
Medium	1.7	1.5	1.5	1.3
Low	1.6	1.4	1.3	1.5
Indian %				
High	0.8	1.3	1.2	1.1
Medium	0.4	0.2	0.5	0.3

Low	-0.01	-0.1	-0.1	-0.4
Others % High	2.4	2.3	2.1	1.9
Medium	0.9	0.9	0.8	0.6
Low	0.9	0.9	0.8	0.6
Total High	1.5	1.7	1.7	1.5
Medium	1.1	1.2	1.1	0.9
Low	0.9	0.9	0.7	0.5

The study has used the Medium Variant for estimation of future populations.

1.3.3 Proposed development areas

Information from interviews with Tavua Town Clerk and officers of the Department of Town and Country Planning and the Housing Authority has been used to assess the likely areas, types and times of large-scale urban development area within the Study region. The locations of these areas are shown on Figure 2 and the details of each are summarised in Table 3.

Table 3: Urban development areas

Locality/Name	Area(Ha)	Class	Lots/Acre	Total Lots	Population (at 5/lot)	Year Start	Year Finish
Korinisalusalu	6	D	12	141	705	2001	2006
Vatuveleka	12.8	D	12	304	1520	2006	2011
Saunakavika	7.8	D	12	184	920	2011	2016
Nabuta		B&C	4.5&8	147	735	2003	2011
Toko		C&D	8&12	750	3750	1999	2011
				1526	**7620**		

The classification and/or timing of development and hence the number of residents could be affected by the availability of reliable water and sewerage reticulation. Class C and D development section sizes are considered too small to allow adequate septic tank leaching areas. For the purpose of this study, sewerage reticulation is assumed to be available.

The present Town Plan shows a 1996 approved sub division located at the south end of Nabuna Street. The development of this subdivision is reportedly waiting for sewerage reticulation within the town. Further areas of land allocated for residential use are located on the town extreme south boundary between the Fijian Burial Ground and Kings Road and a long narrow strip between Vatukoula Road and Nasivi River.

A Tax Free Zone development, located partly within the town boundary, has been approved on "tiri" land at the northwest corner of the town. This development did not proceed for reasons included in the Department of Town and Country Planning Tavua Town Survey Report of 1997. An industrial subdivision, located partly within the town boundary, at the northwest corner of the town has also been proposed but has not proceeded.

1.3.4 Vatukoula urban area.

EGM has about 2,300 employees on site each day and some 30 percent of staff live in areas away from Vatukoula.

EGM have sold most of former company houses in Vatukoula to employees but retains land ownership. EGM encourages home owners to move their homes off EGM land but is not forceful on this matter.

The Study has assumed that because of EGM policy of encouraging relocation, the availability of development areas, and a probable reluctance on the part of WAF to take over existing EGM water reticulation pipework, there will be a lack of treated water supply to houses in Vatukoula. That ultimately the urban population in Vatukoula will be relocated to the new housing development areas at Toko, around Tavua and Nabuta. This concept was discussed with local and central government officials who were concerned with this approach.

1.3.5 Population distribution

Table 4 sets out the possible population distribution within the region using the foregoing Fiji national growth projections in Table 5, which are more relevant to Tavua.

Table 4 - Population distribution - Case 1

Census Year	1976		1986		1996		2006		2016
Headwater	7779	i.	6065	ii.	7911	ii.	8702	iv.	10007

Central	4107	i.	3776	ii.	4078	iii.	4486	iv.	5159
Western	3175	i	4634	ii.	5005	iii.	5505	iv.	6331
Eastern	1088	i.	2717	ii.	2934	iii.	3228	iv.	3712
Far Eastern	1400	i.	1450	ii.	1566	iii.	1723	iv.	1981
TOTAL	17549		18642		21494		23644		27190

Notes: *(i) Guess based on HGC info of 1984 Health Dept. Survey at 1996 Census; (ii) 1986 Census x 1.08 (8% over 10 years); (iii) 1996 values x 1.10 (10% over 10 years); (iv) 2006 values x 1.15 (15% over 10 years)*

Table 5 - Population distribution - Case 2

Census Year	**1976**		**1986**		**1996**		**2006**		**2016**
Headwater	7779	i.	6065	ii.	7911	iii.	9098	iv.	10917
Central	4107	i.	3776	ii.	4154	uii	4777	iv.	5732
Western	3175	i.	4634	ii.	5097	iii.	5862	iv.	7034
Eastern	1088	i.	2717	ii.	2989	iii.	3437	iv.	4124
Far Eastern	1400	i.	1450	ii.	1595	iii.	1834	iv.	2201
TOTAL	17549	i.	18642		21746		25008	iv.	30009

Notes: *(i) Guess based on HGC info of 1984 Health Dept. Survey at 1996 Census; (ii) 1986 Census x 1.08 (8% over 10 years); (iii) 1996 values x 1.10 (10% over 10 years); (iv) 2006 values x 1.15 (15% over 10 years)*

Table 4.3 shows the adopted population distributions for settlements and villages. These projections are more pertinent to Tavua and have been agreed with the Bureau of Statistics. These are the projections adopted for the study.

1.3.6 Current demand

Estimates of per capita and institutional demands were assessed from a number of different records. As noted earlier, the supply meter at Vatukoula treatment plant has been in operation since May 1997 and the last record indicates consumption of 90,000 kL/month or 3,000 kL/d.

The number of metered connections is approximately 2450. Using 1986 census data the average dwelling occupation density for Tavua and Vatukoula urban areas is 5.7 persons/dwelling. Using an overall value of 5.7 persons per dwelling for the supply area and assuming 2300 dwelling connections, the present average per person consumption is 229 litres per day. This residential consumption rate includes commercial and government facility consumption, and is considered inclusive of "unaccounted for" water. A current population data as gathered by the Ministry of Health in the Region, the population as of 2013 is approximately 19,000 people. With the current demand as loaded in the model a total of 3800 kL/d consumption as allocated to nodes, this is approximately 200L/d/capita consumption.

These data are based on verified checks, and thus regarded more reliable than those based on various assumptions and projections. Moreover, the peak capacity (current and/or after a potential upgrade) of the water plant constitutes an evident physical limit to water supply. Therefore, in this study the current water plant capacity was considered a realistic controlling limit, and was used for scenario modelling purposes. It should be noted that the used hydraulic model can be easily modified to accommodate different and/or future data.

2. LITERATURE REVIEW

2.1 MUNICIPAL WATER DEMANDS

A municipal water network system has two main functions. Firstly, is to supply consumer demand, which represents the flow required to meet daily supply to homes, businesses, institutions and municipal services; and the second is to maintain adequate and reliable supply for fire protection (Senyondo, 2009). In addition, these main functions should be satisfied economically while satisfying prescribed water quality requirements. One of the most pressing concerns in civil engineering is the integrity and reliability of the nation's infrastructure. Of principal concern is the condition of water distribution networks. In the last few years, it has become clear that many of our existing water distribution systems are going to be upgraded and modified if utilities are to continue to provide reliable systems for distributing water to people in urban and rural areas. Good engineering decisions based on sound analysis procedures will be required if the alterations and improvements to these systems are to be effective and economical.

During the last few decades several algorithms have been proposed for solving the basic hydraulic network equations. A general review and comparison of the various techniques has been provided by (Ormsby and Wood, 1986).

2.1.1 Pressure systems

Pressure piping network analysis includes well pumping systems, heating and cooling systems. This report deals principally with the topic of pressure piping as it relates to drinkable water distribution systems.

The main purpose of a water distribution system is to meet demands for potable water. People use water for drinking, cleaning, gardening, and any number of other uses, and this water needs to be delivered in some fashion. A secondary purpose of many distribution systems is to provide water for fire protection.

If designed correctly, the network of interconnected pipes, storage tanks, pumps and regulating valves provides adequate pressure, adequate supply and good water quality fire protection and even present health risks.

2.1.2 Water demands

Just as storm sewer analysis is driven by the watershed runoff flow rate, water distribution system analysis is driven by customer demand. Water usage rates and patterns vary greatly from system to system and are highly dependent on climate, culture and local industry.

Every system is different, so the best source of information for estimating demands is directly recorded system data.

2.1.3 Metered demand

Metered demands are often a modeller's best tool, and can be used to calculate average demands, minimum demands, peak demands, and so forth. These data also can be complied into daily, weekly, monthly and annual reports that show how the demands are influenced by weather, special events and other factors.

Unfortunately, many systems still do not have adequate, let alone complete system metering. For these systems, the modeller is often forced to use other estimation tools (including good engineering judgement) to obtain realistic demands.

2.1.4 Demand patterns

A demand pattern in this context is a function relating water use to time of day. Patterns allow the user to apply automatic time-variable changes within the system. Different categories of users, such as residential or industrial customers, will typically be assigned different patterns to better reflect their particular variations. A diurnal curve is a type of pattern that describes changes in demand over the course of a daily cycle, reflecting times when people are using more or less than average water. Most patterns are based on a multiplication factor versus time relationship, whereby a multiplication factor of 1.0 represents the base value (often and conveniently the average value). In equation form this relationship is written as

$$Q_t = A_t \cdot Q_{base} \qquad (1)$$

With Q_t demand at time t, A_t multiplier for t and Q_{base} baseline demand.

In a representative diurnal curve for a residence as shown in the figure below, there is a peak in the diurnal curve in the morning as people take showers and prepare breakfast, another slight peak around noon, and a third peak in the evening as people arrive home from work and prepare dinner. Throughout the night, the pattern reflects the relative inactivity of the system, with low flows compared to the average. Of course, this curve is conceptual and should not be construed as representative of any particular network.

There are basic forms for representing a pattern: stepwise and continuous. A stepwise pattern is one that assumes a constant level of usage over a period of time, and then jumps instantaneously to another level where it again remains steady until the next jump. A continuous pattern is one for which several points in the pattern are known and sections in between are transitional, resulting in a smoother pattern. Notice that, for the continuous pattern in Figure 1, the magnitude and slope of the pattern at the start and ends times are the same- a continuity that is recommended for patterns that repeat.

Because of the finite time steps used in calculations, most computer programs convert continuous patterns for use by the algorithms, with the duration of each step equal to the time step of the analysis.

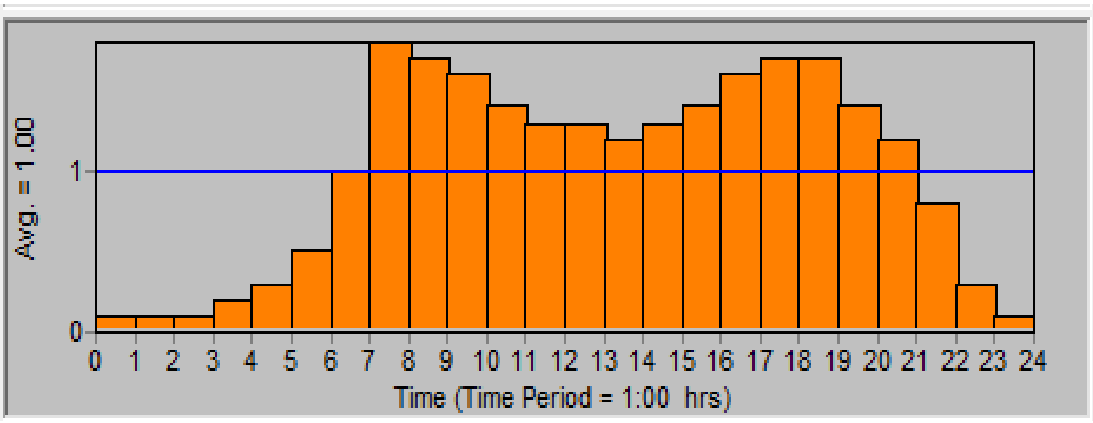

Figure 3 - Typical Diurnal Curve

2.2 WATER DISTRIBUTION SYSTEMS

A water supply distribution system comprises of a complex network of interconnected pipes, service reservoirs and pumps which supplies water from treatment plant to the consumers. The distribution system supplies water to customers via individual service connections, and water for firefighting via hydrants (Anderson, 1990). Water distribution systems are designed to meet the drawoff demands placed on them in terms of satisfactory flow and adequate pressure at all times. The system should also be such that it can be economically operated and maintained. The available pressure should not be excessive as the maintenance of pressure head is an important cost factor, i.e. high pressure is expensive to maintain, and also increases leakage rates. These water demands can be a combination of domestic, commercial, industrial and fire-fighting purposes. Water demand is highly variable by day and by season, whereas the supply source by contrast is fairly constant. Because of this wide variability in demand, storage reservoirs are an integral part of the distribution system. After the water leaves the intake reservoir, the main elements of the system will comprise of the treatment works, trunk mains, storage facilities, pumping stations, pipe reticulation, fire hydrants, house service connections, meters and other appurtenance.

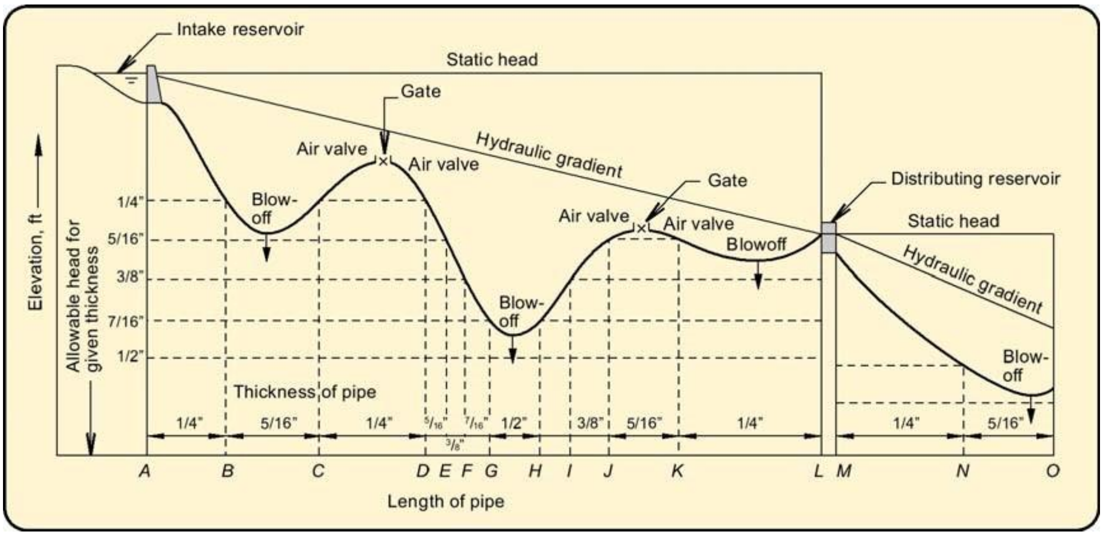

Figure 4: Hydraulic profile ***(Source: USQ Public Health Engineering Study Book)***

The distribution system comprises:

- **Headworks** which are defined as the works for source development and for water treatment, and
- **Distribution works** which are as the works downstream of the treatment facilities for distribution to and within the area served.

The various components of the water supply systems are discussed below. A typical simple system is shown in figure 5.

- **Headworks** include dams, weirs, river intake, bores, associated pumping stations, trunkmains, tunnels and water treatment plants which may be required to upgrade the source water quality to an acceptable standard. Sources most commonly used are surface water and/or groundwater.
- **Distribution works** include service reservoirs and associated pumping stations, trunk mains and reticulation.
- **Trunk mains** to supply the water to reservoirs and to gravitate water from the reservoirs to the major demand areas. Trunk mains are relatively large diameter pipelines which deliver water from the source to the area to be served. Rising mains transfer water solely by pumping, gravity mains by gravity flow and boosted mains can transfer water by either gravity or pumping.
- **Reticulation mains** to distribute the water from the trunk mains to the consumers. Most reticulation mains would be 100 mm in diameter.

- **Terminal storages** are storages at the end of a long trunk main, usually capable of holding substantial (a day or more) supply volume.
- **Balance tanks** are relatively small structures which usually hold up to two hours' supply, and which are used to provide storage to balance differences between inflow (generally a pump delivery) and outflow.
- **Break-pressure tanks** are small generally above ground structures used to limit the head on the system downstream of the tank to the water level in the tank. They may also function as balance tanks.
- **Pumping stations,** which house pumps in various numbers and configurations, are located strategically in the system for pumping or boosting the flow in pipelines.
- **Service reservoirs** are structures, which are generally located above ground and close to or within the reticulated area, and which provide storage to balance demand fluctuations, limit variations in reticulation pressure, and provide a reserve for firefighting and breakdown of other components. A service reservoir typically holds between 25%-100% of the peak day's supply.
- **Reticulation** network comprise relatively small diameter water mains which distribute water from trunk mains to consumer and provide water for firefighting purposes. Each consumer is supplied from the reticulation system by a **service connection** which usually comprised of a small diameter pipe from the town water main to a stop-cock and meter located immediately inside the property boundary, and which connects with the consumer's pipework.
- **Dual systems** supply two grades of water through separate pipe networks. High quality water is provided for drinking, culinary and other domestic purposes, and a lower quality water is provided for external applications such as garden watering.

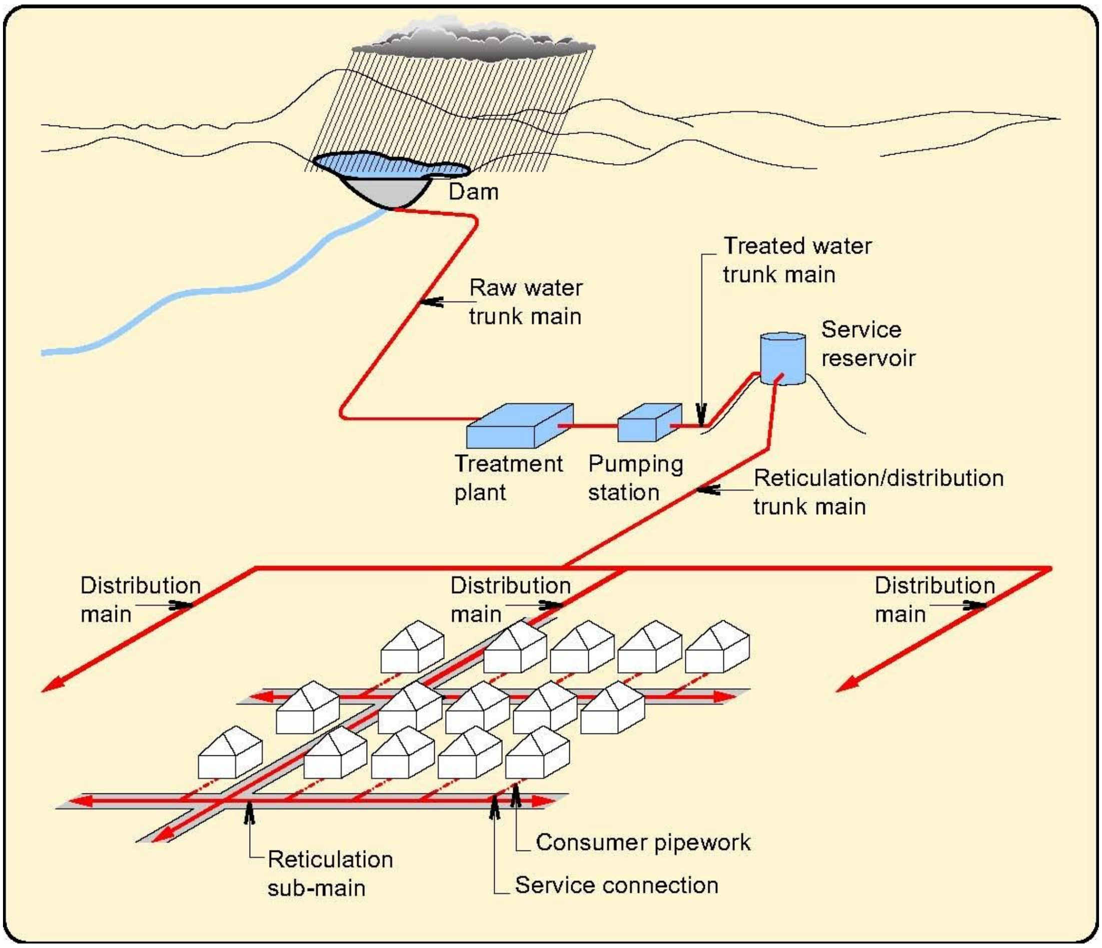

Figure 5 - Typical water supply system *(Source: USQ Public Health Engineering Study Book)*

Water demand is highly variable, both by day by day and by season. By contrast, supply tends to be more constant over time (Chadwick, 2013). Consequently, the distribution system must include storage elements and must be capable of flexible operation. Water pressures within the system are normally retained between a practical maximum (about 70 m head) and minimum (typically 20 m head) value. These limits try to ensure that consumer head is met and that undue leakage due to excessive pressure does not occur. The topography of the demand area plays an important part in the design of the distribution system, particularly if there are large variations of ground levels. In such cases, several independent networks may be required to keep within pressure limitations. For greater operational flexibility, however, they are usually interconnected through booster pumps or pressure reducing valves. Water distribution network analysis provides the basis for the design of new system and the extension of existing systems. Design criteria are that specified minimum flow rates and pressure heads must be attained at the outflow points of the network. The flow and pressure distributions across a network are affected by the arrangement and size of the pipes and the distribution of the outflows. Since a change of diameter in one pipe length will affect the flow and pressure distribution anywhere, network design is not an explicit process. Optimal design methods almost invariably incorporate the hydraulic analysis of the system in which the pipe diameter is systemically altered (Marriot, 2009).

In addition to new distribution systems, a common need is for improvement to existing (and inevitably ageing) systems. It is good practice to use a ring main system preference to a branching system. The implied redundancy prevents the occurrences of "dead ends" with the consequent risk of stagnant water, and permits more flexible operation, particularly when repairs must be carried out. Many existing systems have high leakage rates (30%-50%). Leaks are often difficult to detect and costly to eliminate, yet have an important bearing on the accurateness of any hydraulic analysis of the system.

As essential prerequisite to the improvement of an existing system is to have clear understanding of how that system operates. This is often quite difficult to achieve. Plans of the pipes network, together with elevations, diameters, water levels, etc., are required. In addition, the demands must be estimated on a per capita consumption basis or preferably by simultaneous field measurements of pressure and flows at key points in the system.

The analysis of such systems is normally carried out by computer simulation, in which a numerical model of the system is initially calibrated to the field data before being used in predictive mode. The model is a simplified version of the real system, and in particular, demands from the system are assumed to be concentrated at pipe ends or junctions. This allows relatively simple models to be used without great loss of accuracy, providing that a judicious use of pipe junctions is made.

Such models have been successfully used to locate areas of leakage within networks. However, if leakage rates are high and of unknown location, then a computer simulation may give misleading results and therefore be of little value.

2.2.1 Sources of potable water

Untreated water (also called raw water) may come from groundwater source or surface such as lakes, reservoirs and rivers. The raw water is usually transported to a water treatment plant, where it is processed into treated water (also known as potable, product, or finished water). The required degree of treatment mainly depends on raw water characteristics and relevant drinking water standards, and also impacted by the characteristics of the distribution system.

Before leaving the plant and entering the water distribution system, treated surface water usually enters a unit called a clearwell. The clearwell serves three main purposes in water treatment. First, it provides contact time for disinfectants such as chlorine that are added near the end of the treatment process. Adequate contact time is required to achieve acceptable levels of disinfection.

Second, the clearwell provides storage that acts as a buffer between the treatment plant and the distribution system. Distribution systems naturally fluctuate between periods of high and low water usage, thus the clearwell stores excess treated water during periods of low demand and delivers it during periods of peak demand. Not only does this storage make it possible for the treatment plant to operate at a more stable rate, but it also means that the plant does not need to be designed to handle peak demands. Rather, it can be built to handle more moderate treatment rates, which means lower construction and operational costs.

Third, the clearwell can serve as a source for cleaning plant filters that, when needed, is used at a high rate for a short period of time.

In the case of groundwater, it is true that many sources offer up consistently high quality water that could be consumed without disinfection. However, the practice of maintaining a disinfectant residual is almost always adhered to for protection against accidental contamination and microbial regrowth in the distribution system. Disinfection at groundwater sources influenced by surface water in that it is usually applied at the well itself.

2.2.2 Customers of potable water

Customers of a water supply system are easily identified since they are the reason that the system exists in the first place. Homeowners, factories, hospitals, restaurants, golf courses and thousands of types of customers depend on water systems to provide everything from safe drinking water to irrigation. As demonstrated throughout the book, customers and the nature in which they use water is the driving mechanism behind how a water distribution system behaves. Water use can vary over time both in the long-term (seasonally) and the short-term (daily), and over space. Good knowledge of how water use is distributed across the system is critical to accurate modelling.

2.2.3 Transport facilities

Moving water from the source to the customer requires a network of pipes, pumps, valves and other appurtenances. Storing water to accommodate fluctuations in demand due to varying rates of usage or fire protection needs requires storage facilities such as tanks and reservoirs. Piping, storage and the supporting infrastructure are together referred to as the water distribution system (WDS).

2.2.4 Transmission and distribution mains

This system of piping is often categorised into transmission/trunk mains and distribution mains. Transmission mains consist of components that are designed to convey large amounts of water over great distances, typically between major facilitates within the system. For, example, a transmission main may be used to transport water from a treatment facility to storage tanks throughout several cities and towns. Individual customers are usually not served from transmission mains.

Distribution mains are an intermediate step toward delivering water to the end customers. Distribution mains are smaller in diameter than transmission mains, and typically follow the general topography and alignment of the city streets. They are often laid in an allocated section of the footway on one side of the street. Mains on either of the street may also be economical where most of the lots are developed with houses or where streets are wide, thus minimising disturbance of sealed pavements. Elbows, tees, wyes, crosses and numerous other fitting are used to connect and redirect sections of pipe. Fire hydrants, isolation valves, control valves, blow-offs and other maintenance and operational appurtenances are frequently connected directly to the distribution mains. Services, also called services lines, transmit water from the distribution mains to the end customers.

Homes, business and industries have their own internal plumbing systems to transport water to sinks, washing machines, hose bibbs and so forth. Typically, the internal plumbing of a customer is not included in a WDS model.

2.3 CONTROL VALVES

There are numerous types of valves that may be used in a water distribution network system. Valves are installed in pipelines to control flow by imposing high head losses through the valves (Hwang, 1981).These valves performs differently and have different applications, but all valves are used to automatically control parts of the system, opening, closing, or throttling to achieve the anticipated result.

2.3.1 Check valves (CVs)

Check valves are used to maintain flow in one direction only by closing when the flow begins to reverse. When the flow is in the same direction as the specified direction of the check valve, the valve is considered to be fully open.

2.3.2 Flow control valves (FCVs)

A flow control valve limits the flow rate through the valve to a specified value in a specified direction. A flow rate is used to control the operation of a flow control valve. These valves are commonly found in areas where a water district has contracted with another district, or a private developer to limit the maximum demand to a value that will not adversely affect the provider's system.

2.3.3 Pressure reducing valves (PRVs)

Pressure reducing valves are often used to separate pressure zones in water distribution networks. These valves prevent the pressure downstream from exceeding specified level, in order to avoid pressure and flows that could otherwise have undesirable effects on the system. A pressure or a hydraulic grade is used to control the operation of a PRV.

2.3.4 Pressure sustaining valves (PSVs)

Pressure sustaining valves maintain a specified pressure upstream of the valve. Similar to the other regulating valves, PSVs are often used to ensure that pressure in the system (upstream, in this case) will not drop to unacceptable levels. A pressure or hydraulic grade is used to control the operation of a pressure sustaining valve.

2.3.5 Pressure breaker valves (PBVs)

Pressure breaker valves create a specified head loss across the valve and are often used to model components that cannot be easily modelled using standard minor loss elements.

2.3.6 Throttle control valves (TCVs)

Throttle control valves simulate minor loss elements whose head loss characteristics change over time. With a throttle control valve, the minor loss K is adjusted based on some other system flow or head.

2.3.7 Air valves

Air valves are used to release or admit air to pipelines at high points or other critical locations. Valves with large orifices are used to permit exit or ingress of large volumes of air, while small orifice valves are used to permit the exit of small quantities of air which may accumulate in the main during normal working conditions.

Double Air Valves (combining large and small orifices) release air from a main when filling, admit air when a pipe is emptying, and release small amounts of dissolved air which may accumulate.

Air valves are rarely required in a reticulation network since any dissolved air can escape through service connections. When reticulation is being filled or emptied, air can usually be satisfactorily released through fire hydrants.

2.3.8 Other valves

Scour Valves, located at low points, are used for dewatering pipelines. Gate valves are normally used for this purpose. Scour valves are also installed at service reservoirs to enable them to be drained for cleaning and other maintenance purposes.

Energy Dissipating Valves, such as fixed-cone dispersion valves, are designed to dissipate the energy of a high pressure supply safely in either closed circuits or open discharge.

Surge Relief Valves reduce pressure in a pipeline by discharging water when the pressure rises to a pre-set level and are used to reduce waterhammer effects or to safeguard the supply system in the case of an accidental increase in pressure.

2.4 COMPONENTS OF HYDRAULIC MODELS

Water distribution modelling is the latest technology in a process of advancement that began two millennia ago when the Minoans constructed the first piped water conveyance system (Walski, 2001). Today, it is critical part of designing and operating water distribution system that are capable of serving communities reliably and safely both now and in the future. The availability of increasingly sophisticated and accessible models allows these goals to be realised more fully than ever before.

Hydraulic modelling of a water distribution system has proven to be an effective and reliable approach to assess and predict the performance of an existing or proposed system under a wide range of hydraulic conditions. Such hydraulic modelling is most easily accomplished by computer software to determine pressure and flow distributions through network.

A hydraulic model of a water distribution system is represented as a collection of links connected to nodes. The links represent pipes, pumps, and control valves and the nodes represent junctions, tanks, and reservoirs. Figure 7, below illustrates an example system of nodes and links that connect them to form a network.

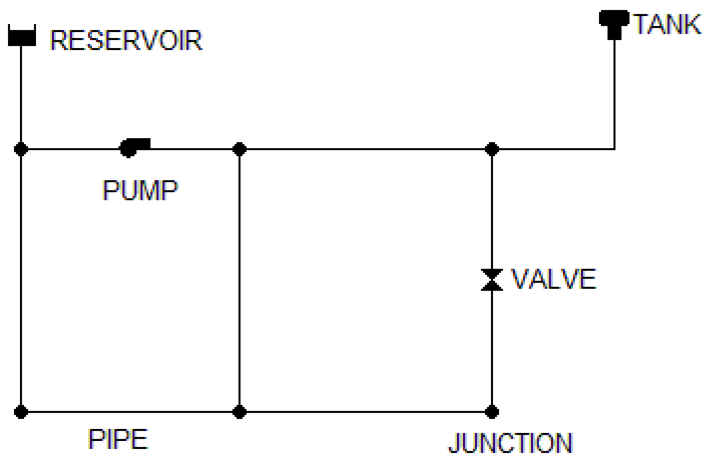

Figure 6 - Physical components of a water distribution system

Each node can contain information about its elevation, water demand, and initial water quality. A successfully run simulation can compute hydraulic head, water pressure, and water quality for each note at various times. The solution for heads and flows at a particular point in time involves simultaneously solving the conservation of flow equation for each junction and the head loss relationship across each link in the network. This process, known as “hydraulic balancing” the network, requires using an iterative technique to solve the nonlinear equation involved, (Rossman, 2000).

2.4.1 Junctions

Junctions (also termed nodes) are points in the network where links join together and where water usually enters or leaves the network. The basic input data required for junctions are:

- elevation above some datum/reference (usually mean sea level)
- water demand (rate of withdrawal from the network)
- initial water quality (if quality is also modelled).

The output results computed for junctions at all time periods of a simulation are:

- hydraulic head (internal energy per unit weight of fluid)
- pressure
- water quality.

Junctions can also:

- have their demand vary with time
- have multiple categories of demands assigned to them
- have negative demands indicating that water is entering the network

- be water quality sources where constituents enter the network
- contain emitters (or sprinklers) which make the outflow rate depend on the pressure.

2.4.2 Reservoirs

Reservoirs are nodes that represent an infinite external source or sink of water to the network. They are used to model such sources as lakes, rivers, groundwater aquifers, and tie-ins to other systems. Reservoirs can also serve as water quality source points.

The primary input properties for a reservoir are its hydraulic head (equal to the water surface elevation if the reservoir is under atmospheric pressure) and its initial quality for water quality analysis.

Since reservoirs are boundary infinite points to a network, their head and water quality cannot be affected by what happens within the network. Therefore they have no computed output properties. However, their heads can be made to vary with time by assigning time patterns to them.

2.4.3 Tanks

Tanks are nodes with storage capacity, where the volume of stored water can vary with time during a simulation. The primary input properties for tanks are:

- bottom elevation (where water level is zero)
- diameter (or shape if non-cylindrical)
- initial, minimum, and maximum water levels
- initial water quality.

The principal outputs computed over time are:

- hydraulic head (water surface elevation)
- water quality.

Tanks are required to operate within their set minimum and maximum levels. EPANET stops outflow if a tank is at its minimum level and stops inflow if it is at its maximum level. Tanks can also serve as water quality source points.

2.4.4 Emitters

Emitters are devices associated with junctions that model the flow through a nozzle or orifice that discharges to the atmosphere. The flow rate through the emitter varies as a function of the pressure available at the node

$$q = C \cdot p^{\gamma} \qquad (2)$$

with q flow rate, p pressure, C discharge coefficient, and γ pressure exponent.

For nozzles and sprinkler heads γ equals 0.5 and the manufacturer usually provides the value of the discharge coefficient in units of gpm/psi$^{0.5}$ (stated as the flow through the device at a 1 psi pressure drop).

Emitters are used to model flow through sprinkler systems and irrigation networks. They can also be used to simulate leakage in a pipe connected to the junction (if a discharge coefficient and pressure exponent for the leaking crack or joint can be estimated) or compute a fire flow at the junction (the flow available at some minimum residual pressure). In the latter case one would use a very high value of the discharge coefficient (e.g., 100 times the maximum flow expected) and modify the junction's elevation to include the equivalent head of the pressure target. EPANET treats emitters as a property of a junction and not as a separate network component.

2.4.5 Pipes

Pipes are links that deliver water from one point in the network to another. EPANET assumes that all pipes are full at all times. Flow direction is from the end at higher hydraulic head (internal energy per weight of water) to that at lower head. The principal hydraulic input parameters for pipes are:

- start and end nodes
- diameter
- length
- roughness coefficient (for determining headloss)
- status (open, closed, or contains a check valve).

The status parameter allows pipes to implicitly contain shutoff (gate) valves and check (non-return) valves (which allow flow in only one direction).

The water quality inputs for pipes consist of:

- bulk reaction coefficient
- wall reaction coefficient.

Computed outputs for pipes include:

- flow rate
- velocity
- headloss
- Darcy-Weisbach friction factor
- average reaction rate (over the pipe length)
- average water quality (over the pipe length).

The hydraulic head lost by water flowing in a pipe due to friction with the pipe walls can be computed using one of three different formulas:

- Darcy-Weisbach formula

$$h_f = \frac{f \cdot L \cdot v^2}{2 \cdot g \cdot D} \qquad (3)$$

with h_f friction loss, f friction factor, L pipe length, v mean velocity; g constant of gravity acceleration, and D pipe diameter.

- Hazen-Williams formula

$$v = 0.849 \cdot C_{HW} \cdot R^{0.63} \cdot S_f^{0.54} \qquad (4)$$

with v mean velocity, C_{HW} roughness coefficient, R hydraulic radius and S_f slope of the energy line.

- Manning formula

$$v = \frac{R^{2/3} \cdot S_f^{1/2}}{n} \qquad (5)$$

with n roughness factor.

The Hazen-Williams formula is the most commonly used headloss formula in the US. It cannot be used for liquids other than water and was originally developed for turbulent flow only. The Darcy-Weisbach formula is the most preferred, especially for use in detailed analyses that require higher accuracies.. It applies over all flow regimes and to all liquids. The Manning formula is more often used for open channel flow but also useful in case of pipes with high relative roughness.

Each of the above formula uses the following equation to compute the headloss between the start and end node of the pipe

$$h_L = A \cdot q^B \qquad (6)$$

with h_L headloss over L length, q flow rate, A resistance coefficient, and B flow exponent.
With the Darcy-Weisbach formula EPANET uses different methods to compute the f friction factor, depending on the flow regime:

- The Hagen–Poiseuille formula is used for laminar flow (Re < 2,000).
- The Swamee and Jain approximation to the Colebrook-White equation is used for turbulent flow (Re > 4,000).
- A cubic interpolation formula for transitional flow (2,000 < Re < 4,000).

2.4.6 Energy equation

Conservation of energy is important for pipe flow. Total energy of the flow is given by the sum of the pressure head, velocity head and elevation

$$E = \frac{p}{\rho \cdot g} + \frac{\alpha \cdot v^2}{2 \cdot g} + Z \tag{7}$$

The well-known energy (Bernoulli) equation between two points can be written as:

$$\frac{p_1}{\rho \cdot g} + \frac{\alpha \cdot v_1^{\,2}}{2 \cdot g} + Z_1 + h_p - h_f - h_m = \frac{p_2}{\rho \cdot g} + \frac{\alpha \cdot v_2^{\,2}}{2 \cdot g} + Z_2 \tag{8}$$

with h_p pump head, h_f friction and h_m minor head losses.

2.4.7 Pumps

Pumps are links that impart energy to a fluid thereby raising its hydraulic head. The principal input parameters for a pump are its start and end nodes and its pump curve (the combination of heads and flows that the pump can produce). In lieu of a pump curve, the pump could be represented as a constant energy device, one that supplies a constant amount of energy (horsepower or kilowatts) to the fluid for all combinations of flow and head.

The principal output parameters are flow and head gain. Flow through a pump is unidirectional and EPANET will not allow a pump to operate outside the range of its pump curve.

Variable speed pumps can also be considered by specifying that their speed setting be changed under these same types of conditions. By definition, the original pump curve supplied to the program has a relative speed setting of 1. If the pump speed doubles, then the relative setting would be 2; if run at half speed, the relative setting is 0.5 and so on. Changing the pump speed shifts the position and shape of the pump curve (see the section on Pump Curves below).

As with pipes, pumps can be turned on and off at pre-set times or when certain conditions exist in the network. A pump’s operation can also be described by assigning it a time pattern of relative speed settings. EPANET can also compute the energy consumption and cost of a pump. Each pump can be assigned an efficiency curve and schedule of energy prices. If these are not supplied then a set of global energy options will be used.

Flow through a pump is unidirectional. If system conditions require more head than the pump can produce, EPANET shuts the pump off. If more than the maximum flow is required, EPANET extrapolates the pump curve to the required flow, even if this produces a negative head. In both cases a warning message will be issued.

The figure below shows a typical pump curve.

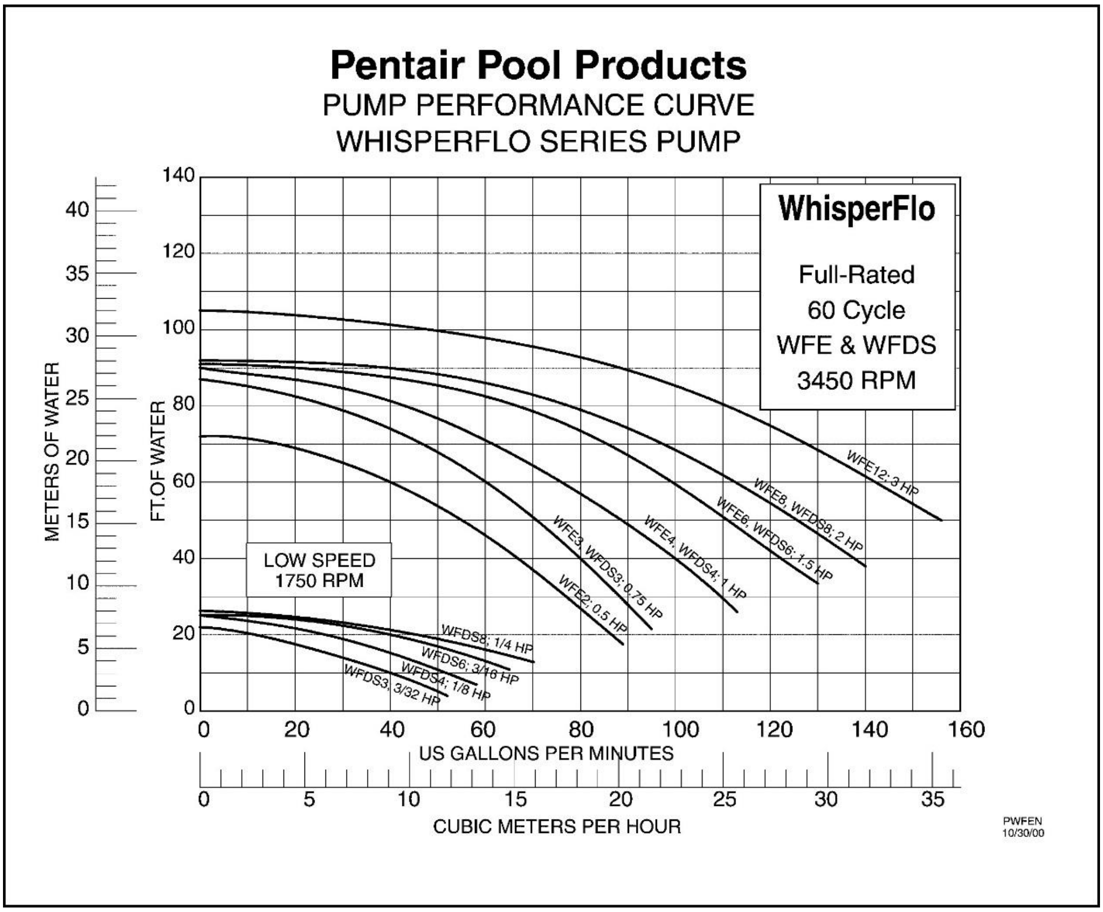

Figure 7 - Typical pump curve ***(source: Pentair, Switzerland)***

2.4.8 Water quality modelling

Hydraulic modelling software tracks the fate of discrete parcels of water as they move along pipes and mix together at junctions between fixed-length time steps. It calculates the concentration and size for each of a series of non-overlapping segments of water filling each pipe of the network. Water quality can be analysed as a fate and concentration of a reactive chemical, or in terms of water age, which refers to average length of time that water at a specific location has been circulating within the pipeline. For each water quality time step, the contents of each water segment are subjected to reaction, and a cumulative account is kept of the total mass and flow volume entering each node. The new positions of the segments are then updated (Rossman, 2000). Water at nodes can have a slightly higher age than water along the pipes they are a part of if they are dead ends because they represent the age at the end of the pipe rather than the average along a pipe length. Correspondingly, water within pipes can have higher age than water at nodes if water flow is continually reversing direction within the pipe, causing water near the middle of the pipe to be the oldest.

EPANET is also capable to mode the temporal and spatial distribution of conservative and non-conservative compounds in water. This feature is most often used to study the decay of chlorine residual concentrations in the network.

2.5 HYDRAULIC SIMULATION SOFTWARE

Researchers and engineers use hydraulic simulation software to better understand properties of a water system and to investigate alternatives without the need for rigorous hand calculations for each alternative. In the 1990's, the EPA's Water Supply and Water Resources Division developed a software program to perform extended-period simulation of the hydraulic and water quality behaviour within pressurised pipe networks (Lea, 2009). This software, EPANET2, is free of charge and available for download at the EPA website, http://www.epa.gov/nrmrl/wswrd/dw/epanet.html.

EPANET is designed to be a research tool for improving our understanding of the movement and fate of drinking water constituents within distribution systems. It contains a state-of-the-art hydraulic analysis engine that includes the following capabilities: (Rossman, 2000).

- computes friction head loss using Hazen–Williams, Darcy-Weisbach or Chezy-Manning formula
- places no limit on the size of the network that can be analysed
- includes minor head losses for bends, fittings, etc
- models various types of valves including shutoff, check, pressure regulating, and flow control valves
- models constant or variable speed pumps
- computes pumping energy and cost
- allows storage tanks to have any shape (i.e. diameter can vary with height)
- consider multiple demands categories at nodes, each with its own pattern of time variation
- models pressure-dependent flow issuing from emitters (sprinkler heads)
- can perform system operation on both simple tank level and timer controls and on complex rule-based controls.

EPANET's Windows user interface provides a network editor that simplifies the process of building piping network models and editing their properties. Various data reporting and visualisation tools such as graphical views, tabular views, and special reports, and calibration are used to assist in interpreting the results of the network analysis (EPA, 2000).

2.6 FOREIGN WATER UTILITIES

This section briefly discusses the characteristics, operational philosophies and experience of several selected public utilities and systems, since these can be partly applicable to Fijian conditions.

2.6.1 United Kingdom

Significant changes have occurred in the water utilities of the United Kingdom during the past decades. Due to the changes, new technologies have been developed and implemented in the management of the distribution systems. Their 10 water companies provide water services to 37.9 million people and are regulated by the director general of the Office of Water Services (OFWAT). The regulation of these companies has focussed attention on the need for higher levels of customer service and satisfaction. The drinking water inspectorate conducts technical audits of water service companies to verify that the companies are fulfilling these obligations with regard to sufficiency and quality of water supplies. The regulatory regimes of water companies focus on

- Level of service, reliability, performance, and quality
- Customer care
- Operating and capital costs
- Positive innovation and efficiency

Water companies are required to develop asset management plans (AMPs) based on medium and long-term investment forecasts linked to maintenance and improvement service. These AMPs are used by OFWAT to measure the efficiencies of water companies. Since 1990, the water companies have made progress in developing innovative techniques to operate and maintain distribution systems. The regulations require water companies to annually submit information "level of service" indicators, which include reports on their achieved and projected performance regarding pressure in mains, interruptions to the water supply, water usage restrictions, response to billing queries, and response to customer complaints. Each of these is backed by a "guaranteed standard scheme," which provides for refunds to customers if the water companies' level of services does not meet the requirements.

The level of service indicators by which water distribution system performance measures are evaluated include

- Water pressure in mains
- Interruptions to water supply
- Leakage control
- Asset management plans

Water pressure in mains is used to show the number of residential properties at risk of receiving water pressure lower than the reference level because of deficiencies in the supply and in the distribution system. The reference level of service for water pressure is defined as 10 m (14 psi) water head at the curb stop at a flow of 9 L/min (2.4 gpm). When a company is unable to demonstrate the required pressure head at the curb stop, it is acceptable to have demonstrated 15 m (21 psi) of water head in the adjacent distribution main. If a length of main has pressure below the level of service pressure, then all properties connected to that main are described as receiving below the reference level of service. A detailed method of recording pressure head violations is outlined in the regulations. The yearly report also identifies measures taken to improve water pressure, either by operational improvements such as adjusting valves, properly operating pumps, or changing operating procedure or by capital improvements such as replacement or cleaning and lining of water mains.

The reference “level of service” for interruptions to supply is defined as the unavailability of water for 12 hours or more. The total number of properties experiencing interruptions above the reference level of service are reported and recorded in as much detail as is practical. The records include time and date of incident, properties affected, causes and interruption to supply, and actions taken.

In the UK regulatory environment, water companies are required to prepare a comprehensive analysis of leakage information, which would include the leakage control methods adopted (OFWAT, 1993).

A National Leakage Control Initiative committee was formed to develop improved, up-to-date, and more consistent leakage reporting methodology.

UK water companies are required to develop asset management plans for operational and capital improvements, with the objective of improved performance. These plans include detailed information on the existing conditions of the water mains, water quality, hydraulics and customer complaints, as well as medium-and-long- range plans for capital investment to improve levels of service.

As a result of the emphasis on large capital improvement programs and of new incentives for innovative and efficient operation and maintenance plans, water companies in the United Kingdom have developed sophisticated information collection, analysis, and management methods. These methods include innovative technologies for real-time measurement of water pressure and flows, for assessing the condition of water mains and water quality in distribution systems, for prioritising water main rehabilitation and replacement projects, and for the use of customer complaint data to effectively manage water system.

2.6.2 Germany

Information from the former West Germany indicated that there are over 6,000 water systems, all of which are metered. Percentages of unaccounted-for-water in individual systems vary between 0 and 25 percent (Deb, 1995). In 1986, the concept of specific loss was introduced; it involves calculating distribution losses per kilometre of pipe network for leakage loss estimation. In Germany, the average rate of water loss because of water main breaks is 4.5 kL/h (20 gpm). The philosophy of water systems in Germany is to maintain high water main pressure so that leaks show up on the surface quickly. Buildings in high-pressure areas have been fitted with individual regulators to reduce pressures on the plumbing system.

2.6.3 France

In France a national average of 33 percent of water is lost during transmission and distribution (Deb, 1995). There is no requirement for publishing water supply statistics in a standard format for each water company, as is required in the United Kingdom. A number of performance measures for annual distribution losses are defined by French water companies, but loss per kilometre of distribution system per day seems to be the most prominent measure.

2.6.4 Norway

In Norway there are no national water distribution system performance goals and standards (Deb, 1995). Leakage from distribution system is high-in many cases as much as 50 percent of the total water supply. Individual systems set their own standards. For example, the City of Trondheim (140,000 populations) had established the following performance goals for 1988 to 1992:

- Leakage should be reduced to 33 percent of water supplied.
- No consumers should be without water supply for more than 4 hours.

2.6.5 Shillong, India

Shillong is a city in north-eastern India and capital of Meghalaya state, one of the least populous India state (Ingeduld, 2006). Shillong is about 65 km (40 mi) north of the border with the Bangladesh and about 100 km (60mi) south of the border with Bhutan. The water is being supplied for domestic, small industry and other purpose through Greater Shillong Water Supply Scheme and by Shillong Municipally Board. Water supply and distribution network of Shillong with nearly 346,000 of inhabitants is a complicated large-scale network. The water distribution system of the Shillong is divided into ten supply zones.

The network and its attributes were imported from GIS data base to the MIKE NET with the use of import tool option. The tanks attributes like tank diameter, tank capacity and tank shape were collected from the tank questionnaires. The operation rules were collected from the operator of the water distribution network and then incorporated into the model. The water distribution system is functioned in gravity method. The demand distribution in the model is based on information of number of connections at ferrule points. Several logical discrepancies as well as mistakes were found in the provided GIS data. Large parts of the system are without ferrule points; some ferrule points have zero number of connections. The demand distribution is essential not only for model accuracy but for model functionality. Therefore in some cases 'fictive' ferrules were distributed in the network. The average leakage of 15% has been assumed. The distribution of leakage to nodes using weight of length of connected pipes has been applied. The leakage was applied only during the supply hours; during non- supply hours the leakage stops automatically based on advanced rules system. Thus the leakage distributed in the system must be recalculated by supply coefficient equal to the duration of supply period divided by 24.

The hydraulic model was model was developed to study the low pressure areas and flow pattern in the distribution network. The average leakage of 15% has been assumed. The distribution of leakage to nodes using weight of length of connected pipes has been applied. The leakage was applied only during the supply hours; during non-supply hours the leakage stops automatically based on advanced rules system. Thus the leakage distributed in the system must be recalculated by supply coefficient equal to the duration of supply period/24 (Ingeduld, 2006).

2.6.6 Dhaka, Bangladesh

Dhaka is a capital city of Bangladesh, in Dhaka Division, central Bangladesh which is located on an arm of the Dhaleswari River in the populous and flood-prone Ganges-Brahmaputra delta and is a major commercial, cultural and manufacturing centre.

The water distribution system supplies app. 4.5-6 mil inhabitants, depending on the daily fluctuation when people move into and out of the town. Water demand is approximately 25 kL/s while the available supply is only 17 kL/s. About 85% of water sources are groundwater wells (400) and 15% of water sources are surface water (there are 3 water sources) covering the area of more than 200 km^2. There are almost no elevated storage tanks in the system, the total number of elevated tanks is 52, and only 16 tanks are in operation. 60% of service pipes are equipped by the water meters. The intermittent water supply provides consumers with water twice a day. Water is pumped from ground water wells directly to the pipeline network. The pressure is very low and it ranges around 20-25 mbut it can also be only within 2-5 m; the low pressure can cause the infiltration of ground water into water distribution pipelines. Most residents used their own tanks (household tanks) in order to collect water during the supply hours. The leakage level is estimated as 40-45% with the following distribution: main pipes (55%), service pipes (30%) and illegal demand (15%).

The hydraulic model of the whole water distribution system was developed in MIKE URBAN based on the data collected in ArcGIS, this data included pipes, location of tube-wells and elevation data. Node demands were developed based on the population estimates, leakage and unaccounted for water were also taken into account. The hydraulic model was macro-calibrated based on the measured flows and pressures, collected during the monitoring campaign.

Several hydraulic and scenarios were conducted including a dedicated modifying of the EPANET algorithm allowing for modelling of low pressurized conditions. This proved to be necessary requirements for accurate modelling of flow and pressures in the network.

The main objective in developing the detailed water distribution model was to provide backgrounds for the further systematic network rehabilitation and improvement towards continuous water supply during 24 hours.

Water demand of the system exceeds produced water and this was the reason why the standard "demand-driven" EPANET analysis was replaced by a pressure dependent analysis. The results were satisfactory and enabled the model macro-level calibration and prepare the model for the further use in the network rehabilitation and planning.

3. METHODOLOGY

3.1 INITIAL STEPS IN WATER SUPPLY INVESTIGATION

3.1.1 Background review and data collection

The first step consisted of a background review using a range of sources:

- previous Public Work Department of Fiji now WAF or Water Supply Investigation Reports, Water Supply Planning Reports and files on the area (Tavua/Vatukoula),
- strategic planning reports from Councils or other authorities, e.g. river basin reports for the area from Lands and Mineral Resource Department (LMRD), planning studies of Department of Environment and Planning,
- works in the area carried out by other branches of the WAF, particularly Wastewater Engineering and, where appropriate, works carried out by other government agencies.

Collected data generally included:

- population and dwelling data,
- existing and proposed land use and zonings,
- data on water consumption and water quality,
- streamflow data,
- locations, capacity curves and key levels of existing dams and weirs together with details and levels of draw-offs,
- details of river intakes and pump data,
- existing treatment facilities including capacity and operational problems,
- plans and longitudinal sections of all trunk mains and reticulation showing location, size, material, class, age and condition,
- location and capacities and pumps together with pump curves and condition,
- capacity, top water level and bottom water level of all reservoirs and balance tanks,
- history of development, condition and estimated life of existing system components,
- performance data on the existing system, including areas of insufficient pressure, flow rates, incidence of supply failures or restrictions, data on breakages, leakage or other operational difficulties.

3.1.2 Population and tenement projections

Population and development projections are required over the period of the planning horizon in order to estimate future water demands. Domestic demands are more closely related to the number of dwellings rather than the population served, so demand estimates are usually prepared on a per dwelling basis, utilising population projections together with information on type, amount and location of residential development envisaged and occupancy ratios.

Information were assembled from all available sources on the following matters:

- existing populations and population projections,
- existing number of lots and dwellings including flats and town houses,
- future development areas and expected development densities,
- existing and likely future hospitals, schools or institutions,
- existing and likely future commercial and industrial development.

The Department of Environment and Town and Country Planning (DETCP) and the local Council were consulted to assist in determining projections to be used for each development category.

Projections from the Population Projection Group of the DETCP are on a regional or local government area basis and may be general assistance in confirming other assessments of future populations.

The principal sources of information are:

- Fiji Bureau of Statistics for census information,
- DEPTC (including the Population Projection Group),
- Local Council.

Other organisations involved in population or migration projection from time to time include Telecom, Universities and Colleges, and other government departments e.g. Education and Health (Divisional) and Transport. All of these were contacted for their or perceptions of growth or migration.

The basic procedure for the high resolution all pipes model was as follows:

- Collection of network asset information such as pipework data, pump data, reservoir data, and input network asset information into model;
- Collection of elevation data;
- Loading the model with demand data;
- Review of demand patterns and allocation to model nodes; and
- Running of the model and use of available field data to validate the model performance, including operating understanding following discussions with WAF operations staff.

3.2 MODELLING ASSUMPTIONS AND DEFAULT VALUES

3.2.1 Model demand patterns

Model demand patterns were determined using consumption (meter records), the results of previous studies involving estimated data and limited site measurements used for model calibrations. Temporal patterns used for Residential and Business/Commercial demands and losses, and Real Losses are shown in Figures 8 - 10. It should be noted that the patterns for apparent losses (not shown here) are identical to that for residential and commercial demands. This is because the majority of apparent losses are assumed to be under-metering or lack of registration by residential and commercial customers.

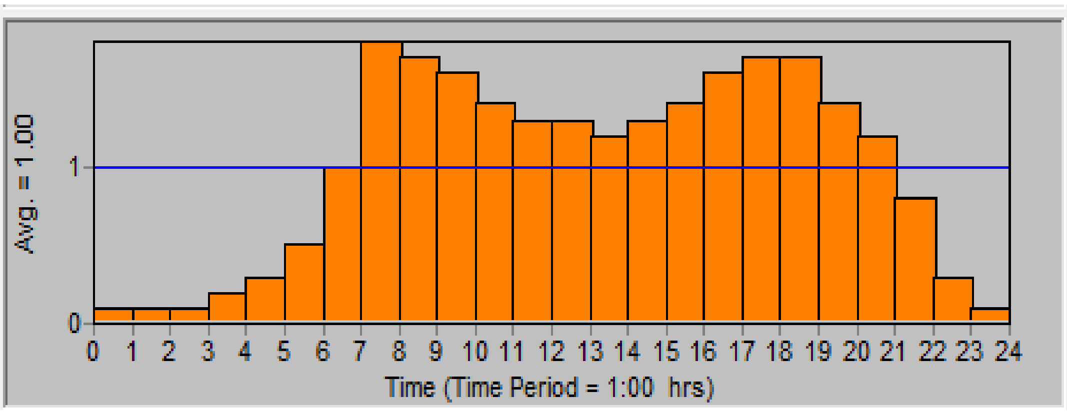

Figure 8 - W1 demand pattern (residential)

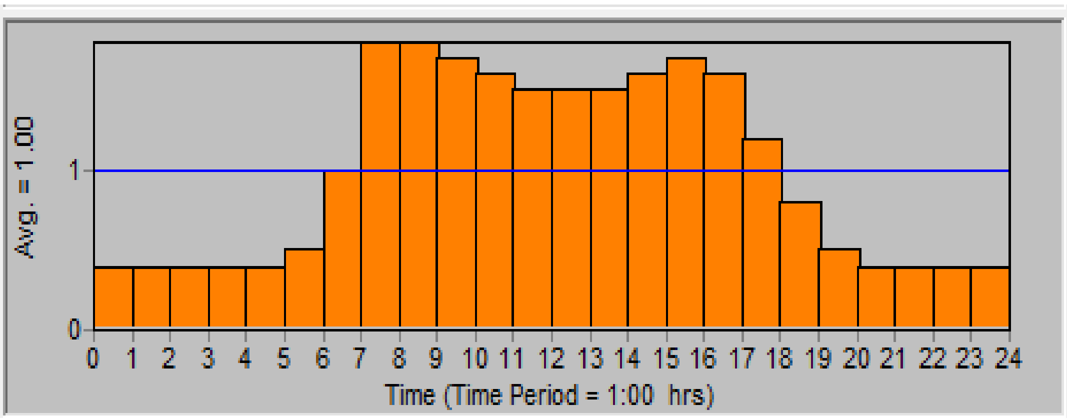

Figure 9 - W2 demand pattern (business/commercial)

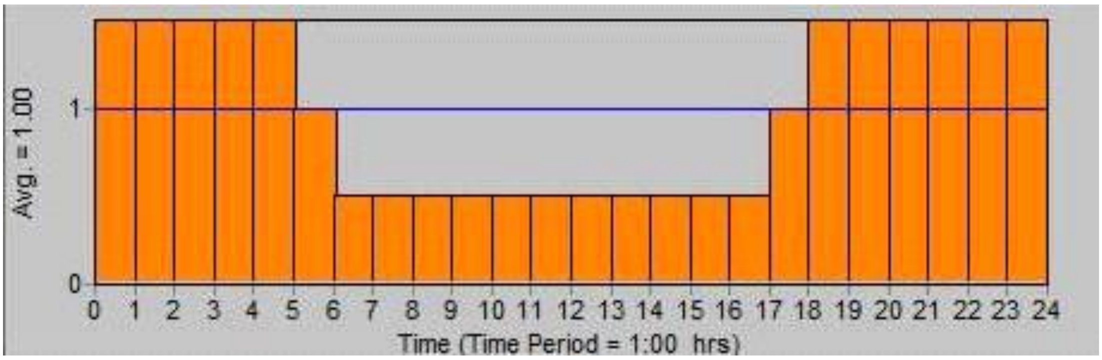

Figure 10 - NRW2 demand pattern (real losses)

Since the headloss is roughly proportional to the square of the velocity (or discharge), the hydraulic effect of the relatively small physical losses (estimated around 15% for this system) is hardly significant, and thus can be neglected safely for common analyses. This argument further can be justified by considering that other parameters, such as the actual pipe internal diameter and roughness values have more significant impact on the accuracy of the results. Moreover, limited calibration measurement involving reservoir levels and pressure data at distinct (end) nodes suggest that the simplified (no physical loss) model provides reasonable results. Would there a justified need arise, it is simple to combine consumption and loss patterns data to the nodes of the system.

3.2.2 Modelling scenarios

Extended simulation over a 24 h period can provide essential information about system conditions in the dry season. In the rainy season many people utilise rainwater, hence the demand falls significantly and pose smaller challenges to supply. Simulation runs include both essential and operational scenarios:

1. Analysis of the average demand in the dry season is used as a typical baseline condition for comparisons.

2. Analysis of the current peak demand (maximum hour) in the dry season reveals the current performance of the system.

3. Analysis of minimum demand conditions (night hours) in the dry season is useful to identify high pressure areas that may cause more frequent pipe bursts and/or increase water losses.

4. Analysis of maximum supply peak hour demand conditions is the ultimate test of system adequacy. It is assumed the water plant is operated at full capacity and the demand is at peak hour. Gaining insight into this condition is especially useful for system upgrade planning purposes.

For operational and planning purposes various scenarios may be of interest:

5. Effect of district isolation on the rest of the system. This may be required due to connecting large customers. pipe bursts, and similar causes. Simulation can predict the extent and severity on supply derogation in affected areas.

6. Similar the to above point, a fire in the town area normally results in the closure of various valves to supply sufficient discharge for fire fighting at increased pressure. The same, however, implies temporarily limited or no supply to other customers.

7. Effect of reservoir isolation on nearby areas. This occurs regularly for cleaning and maintenance purposes (normally in the wet season).

8. Connection of a new large customer (sugar cane processing) at an earmarked area.

9. Water age analysis is useful to estimate water stagnation that derogates water quality.

With regard to the to the scope of this study, the current status of the model (partially calibrated) and page limitations, only the essential and the last two operational/planning scenarios (8 and 9) will be can be presented.

3.2.3 Epanet settings and default parameters

Hydraulics options determine how the hydraulic behaviour of the pipe network should be analysed. The options include:

Flow Units (L/s in this study).

Headloss Formula used (Hazen-Williams in this study, since there is no sufficient pipe roughness data available to use the Darcy-Weisbach equation.

Specific gravity, expressed as ratio of the density of the fluid being modelled to that of water at 4 °C. Using a value of 1.0 versus 0.997 as in this study would causes only a negligible difference in the results.

Relative viscosity is has a very significant effect on head losses and must be considered. For the tropical climate and relatively high ambient temperature od 26 °C, a factor of 0.890 was used.

Maximum trials is 200 by default; meaning the number of trials used to solve the nonlinear equations that govern network hydraulics at a given point in time.

Accuracy is a converge criterion used to signal that a solution has been found to the nonlinear equations that govern network hydraulics. Trials end when the sum of all flow changes divided by the sum of all link flows is less than this number (default value is 0.001)

If Unbalanced is a setting to instruct the solver if a hydraulic solution is not found within the maximum number of trials. Choices are STOP to stop the simulation at this point or CONTINUE to use another 10 trials, with no link status changes allowed, in an attempt to achieve convergence.

Default Pattern is an ID of a time pattern to be applied to demands at those junctions where no time pattern is specified. If no such time pattern exists then demands will not vary at these locations.

Demand Multiplier applies to all baseline demands to make total system consumption vary up or down by a fixed ration, i.e. 2.0 doubles all demands while 0.5 halves them. This parameter should be changed to accommodate various modelling scenarios.

Emitter Exponent is a constant with a default value of 0.5 for “emitters”, such as a hydrant (regarded as orifice), whereby the discharge depends on the actual pressure. This is applicable only for hose connections, since fire trucks use pumps and able to exert suction on the hydrant.

Status Report is a flag that determines the amount of reported simulation information (YES used in this study).

CHECKFREQ sets the number of solution trials that pass during hydraulic balancing before the status of pumps, check valves, flow control valves and pipes connected to tanks are once again updated. The default value is 2, meaning that status checks are made every other trial.

MAXCHECK is the number of solution trials after which periodic status checks on pumps, check valves flow control valves and pipes connected to tanks are discontinued. Instead, a status check is made only after convergence is achieved. The default value is 10, meaning that after 10 trials status is checked only at convergence.

DAMPLIMIT is the accuracy value at which solution damping and status checks on PRVs and PSVs should begin. Damping limits all flow changes to 60% of what they would otherwise be as future trials unfold. The default is 0 which indicates that no damping should be used and that status checks on control valves are made at every iteration. Damping might be needed on networks that have trouble converging, in which case a limit of 0.01 is suggested.

Figure 11 below illustrates a typical setting used in this study.

Hydraulics Options

Property	Value
Flow Units	LPS
Headloss Formula	H-W
Specific Gravity	0.997
Relative Viscosity	0.89
Maximum Trials	50
Accuracy	0.001
If Unbalanced	Stop
Default Pattern	1
Demand Multiplier	1.000000
Emitter Exponent	0.500000
Status Report	Yes
CHECKFREQ	2
MAXCHECK	10
DAMPLIMIT	0.000000

Figure 11 - Model options

]It is possible to export all model in text/ASCII format that allows manual editing (or using Excel etc.). The complete model file used in this study is available in the Appendices, with a slightly edited extract presented below for illustration.

	A	B	C	D	E	F	G
1	No.	Node ID	Elevation	Demand	Pattern	Remark	
2	1	10	39	0		;JUNCTION	
3	2	100	80	0		;New Junction	
4	3	102	80	0		;New Junction	
5	4	104	53	0		;New Junction	
6	5	108	56	0		;New Junction	
7	6	110	10	0.03	W2	;New Junction	
8	7	112	14	0		;New Junction	
9	8	114	13	0		;New Junction	
10	9	116	0	0.09	W2	;New Junction	
11	10	118	14	0.05	W1	;New Junction	
12	11	12	25	0.05	W1	;New Junction	
13	...						
14	583	80	13	0.17	W1	;New Junction	
15	584	88	13	0.27	W1	;New Junction	
16	585	90	0	0		;New Junction	
17	586	92	0	0.06	W1	;New Junction	
18	587	94	0	0.08	W1	;New Junction	
19	588	98	43	0		;New Junction	
20		Total	L/s	43.95	12	W2 pattern	
21			m3/h	158.22	276	W1 pattern	
22			m3/d	3797.28	288	active nodes	
23			per capita	0.199857	588	total nodes	

Figure 12 – Illustrative extract of the model INP file

System summary data reveal that this is a relatively small but complex system.

Number of Junctions: 588

Number of Reservoirs: 1

Number of Tanks: 8

Number of Pipes: 607

Number of Pumps: 6

Number of Valves: 30

Flow Units: LPS

Headloss Formula: H-W

Quality Parameter: Age

There are many interesting features of EPANET and details that could be presented here to fit page limitations set by a dissertation. Even more tempting could be to seek possible remediation options to address various system bottlenecks and problems. However, for such an extensive system any detailed report would require substantial work and efforts from a dedicated team, let alone the need to obtain additional calibration data and other details to make the exercise practical and directly useful. Nevertheless, the typical scenarios analysed (and only partly presented) below can reveal the behavioural characteristics and some limitations of the Tavua/Vatukoula system (which is partly known to operational staff). More importantly, it shows the utility of the model developed in this study, which eventually will be further improved and augmented in the coming years.

4. RESULTS AND DISCUSSION

4.1 BASELINE ANALYSIS: AVERAGE CONSUMPTION CONDITIONS IN THE DRY SEASON

Figure 15 on the next page summarises the pressures at all nodes and mean velocities in pipes through colour coding between 13:00 and 14:00 hours that represents daily average demand conditions. It can be observed that there except of the vicinity of the reservoir, at Vatukoula, and a small part of Balata, sufficient pressure prevails all over the network. The mentioned areas are hilly with relatively high elevations that – for the given level of supply tanks - results in low pressures that are reported as negative values as an artefact of balance calculations in EPANET. In any case, the pertinent pipes are unable to deliver the assigned discharges to supply North Vatukoula and some areas of Balata near the reservoir. It would appear the best remedy for solving this (known) problem might be to use an in-line booster pump. The annotations on the drawing explain that the remaining low pressure nodes cause no real supply problems.

The mean velocities are rather low, nowhere exceeding an arbitrary 2.0 m/s deemed to be the lower limit of 'high values', and only in a few pipes reach the range between 1.0 and 2.0 m/s.

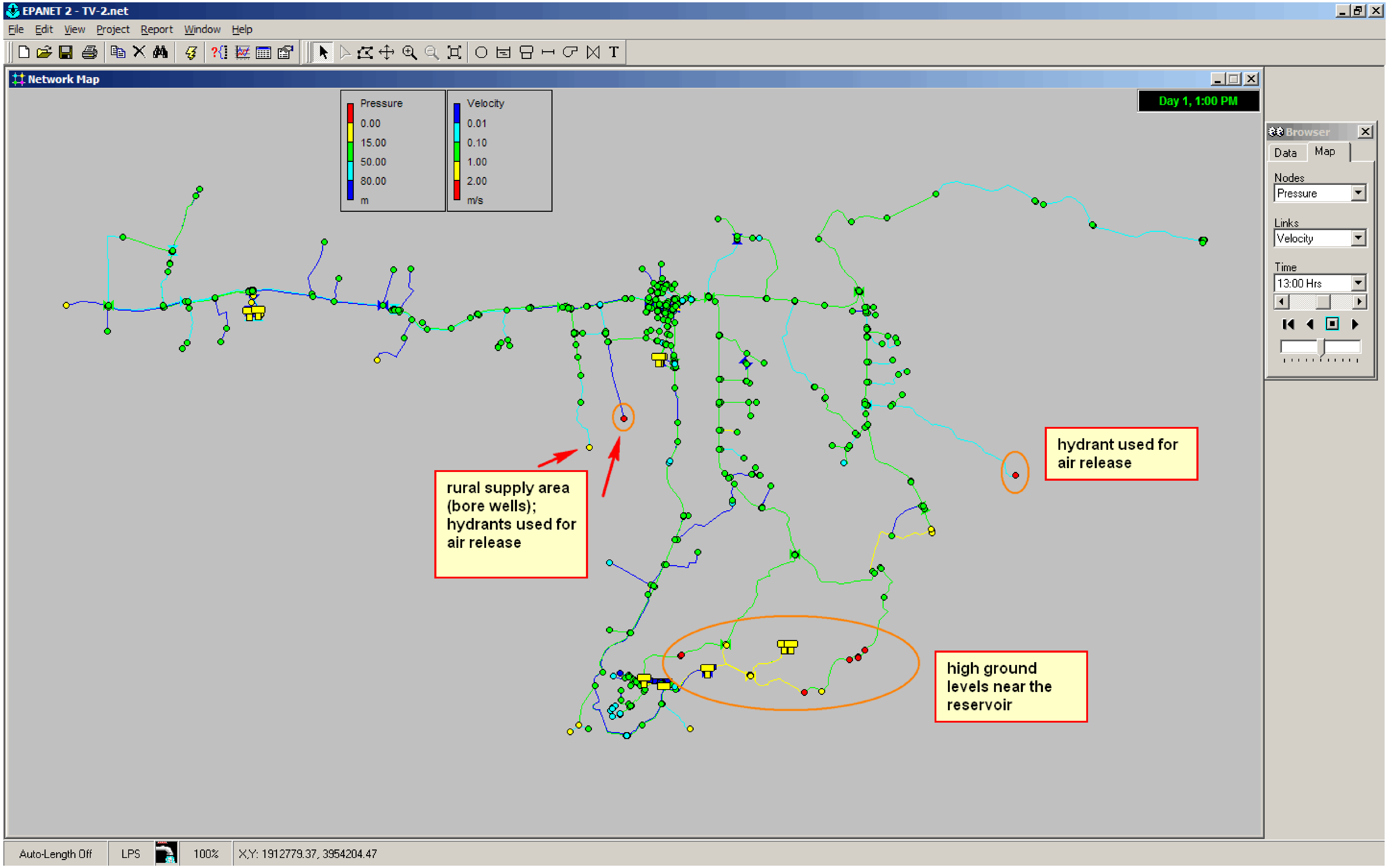

Figure 13 - Tavua/Vatukoula WDN: daily average conditions

4.2 CURRENT PEAK CONSUMPTION (MAXIMUM HOUR) IN THE DRY SEASON

Figure 16 on the next page shows that there are relatively minor changes expected for the peak hour conditions. Yellow coloured nodes (pressure < 15 m) appear in two areas South-East and East of Tavua town, within Tavua proper, and at Korovu in the East. These patterns are in agreement with actual pressure observations. Pressures remain adequate for the gross majority of nodes but it should be clear that remedial actions (system augmentations) will be required in the mid-term to prevent further degradation, and achieve at least adequate supply for consumers located in peripheral districts.

The mean velocities are still typically low, not exceeding 2.0 m/s high limit anywhere. Compared with the previous scenario, the number of pipes having ‘normal’ mean velocities (between 1.0 and 2.0 m/s) is still small. This finding reveals that it would not be efficient to use parallel to improve pressure conditions. In fact, that also would exacerbate existing water stagnation problems. A more logical and effective approach would be to use booster pumps in several locations, perhaps in combination with elevated tanks.

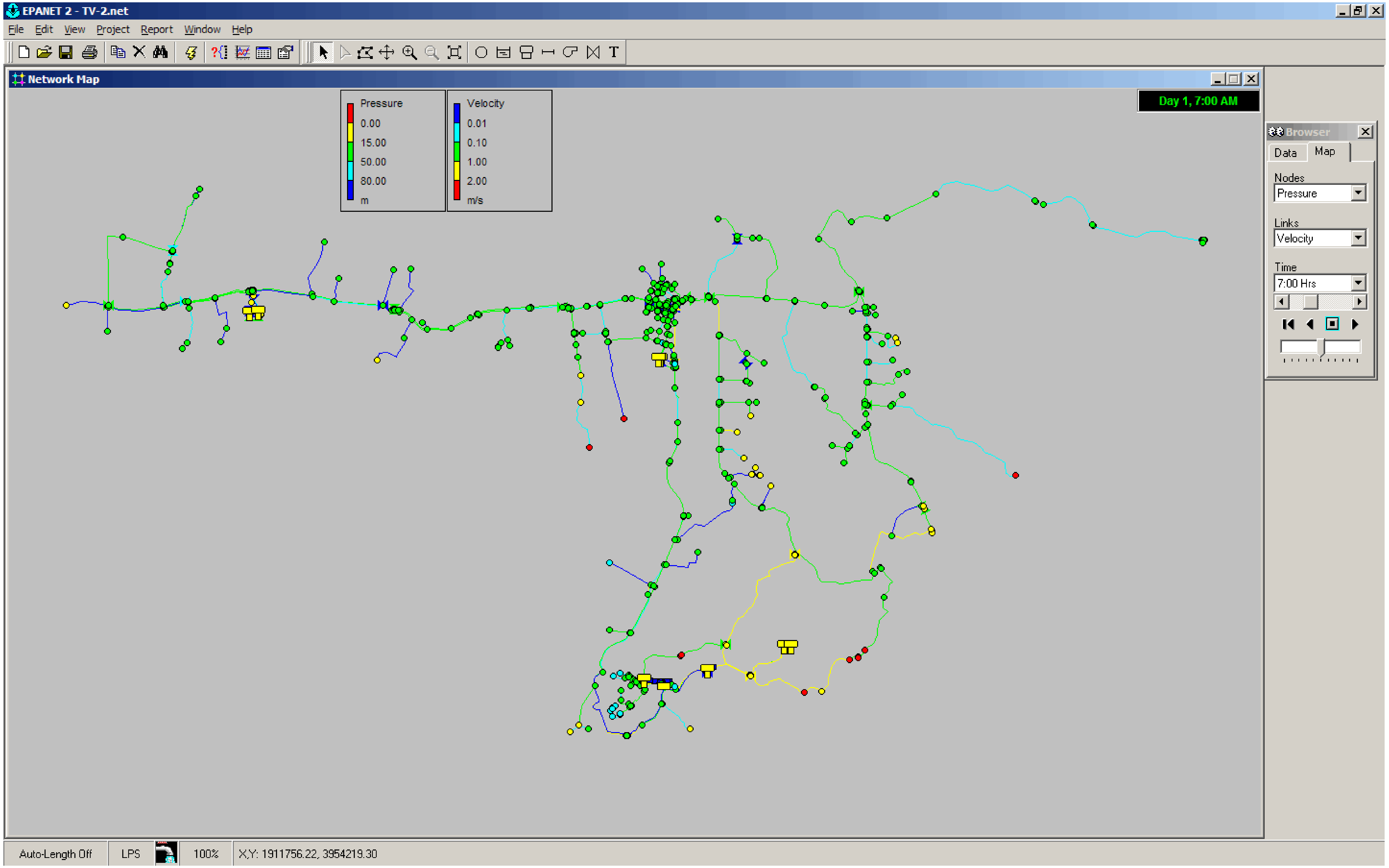

Figure 14 – Tavua/Vatukoula WDN: peak demand hour conditions

4.3 ANALYSIS OF MINIMUM DEMAND CONDITIONS (NIGHT HOURS)

Figure 17 on the next page shows except of a few nodes around Tank 7022 and 7026 the node heads are satisfactory. These problems appear to be caused by low tank water levels, which might be remediated by operational means (changing pumping and valve closing/opening schedules). It is a fortunate still there are only a few nodes with high heads (> 80 m, located at the bottom of the page) that may cause increased losses, or breakages. Relatively frequent pipe bursts occur in that hilly area, and commonly attributed to the effect of stormwater that can cause sudden erosion and even land slides. That may be a factor, and high pressure another contributing factor. Coupled with relatively small demands and resulting high pressures in the wet season, transients and water hammer effects can easily lead to pressure surges that approach or exceed the nominal pressure rate of the pipes.

As expected, only a few pipes have "normal" (between 1 m/s and 2 m/s) mean velocities. Very low, and even stagnating water velocities (> 0.1 m/s) observed at many locations, especially in branch pipes.

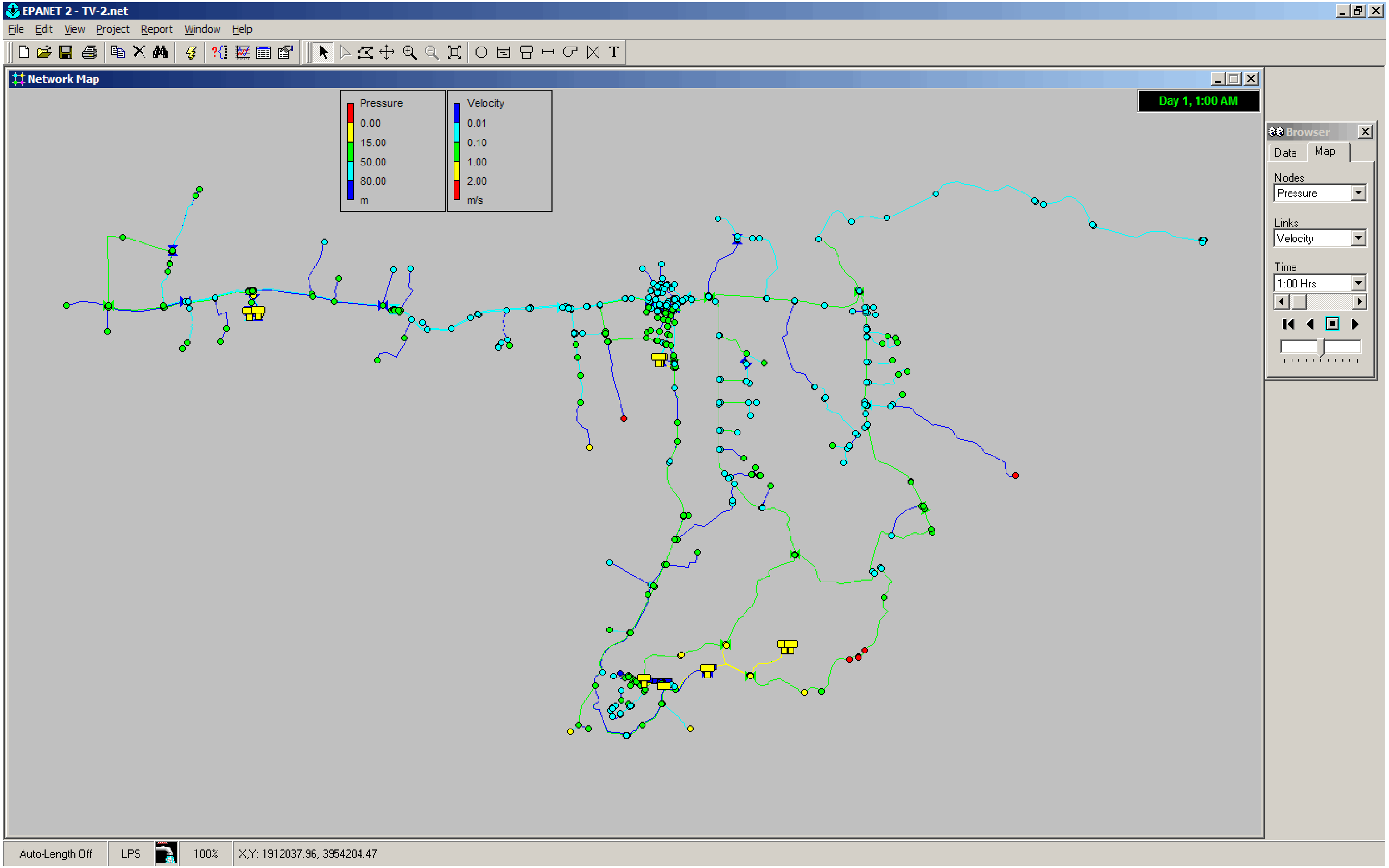

Figure 15 – Tavua/Vatukoula WDN: minimum demand condition

4.4 MAXIMUM SUPPLY PEAK HOUR DEMAND CONDITIONS IN THE DRY SEASON

Figure 18 on the next page shows that while the densely populated areas would have sufficient or at least “working” pressure, very low/grossly inadequate conditions would be dominating several districts South-East and East of Tavua town. In addition, inadequate conditions exist at the end nodes of several branching (single) pipelines. It appears from the map that rectification of the pressure deficiencies might be eliminated using booster (in-line) pumping at several locations, achievable at relatively low costs. In the Nort-East of the town.

The mean velocities remained low, still not exceeding 2.0 m/s high limit anywhere and only about one half of pipes have velocities that are considered “ normal” for many other systems. This characteristics can save significant energy but also result in low residual chlorine levels, especially at peripheral locations.

Overall, the current system would fail to meet pressure requirement in many areas of the township for peak water plant output. Recalling that the plant capacity can be upgraded to 7500 kL/d, the deficiency is even more evident over the planning horizon. It is a lucky circumstance, however that network augmentations to eliminate bottlenecks might be achieved relatively simply and inexpensively.

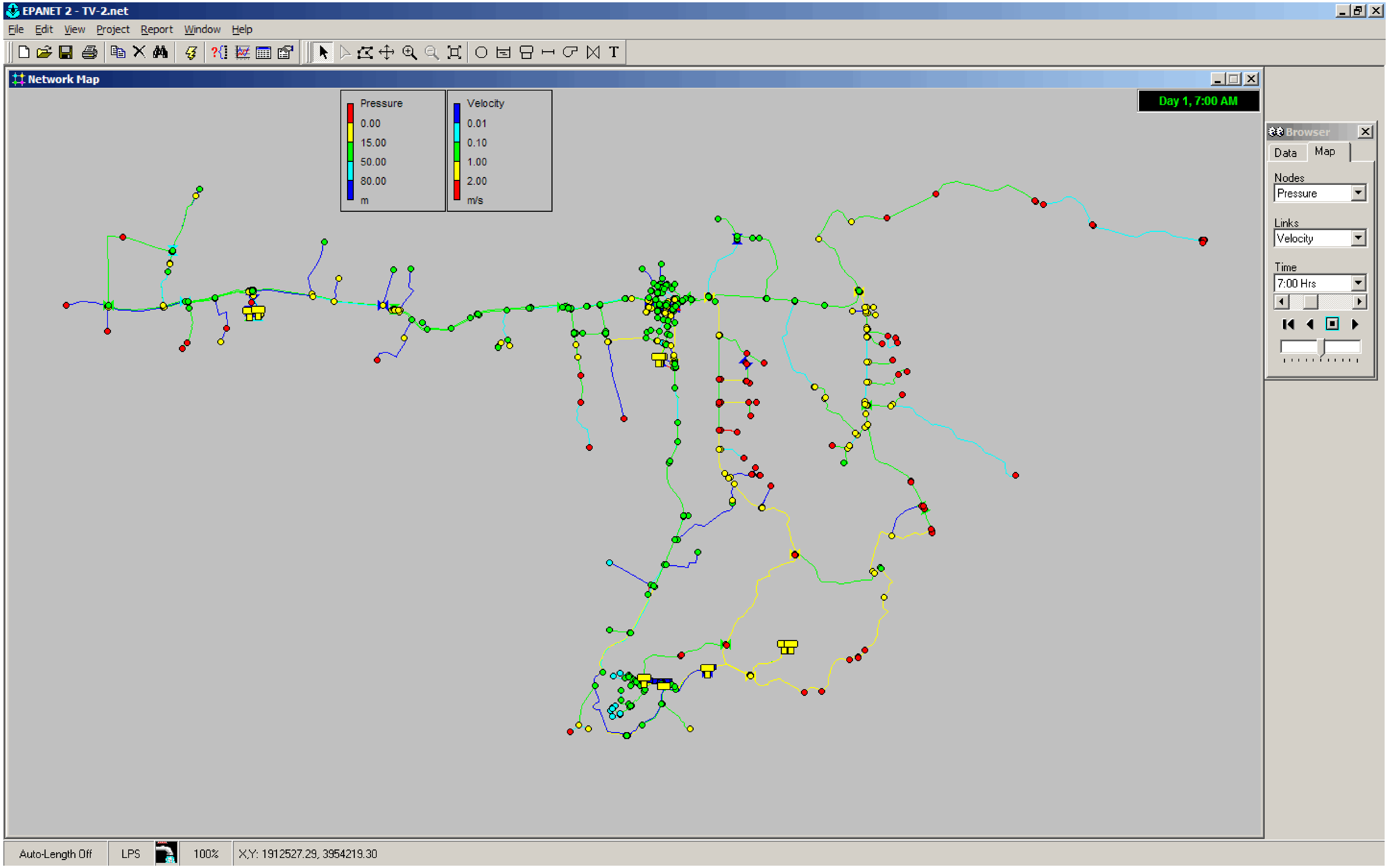

Figure 16 - Tavua/Vatukoula WDN: maximum supply peak hour demand conditions

EPANET also has a counter map display option that allows for a quick identification of problematic areas with low pressures. While Figure 16 provided a relatively detailed overview of the system, similar information can be obtained by observing Figure 17 below.

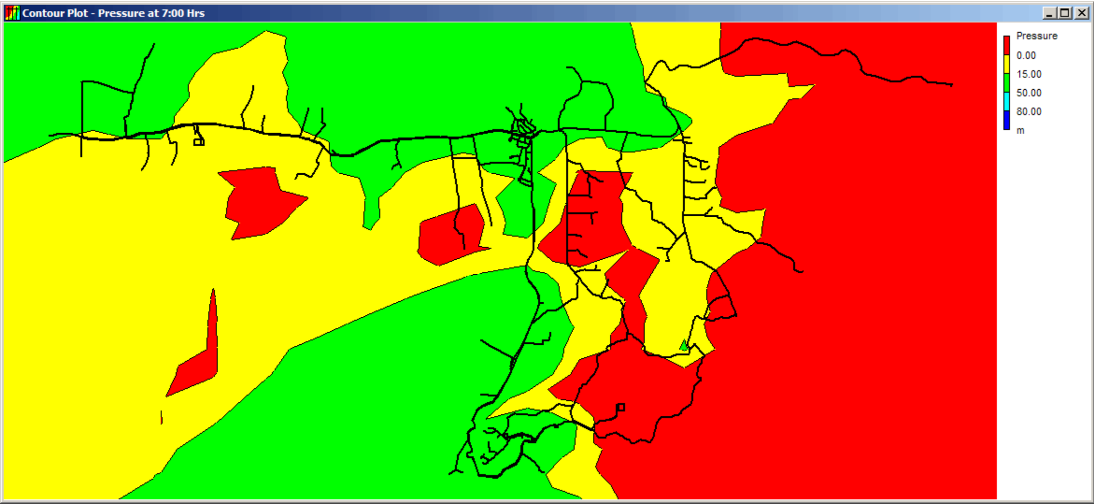

Figure 17 - Tavua/Vatukoula WDN: maximum supply peak hour demand pressure contours

Contour maps also can display other parameters, such as spatial demands as illustrated in Figure 18 below:

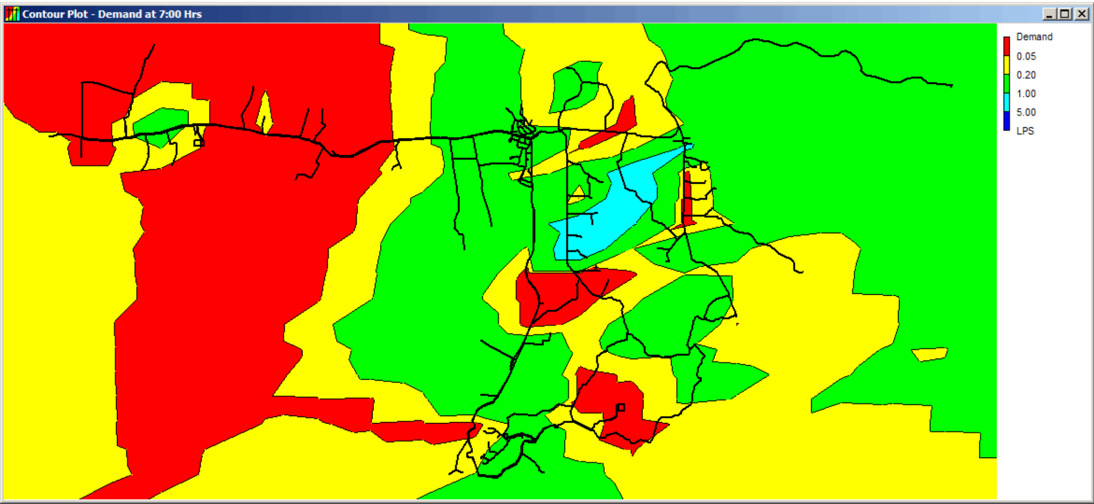

Figure 18 - Tavua/Vatukoula WDN: maximum supply peak hour demand contours

4.5 CONNECTION OF A LARGE NEW CUSTOMER (SUGAR MILL)

A proposed sugar mill would be built south of the town area (Figure 21) and draw supply from Junction 210. The planned cane processing capacity is 250 t/d, with an estimated sugar production of 25 t/d, requiring about 500 kL/d (5.788 L/s) water supply. It is envisaged that the mill would have its own low-level balancing tank, and would use bore water for top-up supply. The estimated demand provided from the network would be about 4.63 L/s, constant over the day.

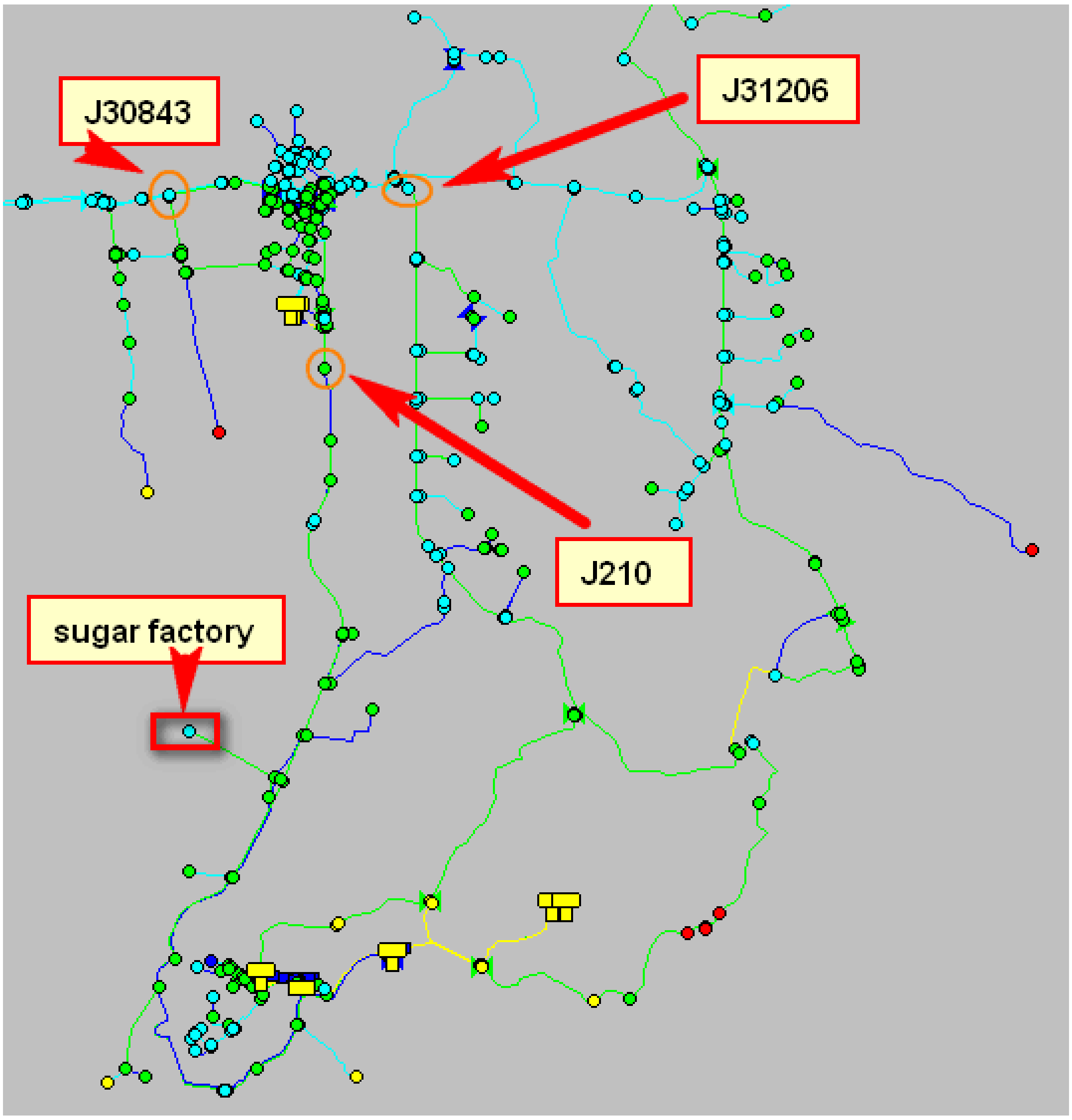

Figure 18 - Location of the proposed sugar mill connection and monitored junctions

The effect of this large customer can be examined and demonstrated, for instance, by presenting estimated pressure profiles at selected nodes, before (Figure 22) and after (Figure 23) connection.

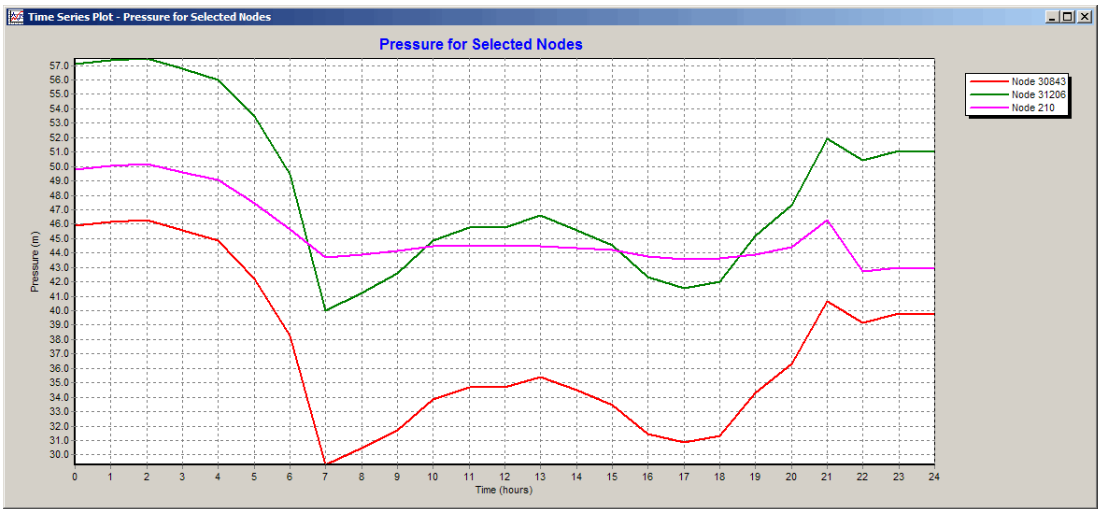

Figure 19 - Pressure of selected nodes before sugar mill connection for baseline demand

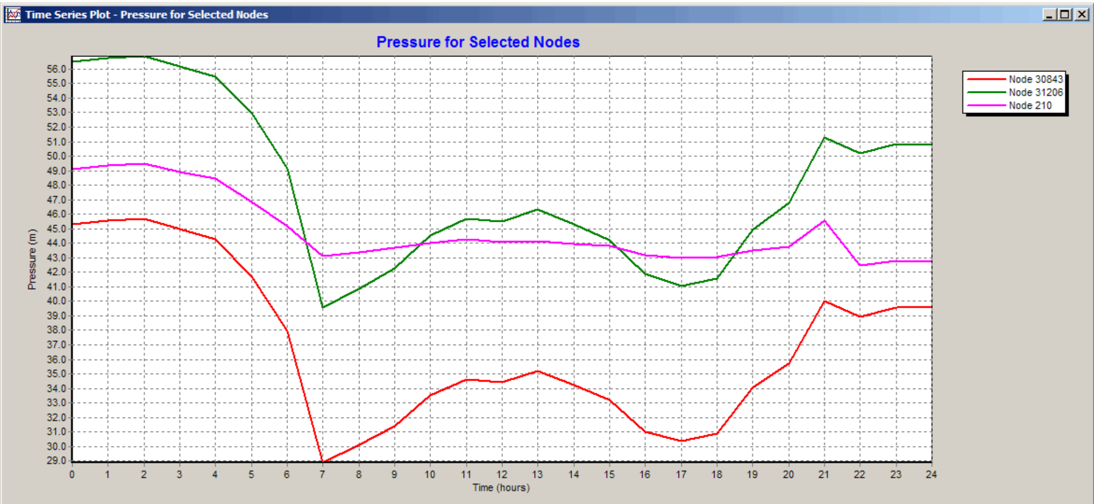

Figure 20 - Pressure of selected nodes after sugar mill connection for baseline demand

Comparison of the charts suggests that there will be only about 0.5-1.0 m pressure reduction (depending on the node) if the mill was connected, and thus the impact on other customers is minor.

4.6 WATER AGE CHARACTERISTIC IN THE SYSTEM

Although EPANET is capable of modelling chlorine decay, it requires sufficient data were unavailable for this study. Nevertheless, water age simulation provides some insight into the water quality degradation (including residual chlorine level decay) expected for this system having low velocities and long pipelines.

For this simulation, water quality analyses were set to 0.05 h intervals, and the simulation period was increased to 168 hours. The interesting and partly unexpected results are presented in Figure 21.

Intuition suggests that areas close to the reservoir and pipelines along major mains should have relatively “younger” water, and in contrast, peripheral areas should have “older” water. Indeed, this is the case in general but the pattern is far from being uniform. In fact, even remote locations such as nearby Rabulu have relatively young, less than 2 days old water. In contrast, most of the branch pipes (expected) and some pipes being part of loops (unexpected) have over 5 days old water, regardless to their vicinity to the reservoir.

Considering the water ages and the relatively high ambient temperature that increases reaction rates, the expected residual chlorine levels should be very low or negligible in many areas. Indeed this assertion is supported by operational data, and the practice of boiling drinking water consumption is a prudent approach.

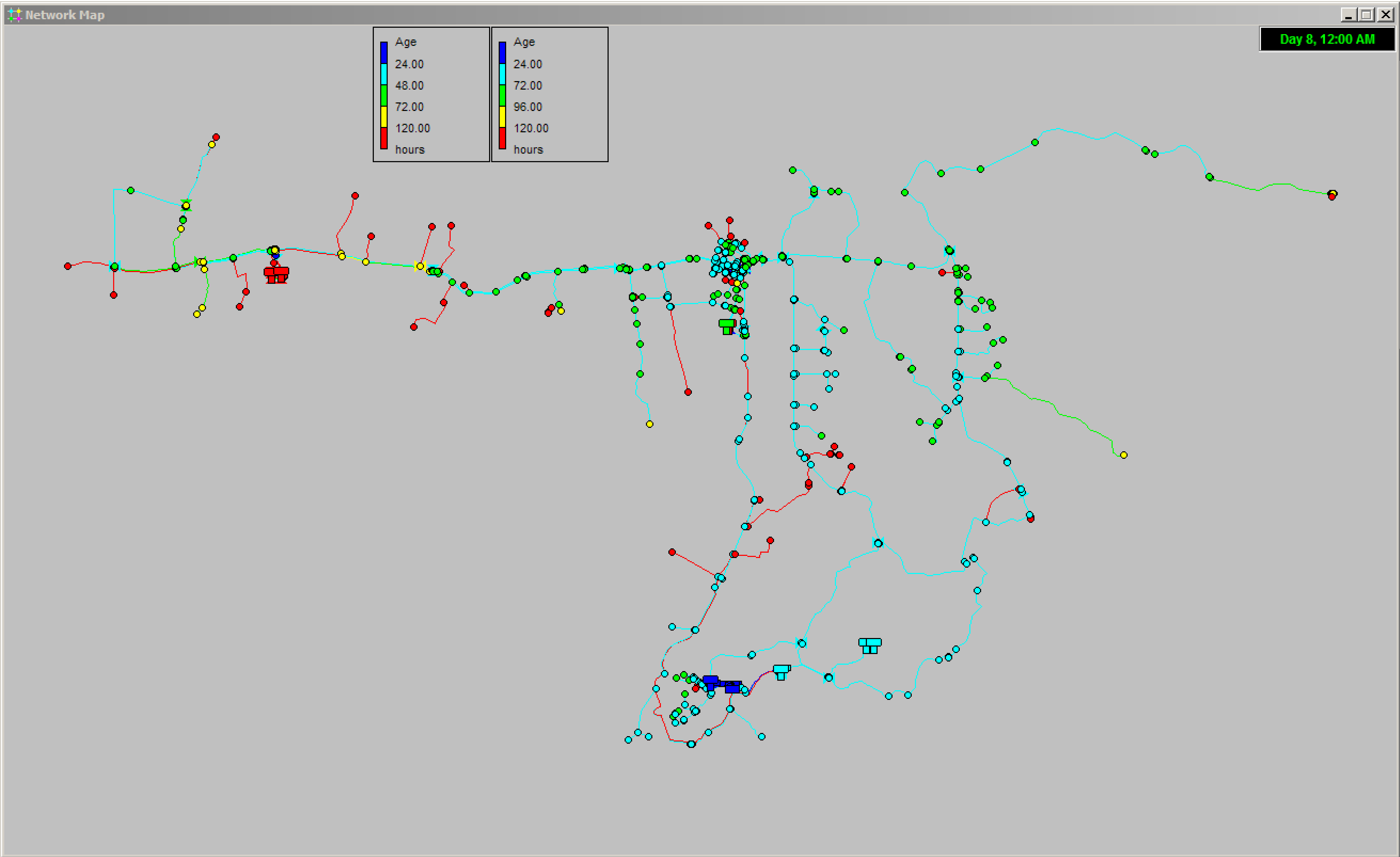

Figure 21 - Water age in the system

5. CONCLUSIONS AND RECOMMENDATIONS

Conclusions

Following a review of the literature that focused on the theory and practice of WDN hydraulic analyses, this study carried out a hydraulic assessment of the water distribution network of Tavua/Vatukoula Township in Fiji.

Time and efforts were spent on finding and choosing the most appropriate tool for analyses. Based on software trials and comparisons, the EPANET v2 was software selected. This software has a clean, minimalist interface and a relatively easy learning curve, and sufficient on-line help is available that were very important for familiarisation and obtaining sufficient user skills.

A substantial part of the allocated time was spent on data gathering that involved searches of office documentations and archives records, and numerous discussions with representatives of the Water Authority of Fiji, Council, and several Departments of the Government. The obtained data comprised of background/geographical information, maps and drawings of the reticulation system, and asset data of network components, census data with population forecast, and information about expected major commercial/industrial developments. Raw data were reviewed and assorted to facilitate the development of a comprehensive model. The EPANET model in its current state is comprised of 588 nodes, 607 pipelines, 6 pumps, and 8 tanks.

Once the model was tested and debugged, hydraulic runs were performed involving both spot and extended period analyses. The findings of the initial runs were reviewed and discussed, and the model was amended according to feedback on (limited) calibration/monitoring data. The finalised model was then used to assess the system, with the essential findings summarised below.

- The baseline analysis showed that in average dry season conditions pressure deficiencies only exist near the reservoir at Vatukoula, and at a small part of Balata. Those are caused by the given tank levels, and to use an in-line booster pump should be the most appropriate remedial action.
- The analysis of pressures at current peak consumption for maximum hour showed insufficient node pressures (< 15 m) in two areas South-East and East of Tavua town and within the town in some areas, as well as at Korovu in the East. These patterns are in agreement with actual pressure observations.

- For the determination of the worst/most demanding operational scenario, it was assumed the water plant will be operated at its full capacity and the system is exposed to peak hour demand. This scenario revealed the major bottleneck is not supply capacity but network deficiency to allow acceptable minimum pressures to many consumers. Insufficient pressure conditions would be dominating several districts South-East and East of Tavua town. In addition, inadequate conditions would exist at the end nodes of several branching (single) pipelines. Again, it appears the rectification program look into the use using booster (in-line) pumping at several strategic locations, which is achievable at relatively low costs

- For all scenarios, mean velocities in pipes remained low, i.e. not exceeding the 2.0 m/s arbitrary high limit anywhere, and in most cases less than one half of the pipes have velocities that are considered "normal" for many other systems. This characteristic is a result of the hilly and sloping geography of the supplied area.

- Analysis of pressures for minimum consumption (night hours) showed that high pressure (> 80 m head) exists in mains near the Reservoir that may cause increased losses and or breakages due to transients and water hammer effects. In this condition typically very low and even stagnating water velocities (> 0.1 m/s) exist at many locations, especially in branch pipes.

- Analyses showed that there is sufficient water plant capacity exist to provide about 4.63 L/s continuous supply to a proposed sugar mill located south of Tavua town. This supply would only marginally (max 1. 0 m) reduce the pressure at earmarked node locations, and thus would have little impact on other customers.

- There was no sufficient data to carry out any comprehensive water quality analysis, chiefly residual chlorine decay analysis. A water age simulation carried out over a 168 hours period showed that water age is quite acceptable (up to 2 days) even in remote locations such as around Rabulu. However, most of the branch pipes deliver over 5 days old water, even close the to the reservoir. This finding suggests that residual chlorine levels should be very low or negligible in many areas.

Recommendations

- The network model is will require regular updates and more calibration data.

- The Water Authority should consider linking the hydraulic model to a GIS database to facilitate rapid upgrade, integrated use including operational and billing data.

- Some uncertainties remain about the model due to limited calibration data. The Water Authority should establish monitoring locations within the system, and appropriately enhance the hydraulic model to bring about its full benefits (suitability for detail design).
- While the Tavua/Vatukoula water supply system has an acceptable leakage rating, the high percentage (30%) of Non Revenue Water is an ongoing concern.
- Some distribution mains, especially at Vatukoula near the Reservoir is exposed to relatively high pressures. There are no serviceable PRV in the system, and the use of manual valve throttling is unreliable. Pressure must be controlled before leakage on smaller reticulation mains can be repaired.
- Several districts have significant pressure deficiencies. Analyses revealed that those are definitely caused by high ground elevations, and therefore the use of booster pumping should offer a rational approach to remediation actions.
- Due to large variations in elevations, it is recommended to establish supply zones/districts, which should ameliorate the predicted pressure problems that will arise with increasing water plant deliveries.

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7. APPENDIX

7.1 APPENDIX A - PROJECT SPECIFICATION

University of Southern Queensland
School of Civil Engineering & Surveying

ENG 4111/4112 Research Project Project Specification

Student: **Kemueli** (**Bainivalu) Senokonoko** (0050105019)

Topic: HYDRAULIC MODELLING OF THE WATER DISTRIBUTION SYSTEM OF TAVUA/VATUKOULA, FIJI

Supervisor: Dr. Laszlo Erdei

Enrolment: ENG 4111 – S1, D, 2014
ENG 4112 – S2, D, 2014

Project Aim: The aim of this project is to investigate the water supply distribution system of a Fijian town in terms of hydraulic performance, with regard to planned developments, network augmentation and pressure and leakage management.

Sponsor: Water Authority of Fiji and School of Civil Engineering & Surveying, USQ

Programme

1. Obtain referencing software (EndNote) and formula editor software (MathType) from the USQ Library site, learn their use, and set up an MS Word template for the dissertation.
2. Conduct a literature review, covering the theory and practice of water supply network hydraulic analysis and related water quality investigations.
3. Review several possible software packages required for the analyses, select the preferred tools, and become skilled in their use.
4. Gather and review distribution system data and pertinent operational information.
5. Conduct hydraulic analyses for using typical operational scenarios involving both snapshot and extended period simulations.

6. Process, review, and analyse the obtained data, discuss the findings and results, and make recommendations in a dissertation, as well as produce a summary for a short presentation.

Agreed:

(Student)

(Supervisor)

14/ 03 / 2014 12 / 03 / 2014

7.2 APPENDIX B - CONSEQUENTIAL EFFECTS

This research topic is the first to be done in Fiji; hence it will emphasise the important of water distribution modelling system for the country. There were a lot of development taken place in the country and there is a need to invest to the most important development, such as water network infrastructure. With this information it will be a guide to anyone who wishes to continue research on the topic. A country like Fiji needs to move forward in terms of development such as new modelling technique.

7.3 APPENDIX C - SAFETY ISSUES

There is no physical risk for this research work. This project mostly involves the use of computer to simulate the water distribution hydraulics. Most of the time spent on this project is on gathering data and information related to the water distribution system network in the area, and familiarisation with the model. The upcoming tasks involve the model calibration and simulation and reporting of results.