AARHUS UNIVERSITY
SCHOOL OF ENGINEERING

The Wonderful Water Supply of Skovby

BTFPM2 Water Supply Engineering 2017

Aarhus University – School of Engineering

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ABSTRACT

The water supply system in the municipality Skanderborg, Skovby is analysed. Included are the analysis of the aquifer, the treatment facilities, the water consumption, the current distribution system modelled with MikeUrban 2014 by DHI and recommendations when dealing with arsenic or pesticides in the groundwater.

There are 3 wells in operation: DGU 88.813, DGU 88.723, and DGU 88.1394. They all belong to a glacial meltwater sand aquifer. The water was found to be suitable for drinking except for some exceeded values (Fe, Mn, NH_3, CO_2) but all of these can be easily addressed by conventional treatment process. Furthermore, the Skovby water work can treat and supply a higher demand if necessary. None of the risks examined were deemed serious for the operation. Skovby is divided into 2 zones: high pressure and low pressure. The low-pressure zone uses more water due to the larger area and the diversity in consumer types (households, industry, institutions). The existing distribution system has low velocity, possibly due to over-dimensioning of pipes. This creates opportunity for future development. A likely scenario of new residential area was devised and modelled. It has no severe impact on the existing distribution system, although some pipes should be replaced to avoid bottlenecks. Lastly, how to treat water sources with elevated level of either arsenic or pesticide was investigated. Both cases were studied to show which should take priority. Arsenic removal was analysed more deeply because although pesticides seem easier to treat (most suitable with activated carbon), currently there is no apparent threat of pesticides to water sources in Skovby. A scheme to remove arsenic with chlorine pre-oxidation and iron chloride coagulant based on iron-manganese removal process was recommended.

FOREWORD

This report is made as a part of the Urban Water Engineering Semester Spring 2017 at Aarhus University, School of Engineering (BTFPM2 Water Supply Engineering 2017).

We considered ourselves as a consulting company hired by the municipality of Skanderborg, for whom the report was written.

References in the report text is given according to Harvard styles and in square brackets, like so: [author/creator, year of publish]. In the end, before the Appendix, a full list of bibliography and references are organized in order of appearance in the report. The Appendix consists of all additional information as well as more detailed description of several parts, hence it is advisable to refer to this section whenever it is mentioned in the report. We use dot (.) to express decimal division and thousands is not divided by any symbol. The numbering of figures and tables is according to 1st tier of headlines. For example, in chapter 3 about Water consumption and Distribution system, the first picture to appear would be numbered FIGURE 3.1, the second to appear is FIGURE 3.2, no matter which sub-section it is in and so on. The same system is applied for tables. Titles of figures are placed beneath the figure, and tables above. The Appendices will be numbered alphabetically (i.e.: Appendix A; Appendix B etc), and referred to in the text where appropriate.

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1. PROBLEM DESCRIPTION

Skovby is a small town in the region of Central Jutland, Denmark. It belongs to Skanderborg municipality and has a population of 2489. About 150000 m^3 of water is supplied to approximately 1300 consumers, consisting of household and industries. The water supply is based on intake from three wells to the waterworks at Hørslevvej 9, Skovby, 8464 Galten. The waterwork consists of two plants next to each other, constructed in the 1960s and 1977 respectively.

FIGURE 1.1 Satellite overview of Skovby [Google Maps, 2017]

The objective of this report is to analyse various aspects of the current water supply system of Skovby.

1.) The characteristics of the intake aquifer, such as material, boundaries, recharge and vulnerability to contaminations. The geological profile and intake wells, and the water quality both in each well and in the system as a whole.
2.) The process of the current treatment facilities. The key parameters of the new and the old waterworks in order to see if they work as they should with the water of each well. Also, the capacity and the risks of this waterworks will be analysed.
3.) The current water consumption in Skovby (total consumption, variation, differences between the two zones) as well as the assumption for the future consumption and the resulting impact on the current water distribution system.
4.) The modelling of the water distribution system with MikeUrban 2014, along with the interpretation of various elements in the system, like the flow, velocity, pressure, water age, fire flow, critical parts and incidents like closed pipes and the impact on the system. Also, the impact for a new residential area according to the future consumption.
5.) A possible scenario was devised in which water sources with arsenic and pesticides is considered potential supply. Brief description of common methods for treating the two would be presented along with an elaborated scheme for arsenic removal integrated into the existing treatment line.

Data for the existing supply system, terrain elevation, locations of pipes and hydrants were taken from webGIS of Skanderborg municipality and Skovby water supply online map. Data for the water consumption and water quality was provided by the water work and taken from Jupiter. Large amount of information in the Arsenic and pesticide removal part can be referred to the U.S. Environment Protection Agency (EPA). Suggestions and useful sources recommended by Peder Maribo and Kristian Vestergaard, Senior Lecturers of Aarhus University were also exploited.

Due to the lack of data, calculations and the modelling are based on a lot of assumptions and simplifications, therefore the results may not be absolutely certain.

2. ABSTRACTION AND TREATMENT OF RAW WATER

This chapter investigates the aquifer and intake wells to the water supply system. This includes looking at the type of aquifer, the geological profile, and recharge to the aquifer. The intake wells and the recharge to each of these will also be examined. The water quality in each well will be looked at, and the overall water quality determined. The most important parameters for water quality are to be studied, and those that need treatment indicated.

2.1 Intake Aquifer

The aquifer that the wells drill into is glacial meltwater sand. This refers to the deposits that would have occurred as a glacier retreated, so contains the sands and gravels that were suspended in the glacier. A sand aquifer tends to have higher porosity than an aquifer of smaller particle sizes (such as clay or silt), with better connected pores. This allows water to flow more freely through the aquifer.

Based on the information from the three intake wells and other wells in the area, the geological profile can be roughly drawn. This gives an idea of the types of layers that are present around the intake wells. Under the assumption that the aquifers are continuous between the wells, the following profile is assumed. This is only a 2D representation, and doesn’t account for the fact that there could be different materials present that haven’t been picked up between the wells.

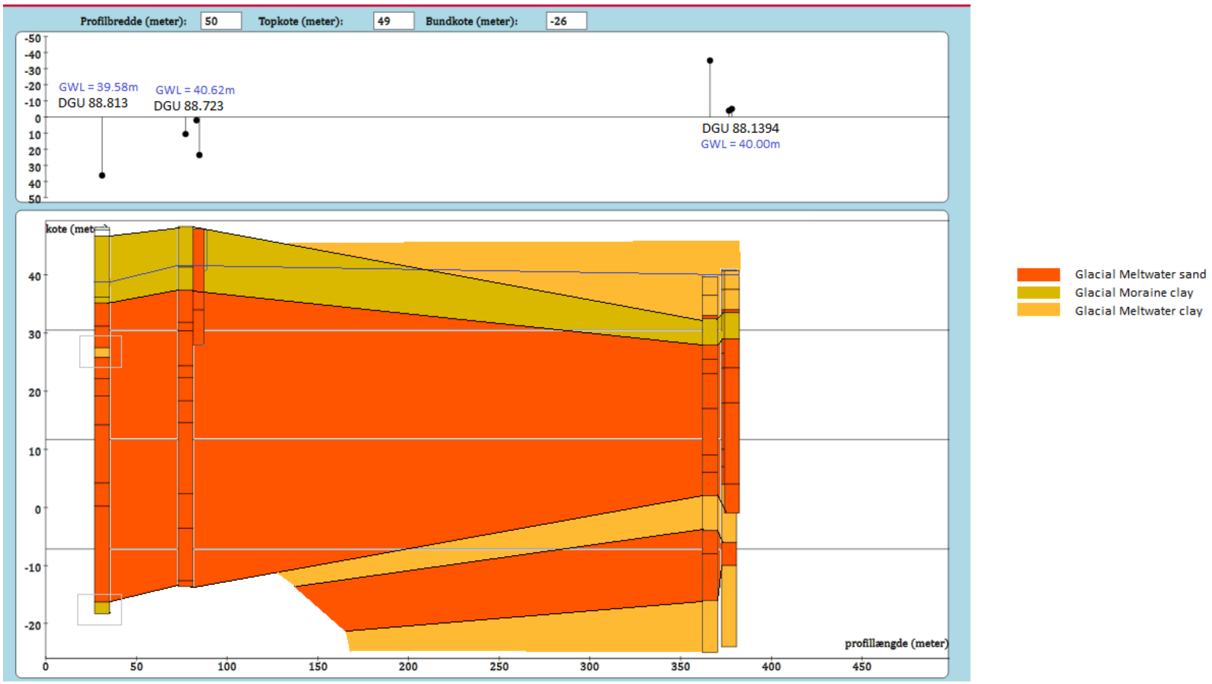


FIGURE 2.1 Geological profile based on well data [GEUS, 2017]

This indicates that the aquifer of interest is confined, as it is overlain by a layer of clay. This is relatively impermeable, and water moves through it slowly, so there is limited groundwater flow into the aquifer from above. The water table lies above the upper boundary of the aquifer. The aquifer is therefore fully saturated, and under pressure.

Since here is a layer of clay above the sand aquifer, it is quite well protected. This means that groundwater in the sand aquifer is not that vulnerable to pesticide and other contaminants on the surface as the water filters through the clay layer before it reaches this.

The aquifer is recharged by the rainfall that infiltrates into the ground. The rainfall in Denmark is shown in Figure 2.2, and the average rainfall for various regions can be estimated based on this map. The area of focus is Skanderborg (located roughly where the red circle is drawn), which has an annual rainfall of approximately 700mm. The average monthly rainfall in mm is detailed in Figure 2.3, which sums up to about 788mm in 2016.

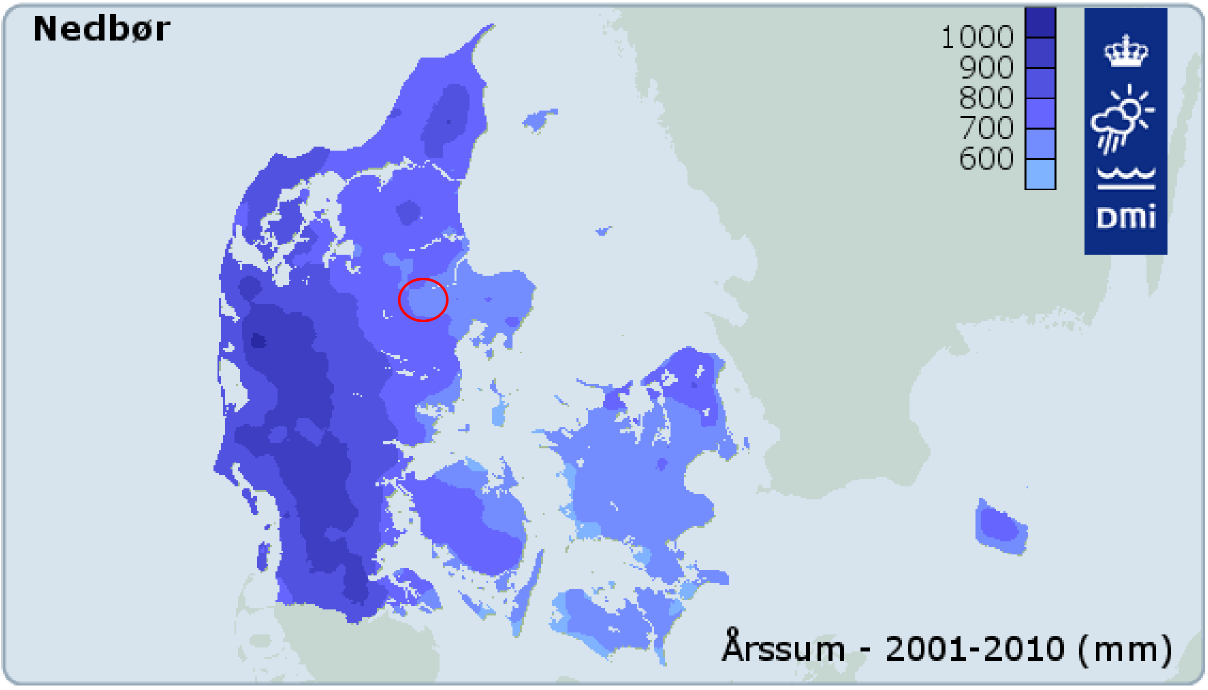


FIGURE 2.2 Rainfall in Denmark [DMI, 2017]

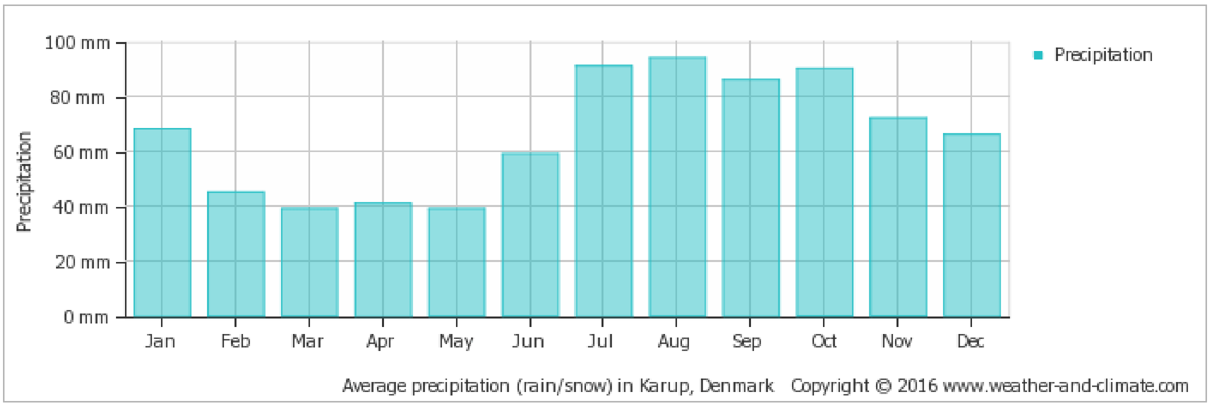


FIGURE 2.3 Rainfall in Skanderborg region in 2016 [World Weather & Climate Information, 2016]

The amount of water that infiltrates into the aquifer depends on many factors, including land usage, geology, vegetation and climate. The greatest factor is the precipitation, such as the amount, duration and intensity of rain or snowfall. The soil type also has an influence, as some materials such as clay absorb water at a slower rate than larger grained material such as sand. If the aquifer is made up of a material that holds less water, there will obviously be greater surface runoff and less infiltration. In relation to this, the soil saturation also influences the amount of water that can infiltrate. If the ground is already saturated from earlier rainfall, less water can be absorbed and becomes surface runoff. Land use and land cover affect how much water can infiltrate. For example, land used for farming and covered in grass has a different infiltration rate to land used for forestry and covered in pine trees. The primary difference comes from whether the vegetation has been removed and the area has been paved. In places in which the ground is covered by structures, there is very limited infiltration and most rainfall runs overland and into drains.

2.2 Intake Wells

2.2.1 Overview

Currently there are three intake wells to the water supply system. These are DGU 88.813, DGU 88.723 and DGU 88.1394 (or Boring 1, 2 and 4 respectively). The current water intake is around 150000 m^3 per year, and has historically been as high as 250000m^3 per year. DGU 88.1307 (Boring 5) lies to the east and is no longer in use. Previous water samples from this well show that the arsenic content is relatively high so this may be why it has been discontinued for water production. The three active production wells lie in the red square, and the well that is no longer active is shown to the right in Figure 2.4.

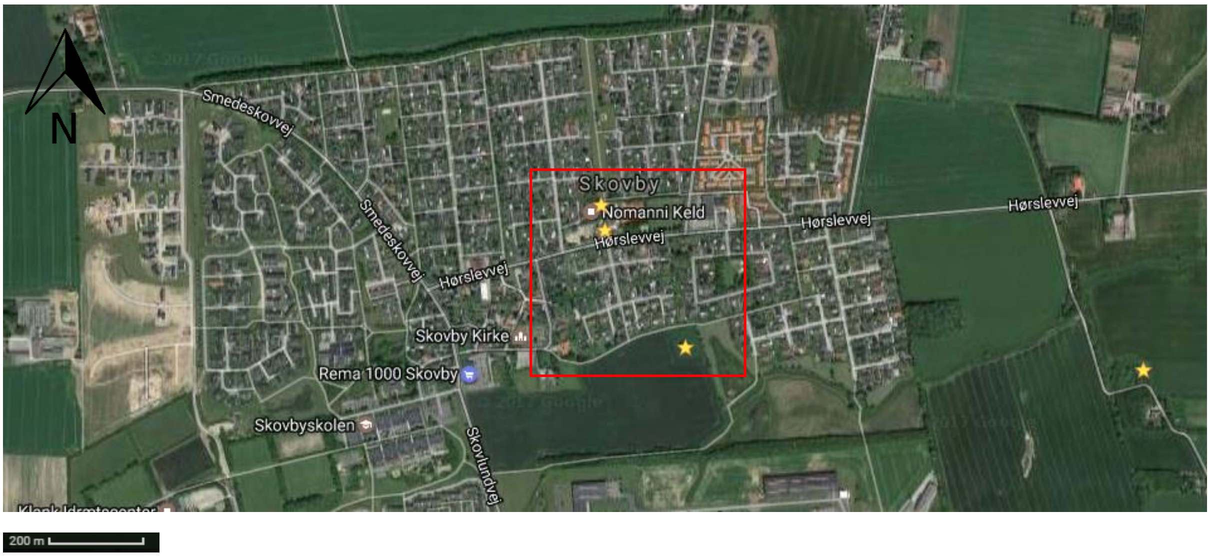


FIGURE 2.4 Satellite image of Skovby area [Google Maps, 2017]

2.2.2 Recharge

Based on satellite imagery, most of the area looks residential or business, and a small part is agricultural or open. This relates to different infiltration depending on the location.

The locations of the three wells are indicated on the figure below. DGU 88.813 and DGU 88.723 are in residential areas, and according to the Urban Storm Drainage Criteria Manual, a lot size of less than quarter of an acre relates to an imperviousness of 45% [UDFCD, 2017]. DGU 88.1327 is located in a park which has an imperviousness of 10%, but is also surrounded by residential areas. With this is mind, it is assumed that the imperviousness in this area is about 35%.

FIGURE 2.5 Locations of intake wells [Google Maps, 2017]

The infiltration rates of soils depend on many factors such as the saturation of the soil, the intensity of the rainfall and the plants present or usage of the land. For more saturated soil, it can absorb less water so there will be more runoff. This also changes depending on the rain conditions, as a long, light rainfall event will allow more water to be absorbed than a short but intense storm. In general, the infiltration rates for different soil types are given in Table 2.1 [FAO, 2017]

TABLE 2.1 Basic Infiltration Rates for Various Soil Types

Soil type	Basic infiltration rate (mm/hour)
sand	less than 30
sandy loam	20 - 30
loam	10 - 20
clay loam	5 - 10
clay	1 - 5

As seen, clay can absorb much less water than sand. This is given in terms of how many millimetres of water can be absorbed per hour. Assuming that an average rainfall event is around 15mm/hr, then clay allows for approximately 30% of rainfall to infiltrate. Since sand can absorb a lot more water, it is assumed that about 60% of rainfall can infiltrate.

The recharge to an aquifer depends on the rainfall, infiltration rate and area of catchment. The catchment depends on the topography as to which way water will run. Although it is not necessarily the same as the groundwater gradient. In simple situations, the abstraction area and the groundwater recharge area are the same, that is, the part of the land area from which rain infiltrates down into the aquifer.

Figure 2.6 shows the groundwater potential contours at 1m intervals for the Skovby area. This indicates the way groundwater will run, from higher to lower areas. This means that the aquifer is likely to have a greater yield in the east of the region.

FIGURE 2.6 Groundwater Potential Contours in Area of Interest [Skanderborg Webkort, 2017]

2.2.3 Boring 1 (DGU 88.813)

This was drilled in January 1977 to a depth of 66.5m, so it is 40 years old. Based on the consumption data from 2015, the abstraction can be estimated based on whether the water level drops in Well 1 or Well 4 when pumping occurs. This shows that approximately 71436 m^3 of water is abstracted from this well.

Since the available flow is the area of interest times the infiltration, the recharge area can be estimated. This will not be entirely accurate as it is based on the water that is being abstracted, not the amount that could be abstracted.

This well is situated with an overlying clay area.

Assuming that 55% of rainfall occurs in an area in which it can infiltrate, and that clay allows for 30% of water to infiltrate, the amount of water going into the aquifer is:

$$788mm \times (0.55 \times 0.30) = 130.02\ mm$$

Thus, the area of recharge for this well is calculated as:

$$A = \frac{Q}{I} = \frac{71436m^3}{0.130m} = 549423\ m^2$$

This amounts to 0.55km^2, situated in the radius around the well.

2.2.4 Boring 2 (DGU 88.723)

This well was drilled in March 1973 to 62m, and is 44 years old. The consumption data from 2015 shows that this boring abstracts about 61106m^3 of water a year.

This well also has a clay layer above the aquifer, so using the same infiltration rate as above

$$A = \frac{Q}{I} = \frac{61106m^3}{0.130m} = 470046\ m^2$$

The recharge area for this well is estimated to be 0.470km^2 around the well.

2.2.5 Boring 4 (DGU 88.1394)

This well has no boring date in the system, but it has a consent last issued in 1999, so is at least 18 years old. It reaches a depth of 42m. Based on the 2015 consumption data (assuming that when the water level drops in this well when pumping occurs, the amount being abstracted comes only from this well), it abstracts approximately 36885m^3 of water annually.

This well is overlain by a sand layer, instead of clay. It is also located in a less residential area than the previous wells, so the imperviousness is lower as explained earlier. Sand allows more water to infiltrate than clay as it has larger particles and pores, so water moves more freely through it. It is likely that sand allows for about 60% of rain to infiltrate, so this amount of water is available to further permeate into the lower aquifers.

$$I = 788\ mm\ \times (0.65 \times 0.60) = 307.32mm$$

$$A = \frac{Q}{I} = \frac{36885m^3}{0.307m} = 120146\ m^2$$

The recharge area is estimated to be 0.120km^2 around the well.

2.2.6 Groundwater Chemistry

In this section, the water quality of each well is going to be analysed to check if it is good enough for the future treatment in order to became drinking water.

The JUPITER database [GEUS, 2017] provides information for each well, and recent samples taken. These detail the chemical composition of the water. The parameters to be analysed for water quality are the pH, ion deviation, degree of ion exchange, redox type, degree of weathering, calcite saturation, non-volatile organic compounds (NVOC), ion balance, and parameters that exceed the criteria for drinking water.

The three wells were studied individually, and returned very similar results. Due to this, the average values have been reported in order to give a more holistic view of general water quality in the area. For the parameters that exceed the drinking water criteria, the worst-case scenario out of the three wells was taken.

The initial data for each well is described in Table 2.2.

TABLE 2.2 Analysis Report of Water Quality Parameters for Each Well

	DGU 88.813	**DGU 88.723**	**DGU 88.1394**	**DGU 88.1307**
Date	*17/11/2015*	*16/11/2016*	*24/11/2014*	*14/05/2002*
NH_4 (mg/l)	0.07	0.06	0.02	0.645
Agg CO_2(mg/l)	5	2	5	n/a
Fe (mg/l)	0.297	0.54	0.417	n/a
Mn (mg/l)	0.582	0.446	0.413	n/a
CH_4 (mg/l)	0.01	0,01	0.01	0.01
SO_4 (mg/l)	62	72	56	16
As (µg/l)	3.4	3.22	2	18
Ca (mg/l)	83	81.5	80.5	43.6
Mg (mg/l)	7.59	8.75	9.08	11
NO_3 (mg/l)	0,5	0.5	0,5	0.5
Na (mg/l)	16.3	17	16.1	77
Cl (mg/l)	31	34	29	25
HCO_3 (mg/l)	214	203	219	2
NVOC (mgC/l)	1.2	1.1	1.4	1.7

pH

The pH value defines the acidity or basicity of the water. The average pH of the three wells was found to be 7.6. This lies in the typical range for groundwater thus does not pose any further problems.

This pH also indicates the acid-base type of the water. In this case, it is buffered, meaning that it resists changes in pH as small amounts of acid or base are added. The pH in this water is slightly higher than neutral because of the dissolved minerals present.

Hardness

The hardness of water relates to the presence of magnesium and calcium ions. To calculate the hardness, the following formula [Maribo, 2017] is used, in German degrees (°dH)

$$dH = \frac{1dH}{10mg/l\_CaO} \cdot \frac{56mgCaO}{mmolCaO} \cdot \left(\frac{Ca^{2+}}{40.1mg/mmol} + \frac{Mg^{2+}}{24.3mg/mmol}\right)$$

The average value of hardness was found to be 13.35 dH, which is a medium value. This indicates that no major problems need to be addressed in terms of the hardness.

Redox

It should be noted that for each individual well the redox type was the same, showing consistency. Based on the concentrations of nitrate, iron, and sulfate it was found that this water is redox type C, with no conflicts. This refers to weakly reduced water.

Degree of Ion Exchange

The degree of ion exchange indicates the age of the water, and how much groundwater flow occurs. The degree of ion exchange was found to be 0.8 indicating a typical ion exchange ratio for groundwater with limited cation exchange capacity. There has not been a change in salt concentration recently which suggests that the groundwater could be young and vulnerable. Taking into consideration the other factors and information available, such as the overlying clay layer, this is unlikely.

Degree of Weathering

Some of the most important processes that takes place in the groundwater are pyrite oxidation, sulfate reduction and ion exchange. To measure whether these processes have taken place at any point, the degree of weathering (F) is calculated [Maribo, 2017].

$$F = \frac{2\left(\frac{Ca}{40.1} + \frac{Mg}{24.3}\right)}{\frac{HCO_3}{61.0}} = 1.38$$

The resulting value was 1.38, so little pyrite oxidation can be assumed to have occurred (as values for pyrite oxidation lie between 1.3 and 3.5) [Maribo, 2017]

Calcite Saturation Index

The calcite saturation index shows if the water is saturated and aggressive. It is found using the following formula [Maribo, 2017].

$$LogSI = pH - 11.4 + log\,(\,Ca^{2+} \times CO_3{}^{-}\,)$$

The equation above is based on assumptions and simplifications, such as a temperature of 25°C, ion strength of 0.005 mol/kg and that no aqueous complexes are formed. The resulting value was 0.4, which indicates saturated, non-aggressive water. This agrees with the pH and typical aggressive CO_2 level.

NVOC

The NVOC (non-volatile organic carbon) value is the content of organic matter in the water. It´s important to know this value because it can generate brown water which is not ideal for drinking water from a consumer point of view. From Jupiter, it is found that the NVOC is 1.2 mg C/l. This means there is a typical value of organic matter in the water.

Ion balance

The ion balance is a calculation that verifies if the charge balance is correct, as an error in this balance indicated that the sample is invalid. The sum of the cations should be equal to the sum of the anions. In this case, the charge balance deviation was 3.5%, which is less than 5% thus the sample can be assumed to be valid.

Exceeding Values

The criteria for various contaminants in drinking water are given in Table 2.3. Given below are the parameters that are exceeded in the most recent samples, and the worst-case value out of the three wells recorded. The criteria for drinking water can be found in Appendix A: Water Quality [Maribo, 2017].

TABLE 2.3 Contaminants that need to be treated to reach Drinking Water Criteria

Parameters	**Concentration (mg/l)**	**DW Criteria (mg/l)**
CO_2	5	2
Fe	0.54	0.1
Mn	0.582	0.02
NH_4	0.07	0.05

All of these issues are easily remedied with standard treatment methods as discussed further in section 2.3 Treatment facilities.

Conclusion

The water in Skovby is of good quality before treatment. The water is buffered, weakly reduced and without conflict, saturated and non-aggressive. The pH, hardness, and organic matter are quite typical. No ion exchange has occurred, but it is possible that some pyrite oxidation has. As the value is so close to that of typical water, and since the pH and hardness are also in the typical range, the amount that may have occurred is likely very small. There are some parameters that exceed the limits for drinking water, but these are easily treated with standard treatment methods. The samples appear to be reliable. It is possible that the water is young and vulnerable based on the ion exchange ratio, but not necessarily, and based on the other data it is thought to be quite well protected.

2.3 Treatment Facilities

In Skovby the water extracted from the wells is pumped to the waterworks. This section is to analyse the water treatment facilities. There are two waterworks, the old waterwork was built in the 1960s and the new one built in 1977. The old waterwork receives raw water from Well 2 (boring 88.723) and has two open gravity sand filters with a free fall aerator. The new waterwork receives water from Wells 1 and 4 (boring 88.813 and 88.1394) and is equipped with 3 open gravity sand filters and an aeration basin. There are also 2 holding tanks, one with a volume of 250 m^3 and a new one from 1977 that has a volume of 500 m^3.

The steps that take place in the waterworks are the aeration and the sand filtration, resulting in clean water and backwash as is explained in Figure 2.7.

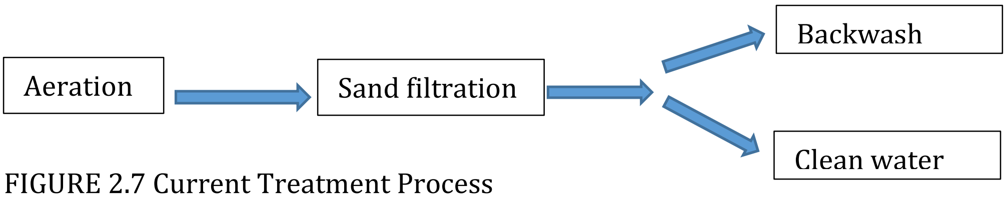


FIGURE 2.7 Current Treatment Process

The following study is about the key parameters of the aeration, the filtration and the backwash.

2.3.1 Aeration

The process of aeration is to get the oxygen in the water that is needed for the removal of components such as iron, manganese, ammonia, methane, hydrogen sulphide and aggressive CO_2.

In the old waterworks, there is a free fall aerator which consists of putting water in air, and the water movement by gravity drop. On the other hand, there is an aeration basin in the new waterworks. This method is different from the free fall because while there is still air in the water, the advantage of the basin is lower humidity which is good for keeping the plant safe from the algae growth.

One of the functions in the aeration is stripping unwanted gasses such as methane, hydrogen sulphide and aggressive CO_2. These gasses can generate bacteria growth or odours. In this case methane and hydrogen sulphide are not a problem because the concentration is very low (0.01 mg/l for both components). The CO_2 has a concentration of 5 mg/l which is a bit higher but still not a problem because this is easily treated by either method.

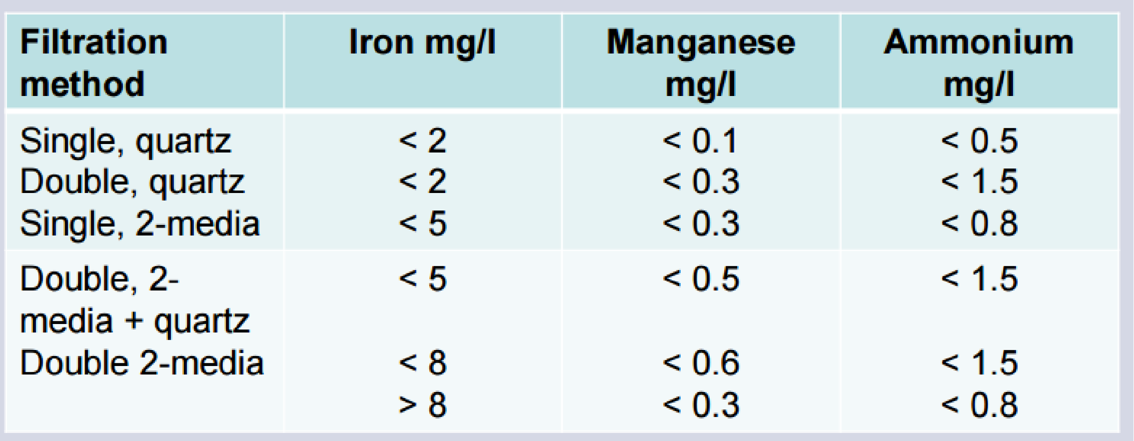
For aeration, it is important to know the total oxygen demand required to remove the contaminants to the desired level. To calculate this parameter the concentrations of iron, manganese and ammonium were used. The oxygen required to treat each component is known, so applying this, the result is that the oxygen needed is 0.5 mg O_2/l. Looking at the results of the measurement of the oxygen in the water of both waterworks before filtration (8.6 mg O_2/l in the old plant and 10.2 mg O_2/l in the new plant) the conclusion is that there is enough oxygen in the aeration and the theoretical oxygen in the water after treatment will be 9.7 mg O_2/l in the old waterworks and 8.1 mg O_2/l in the new plant.

In conclusion both aeration methods work well and there aren´t any problems. The objective of this step is reached.

2.3.2 Sand filtration

<u>Filter design</u>
As all the concentrations of components are known, it is possible to see how the filters in these plants need to be designed.

The typical way to design the filter is considering the iron, manganese and ammonia levels (as seen in the water quality part, these are the components that exceed drinking water criteria). To decide which filter to use, the rules of thumb shown in Figure 2.8 [Kristensen, 2014] can be used as a basis depending on how high the contaminant concentrations are. The values to be used in order to determine the filter type are shown in Table 2.4.



Filtration method	Iron mg/l	Manganese mg/l	Ammonium mg/l
Single, quartz	< 2	< 0.1	< 0.5
Double, quartz	< 2	< 0.3	< 1.5
Single, 2-media	< 5	< 0.3	< 0.8
Double, 2-media + quartz	< 5	< 0.5	< 1.5
Double 2-media	< 8	< 0.6	< 1.5
	> 8	< 0.3	< 0.8

FIGURE 2.8 Rule of Thumb for Filtration Method [Kristensen, 2014]

TABLE 2.4 Levels of contaminants in waterworks

	Iron (mg/l)	Manganese (mg/l)	Ammonium (mg/l)
Old waterworks	0.54	0.45	0.06
New waterworks	0.417	0.582	0.07

According to the rule of thumb for both old and new waterworks the filter that must be used is the double 2-media filter. In reality, the filters are single and not double. This makes sense because the iron (which is normally the decisive component) and the ammonium levels are low so the single filtration is adequate. The only potential problem is the amount of manganese. As can be seen later in the report, this is not an actual problem since the table is only a guideline and not definitive.

Key parameters in the waterworks

The flow of water from each well is taken from the consultancy report [Vand og Teknik A/S, 2016], and the filter area is given in the layout provided by the waterwork.

The old waterworks receives the raw water from Boring 2. There are two filters with a total area of 9.3 m^2 as the plant manager said. The height is 1.55 meters and the flow is 27 m^3/h [Vand og Teknik A/S, 2016]. The sand that is used has a porosity of 26%. With this data, it is possible to calculate the key parameters.

The new waterworks receives water from Borings 1 and 4. It has 3 filters which can be calculated by looking the maps of Skovby waterworks and an area of 20.4 m^2. The height is 1.3 meters. Here there are two flows, 27 m^3/h from Boring 4 and 33 m^3/h from Boring 1. In this waterwork, it should be noted that the two wells never run in parallel, they just alternate. For the following calculations, it is better to use 33 m^3/h because is the most critical flow. Both of these flows are taken from the consultancy report [Vand og Teknik A/S, 2016]. The sand porosity is also 26%.

The first parameter is the filtration rate (Fr= Q/A). In this case the filtration rate is 2.8 m/h for the old waterworks and 1.62 m/h for the new one which are typical values. The next parameter is the empty bed contact time (EBTC = A*H/Q) which is the time that the water will be in the filter bed if it was empty. The final result is 33 minutes for the old waterworks and 48 minutes for the new waterworks. It is also possible to know the residence time (T_{res}=EBTC * porosity) which is the time that the water is actually in the filter bed. This time is 8.6 minutes in the old and 12.5 minutes in the new waterworks. [Maribo, 2017]

The function of the filter is mainly to eliminate the ammonium, iron and manganese. For the ammonia there is a rule of thumb that says every 5 minutes that the water is in the filter the NH_4 concentration reduces by half. Following this method and knowing that the concentration is 0.06 mg/l in the old waterworks and 0.07 mg/l in the new waterworks, thus the water must stay in the filter for less than five minutes to be below the criteria for both of them. This means there are no problems with the ammonium levels. [Maribo, 2017]

The iron and the manganese are retained in the filter so it is necessary to backwash. It is important to know when best to do this because if not the filter will stop working as it should. Looking in the yearly report of 2015 it can be seen that the raw water that goes into the waterworks during the whole year was 145725 m^3. The clean water that went out of the plant was 144047 m^3. The difference between these values is 1678 m^3 which is the amount of water that was used for the backwash in 2015. The plant manager said 1400m^3 of water is used for the backwash each year so the difference could be caused by some loss of water in the system.

To calculate how often to backwash can be done by looking at the time or volume between each backwash. To figure out the time between backwashing, use the concentration of iron plus the concentration of manganese multiplied by the flow and divided by the filter area. This gives an amount in kg/m^2 in one day. The rule of thumb says that the backwash must be performed when the filter contains 0.5 kg/(h*m^2) of iron and magnesium to find the equivalence and calculate the days [Maribo, 2017]. In the old waterworks the result was that backwashing needs to occur every 7.5 days and every 13 days in the new waterworks.

It is also useful to know the volume of water that has to be treated before backwash. The amount of iron and manganese in kg in 1 cubic meter are known and 0.5 kg per cubic meter is the desired level. With just a division it´s possible to know that in the old and waterworks 500 m^3 of water can be treated in each (or 1000 m^3 in total) before a backwash is needed.

The backwash should be done every 7.5 days or 500m^3 of water treated in the old waterworks (whichever occurs first) and every 23 days or 500m^3 of water in the new waterworks. The plant manager said that they did a backwash every 7 day or 1000m^3 of water treated. It appears that this is more often than necessary but the 0.5 kg/m^2 of iron and manganese is only a rule of thumb. It is a suggestion that for the new waterworks backwashing could be performed less often in order to save some water. An example of the backwash calculations can be viewed in Section 4.1.3 Possible complications and solutions (for Arsenic removal).

2.3.3 Capacity

It is important to analyse the capacity of the waterworks and if a higher water demand could be handled.

The old waterworks could potentially handle more but it wouldn´t make a lot of difference so it is better to study of the capacity of the new waterworks, as this gets water from 2 wells and has two pumps.

As previously mentioned in the new waterworks there are two pumps (one for each well) but they never run at the same time. If water consumption needs to be increased, an option could be to try to run the two pumps at the same time.

If the two pumps work at the same time the average flow will be 60 m/h (27 m/h + 33 m/h). With this flow, production could be increased by 35%, but could also be achieved by closing the old waterworks and continue with the same demand.

Now the next step is to see if with this new flow the ammonium could be removed and to see if there will be a large consumption of water for the backwash. Doing the same calculations as before the empty bed contact time will be 26.5 minutes and the residence time will be 7 minutes. The concentration of ammonium is 0.007 so 5 minutes is needed to remove it all. The ammonium won´t be a problem if production is increased.

Last thing to check is the time between backwashes. Applying the method described before this time will be around 7 days which is actually the time they use right now. This means the backwash consumption won´t be a problem. Thus it is definitely possible to expand the production in 35%.

The clean water tanks in the facilities have a volume of 750 m^3. As will be presented in Chapter 3.1, it can be seen that last year the consumption was 144047 m^3. With this data the conclusion is that these tanks could provide water for 2 days approximately if they were full and no water was incoming into the tank.

2.3.4 Risk assessment

This section is about analysing some problems that could occur and what the consequences are (if there is a high chance that they could happen).

The first situation that could happen is that one of the three pumps breaks down. If the pump for Well 2 breaks, the old water treatment plant will have to stop. This isn´t a big problem because as is described before, the new waterworks can only be used by running the two pumps in parallel. If one of the two pumps in the new waterworks breaks this also will not be a problem because normally they are run alternately so they could use one pump constantly while the other is repaired.

The algae growth is also a risk. The risk in the new waterworks is quite low because the aeration is performed by aeration basin, so the humidity is low (which is not a good condition for algae to grow). The risk in the old waterworks is higher but the pump in this well could just be stopped while they clean it and the two pumps in the new waterworks work in parallel.

Bacteria could be a high risk but in this case it is not very likely because the methane and hydrogen sulphide concentrations are very low, so the bacteria would not have enough energy source.

Conclusion: Both of the waterworks (the new and the old) work without problems, the backwash could be done less often but it isn´t a bad idea to do it every 7 days just in case. As the facilities could handle both pumps of the new waterworks working in parallel, they could increase the capacity and the risk is minimum.

3. WATER CONSUMPTION AND DISTRIBUTION SYSTEM

In this section water consumption of Skovby is analysed with data provided by the water work. The analysis is conducted based on the two pressure zones. Total consumption, difference between the zones, day and hour factors and a brief future assessment will be presented.

The work with MikeUrban will also be explained. All important aspects resulting from the simulation in the current system will be shown and analysed. The future consumption calculated in the consumption area will be incorporated into the analysis of the network through MikeUrban.

3.1 Water Consumption

3.1.1 Pressure zones

Skovby is divided into two pressure zones, as shown in Figure 3.1, with the circled location as the waterwork. As indicated by the names, the water in the pipes is kept at a constant pressure of 39 mWc in the high-pressure zone, and at 33 mWc in the low-pressure zone. Each zone is supplied by each part of the waterwork, with the east building connected to the high-pressure zone and west building to the low-pressure zone (Figure 3.2).

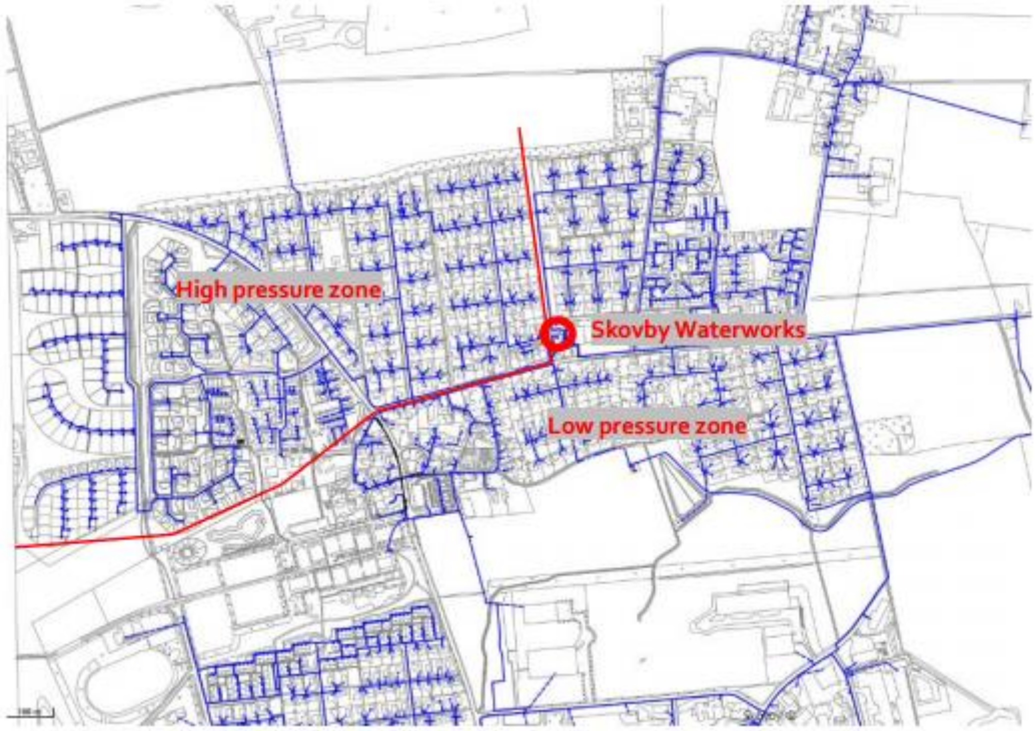


FIGURE 3.1 Two pressure zones of the water distribution network of Skovby [Fink, 2017]

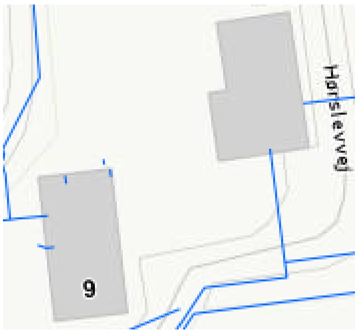


FIGURE 3.2 The two main buildings of the waterwork, each treating and supplying the high-pressure zone (left) and low-pressure zone (right) [Skanderborg Kort, 2017]

This division is supposedly due to the difference in consumer types and hence the water consumption between the two zones. This assists in assuring a constant and adequate supply of water to all consumers. The difference in terrain level matters as well, as the high-pressure zone has the elevation of 50 m above sea level and up while that of the other zone is less than 45m [Skanderborg kort, 2017]. Therefore, most of the data analysed in this section would be divided regarding the high and low-pressure zones as well.

As can be seen in Figure 3.3, the low-pressure zone is apparently much larger, indicating a possible higher water consumption. The high-pressure zone consists of mostly household areas (termed “living area” in the legend) whereas the low-pressure zone is of household, industrial (termed “business location”), recreation and public areas. The latter two could be areas with school, park or stadium, all regarded as “institutions” from now on. The unmarked grey area in the middle seems to be households, hence they are considered so.

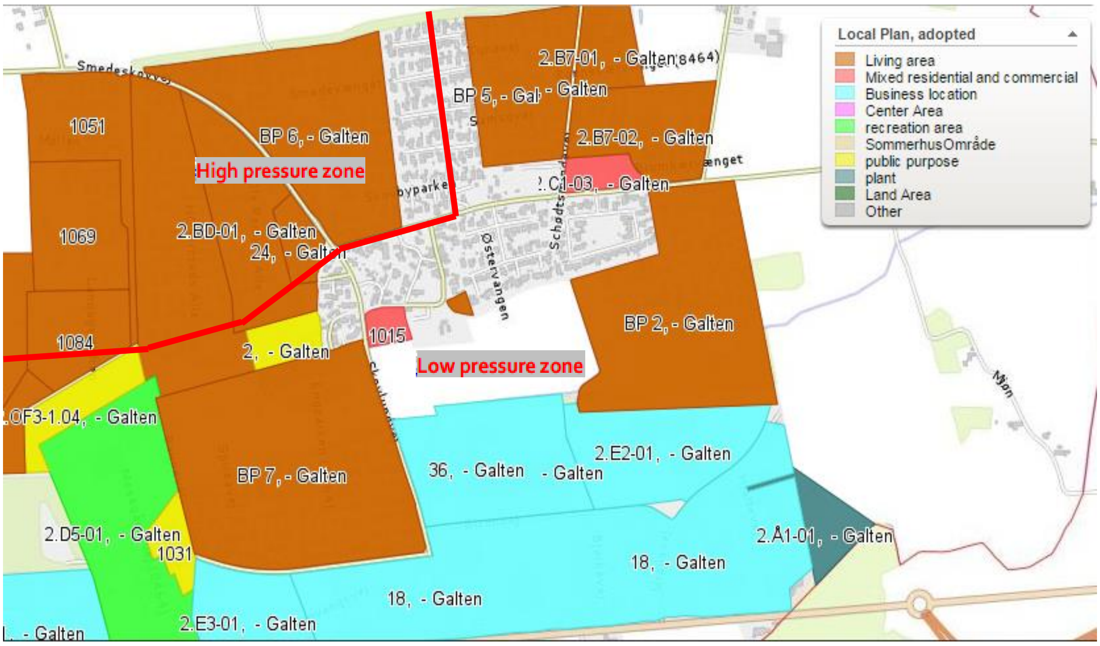


FIGURE 3.3 The two zones divided by purposes of specific areas in Skovby [Skanderborg Kort, 2017]

3.1.2 Consumer types

Water consumption was provided in the year of 2015 in terms of types of consumers. Unfortunately, this data is not divided into the two zones but rather as a whole of Skovby. The total consumption is 140166 m^3 and the division between different consumers is shown in Figure 3.4.

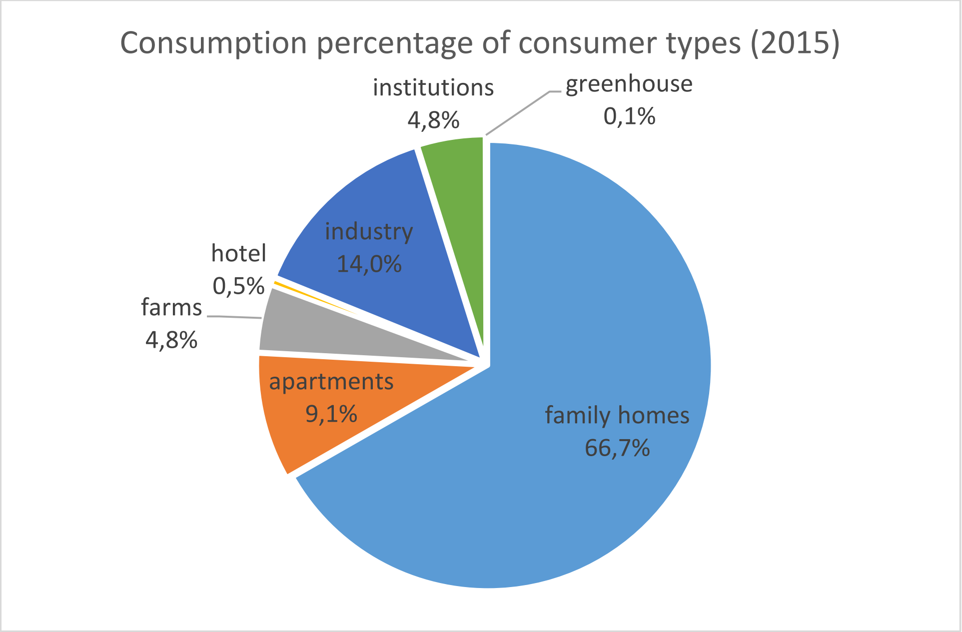


FIGURE 3.4 Percentage of water consumption of different consumer types in Skovby in 2015

Because specific consumption was provided, it was possible to calculate water demand at end nodes for the MIKE Urban model (the process will be described in detail in Section 3.2 Supply network). Households (family homes and apartments) account for about three-quarters of the total consumption, followed by industries at 14%. Each has their own average water consumption per unit. Most institutions are situated in the low-pressure zone, are non-residential and have different water use pattern and demand compared with industries, so they have their own average consumption as well. The greenhouse and the hotel account for little water use (as there is only one each in Skovby), hence they are negligible.

3.1.3 Yearly consumption

The amount of water going out to the two pressure zones is provided from 2013-2015 as in Table 3.1:

TABLE 3.1 Water supply [m^3] to customers of Skovby in the years 2013-2015 [Skovby waterwork, 2015]

	2013	**2014**	**2015**
Low pressure zone	101067	95518	93705
High pressure zone	48259	50147	50342
SUM	149326	145665	144047

Because the amount of water that was pumped out of the water work is closely related to the water demand from the area, analysing the former can give insight into the changes of the latter. Over this period, consumption in the low-pressure zone has been decreasing while that of the high-pressure zone has been increasing. However, the total consumption has lowered. As predicted, more water is consumed from the low-pressure zone, almost twice as much water than the high-pressure zone.

Apart from the data presented in the previous table, no other data of the year 2013 and 2014 was given. Therefore, from now on all the analysis will focus on the data from 2015. The water demand used for the MIKE Urban model in Section 3.2 is also based just on data of year 2015. From the yearly report 2015 provided by the water work, the recorded total amount of raw water is 145725 m^3, and the amount of clean water exiting the waterwork is 144056 m^3. The difference between the two numbers may be interpreted as treated water used for backwash and internal use in the waterwork, which is 1669 m^3, or approximately 1.1% of raw water abstracted. Moreover, from the water production and taxes report in the same year, the amount of water supplied to consumers was 144047 m^3, while the received amount registered at consumers was 140004 m^3, meaning 4043 m^3 (or 2.81% of supplied amount) was lost along the way in the distribution system. Part of this discrepancy could be due to the different accuracy of the water meters employed at the water work from those installed at individual households.

The flow data is divided into the two zones stated above, with a 2-minute interval between each measurement and unit is m^3/h. To calculate the average daily flow, the average value of all the flows recorded in each day was calculated and multiplied by 24 hours. After that the data was divided further into months, in order to show a clear picture of the fluctuation of water consumption over the whole year, as illustrated in Figure 3.5 and 3.6.

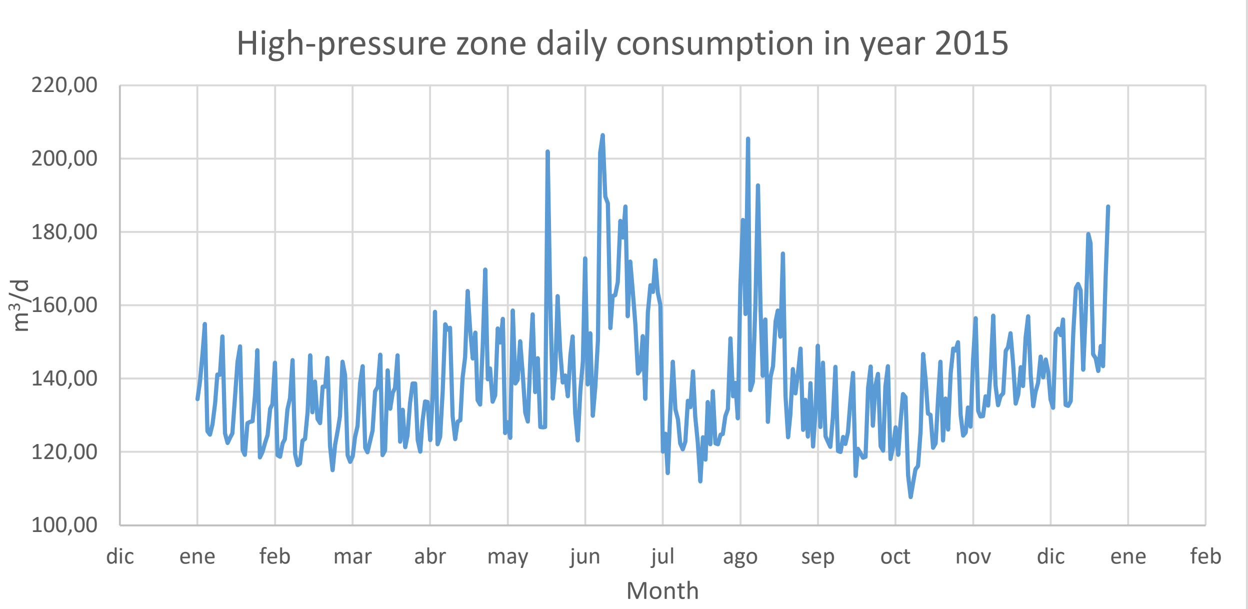


FIGURE 3.5 Daily water consumption divided into months in high pressure zone in 2015

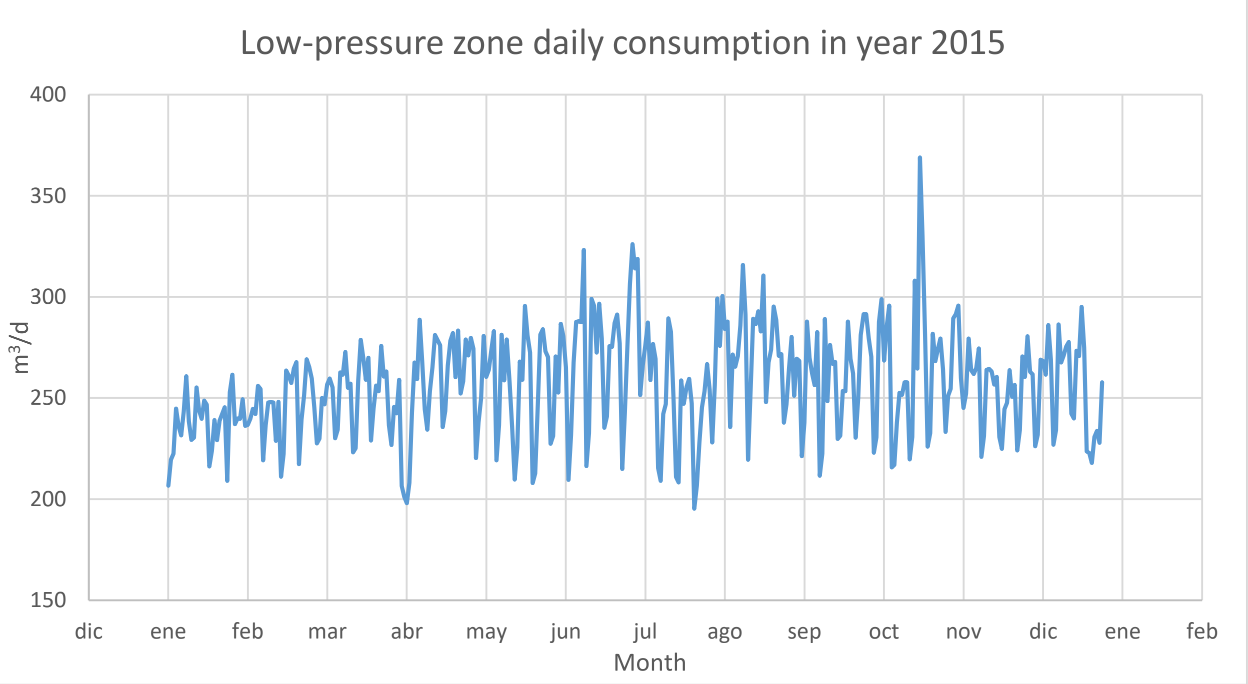


FIGURE 3.6 Daily water consumption divided into months in low pressure zone in 2015

The months with the highest consumption in high-pressure zone are June and August whereas those in low-pressure zone are June and October. This may be explained by the consumer types available in each zone. High-pressure zone is made up mostly by household areas, so the water use pattern would be more homogeneous than in low-pressure zone, where there are different customers with different demands and usage habits. Therefore, the water consumption in the latter would equal out among different users and produce a more averaged plateau pattern while that of the high-pressure zone fluctuates greatly and clearly throughout different months. For example, in high-pressure zone, the two highest usage months are June and August, both in the summer and during summer holidays. People naturally drink more water and engage in more water-involved activities in this season. The low consumption in July may be accounted for by people going out of town for vacation elsewhere. Another two months with higher-than-average consumption are April and December, when Easter and Christmas holidays occur, meaning more time and meals spending at home with families and hence, more water usage. Meanwhile, in the low-pressure zone, there is no clear pattern to be interpreted on a monthly scale. Also, because of the mix of housing and industrial areas, it is likely that when one consumer type ceases using water, another area's usage increases (for example, when people go to work, water usage would decrease at home and increase at work) and cancel out the difference.

3.1.4 Consumption variation and differences between the two zones

Two particular months were chosen to investigate the water consumption variation throughout a typical month and the difference between weekends and weekdays. These are June and November, since June is a typical summer holiday month and November a typical working month with no major holidays. Therefore, the pattern discovered would supposedly be the normal situation for most of the time.

First, it was found that people use more water on weekends in the high-pressure zone but vice versa in the low-pressure zone. In the former, Sunday is the highest consumption day in November and average-to-high in June while in the latter, Saturday is the lowest consumption day in both months. This is illustrated in Figure 3.7 and 3.8, where the mentioned days are accentuated with colour-filled markers. Possibly, when people leave homes (mostly located in high-pressure zone) and go to work (industries, mostly in low-pressure zone) on weekdays, this transfers the use of water from one zone to another and cause the difference in usage.

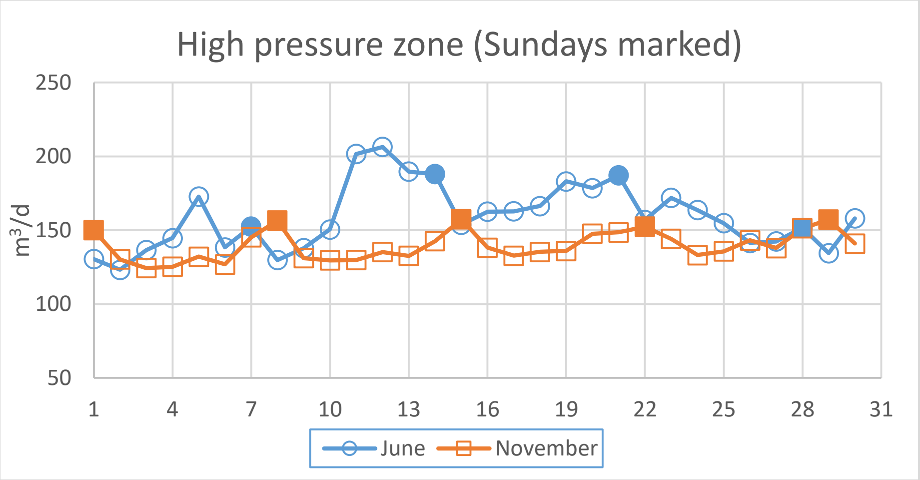


FIGURE 3.7 Water consumption variation through the months June and November 2015 in high-pressure zone (with Sundays accentuated)

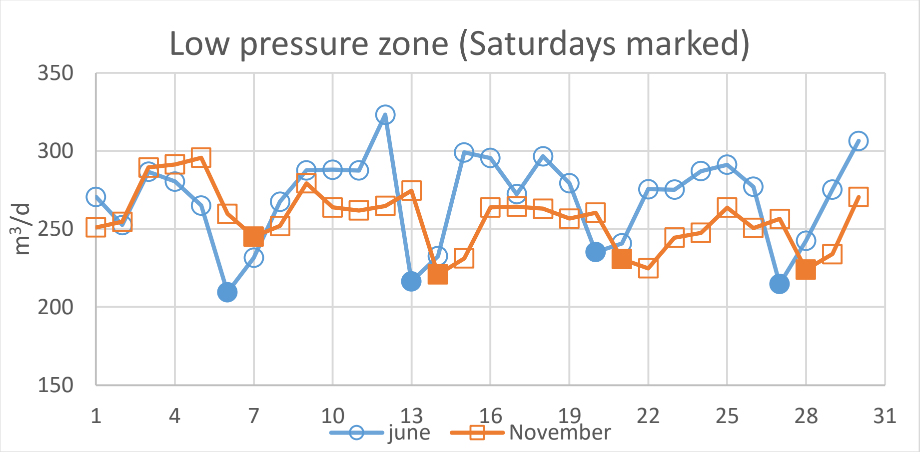


FIGURE 3.8 Water consumption variation through the months June and November 2015 in low-pressure zone (with Saturdays accentuated)

Hourly variation was averaged out for all Sundays and Wednesdays of the whole year without consideration of Daylight Saving Time just to show the difference of usage habit between weekend and weekdays (Figure 3.9 and 3.10). On Wednesday, the morning peak is between 7-8 a.m. in both zones and the low-pressure zone has another peak at 9-10 a.m., probably the time work starts. Then the high-pressure zone sees a dip in usage until 19-20 hour while the other zone has a somewhat constantly high water demand through the day. The usage pattern is much more similar between the two zones on weekend, when most people are at home. The morning peak is naturally later (at around 10-11), followed by a dip in the afternoon and the evening peak at 19-20. The morning and evening peaks correspond to when people wake up, have breakfast and go home, have dinner and shower.

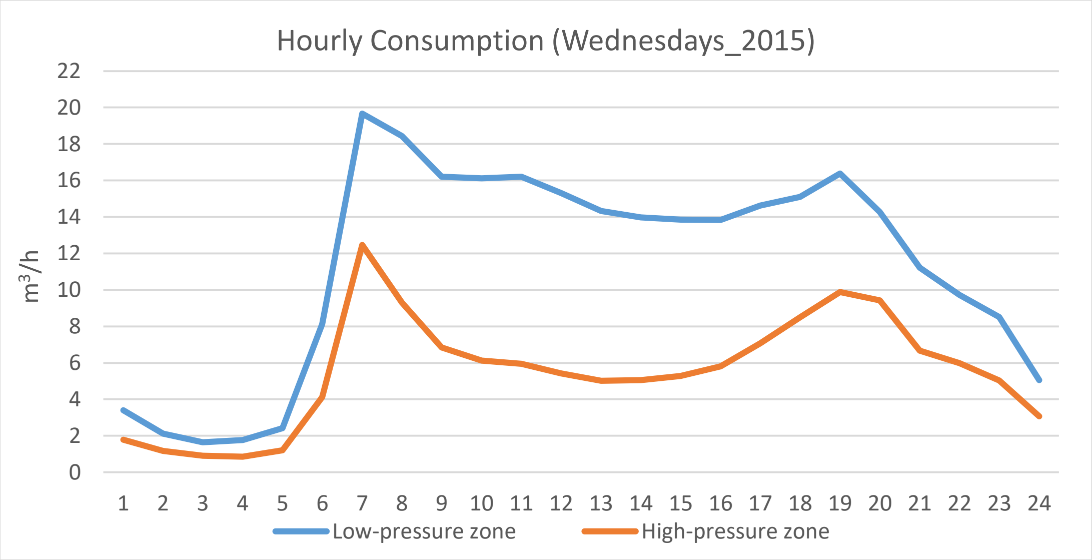


FIGURE 3.9 Hourly consumption of all Wednesdays in 2015 in the two zones

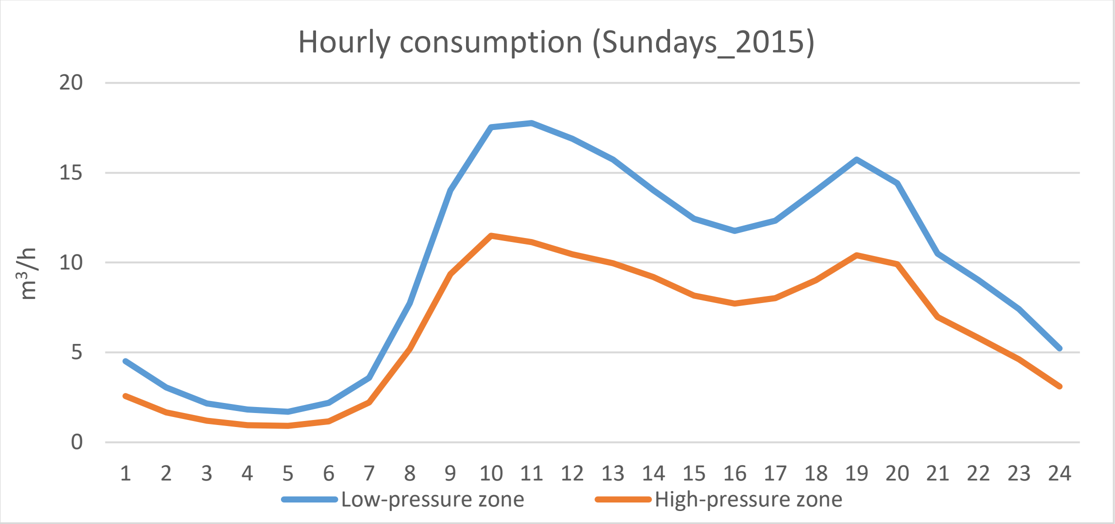


FIGURE 3.10 Hourly consumption of all Sundays in 2015 in the two zones

3.1.5 Daily and hourly factors

To calculate the day factor, the date with the highest average flow per hour was picked out from each month and this value is then divided by the average flow of the whole year, so that there is a day factor for every month. The average and maximum of all these day factors were then sorted (Table 3.2):

TABLE 3.2 Daily factor of the two zones (2015)

	Average flow of the year [m³/h]	Average day factor	Max day factor
High-pressure zone	5.8	1.23	1.49
Low-pressure zone	10.7	1.01	1.44

To calculate the hour factor, from each month the hour with the highest consumption was picked, then this value is divided by the average flow of the corresponding date and an hour factor was also produced for every month. The average and maximum of all these hour factors were then sorted (Table 3.3):

TABLE 3.3 Hour factor of the two zones (2015)

	Average hour factor	Max hour factor
High-pressure zone	2.73	4.89
Low-pressure zone	1.81	2.82

Theoretically, the factor describing the most extreme situation would be combined by multiplying the max day factor and the max hour factor. However, the maximum hour factor is too high and the hour during which the flow is highest does not necessarily falls on the day with the highest consumption (in either zone). Therefore, the max day factor and the average hour factor would be used.

3.1.6 Uncertainties

There were some assumptions made during the processing of the raw data provided. First, there are some missing values (presented as "???" in the original dataset) occurring around the middle of the month at 3 a.m. for around 8 minutes in both zones. One suggests that this might be caused by the measuring equipment resetting at a fixed time every month. Second, in the low-pressure zone, on February 5th from 7:54-8:34 (40 min), there is a constant outgoing flow at 100 m^3/h. One possible explanation may be pipe burst, and the measuring equipment maximum threshold is 100 m^3/h, so even if the flow is higher, it can only be registered as 100. Following this there are some values at around 90 m^3/h for the next 18 minutes (8:36-8:54). This may indicate the effort of fixing the pipe burst because after that the flow seems to return to normal range. Therefore, these 90 to 100 values may not represent the normal situation. Finally, the raw data includes some measurements of 31.12.2014, just before the New Year's Eve. But since only the data of 2015 were of interest, these values, along with the abnormal ones mentioned above, were removed before any calculation took place.

3.1.7 Future assessment

As mentioned, there seems to be a residential area (the black-framed part) under construction in the high-pressure zone. Since this area is already composed of mostly households, another residential area would not alter the pattern of water using significantly. The total consumption would surely increase, but the variation pattern would remain somewhat the same.

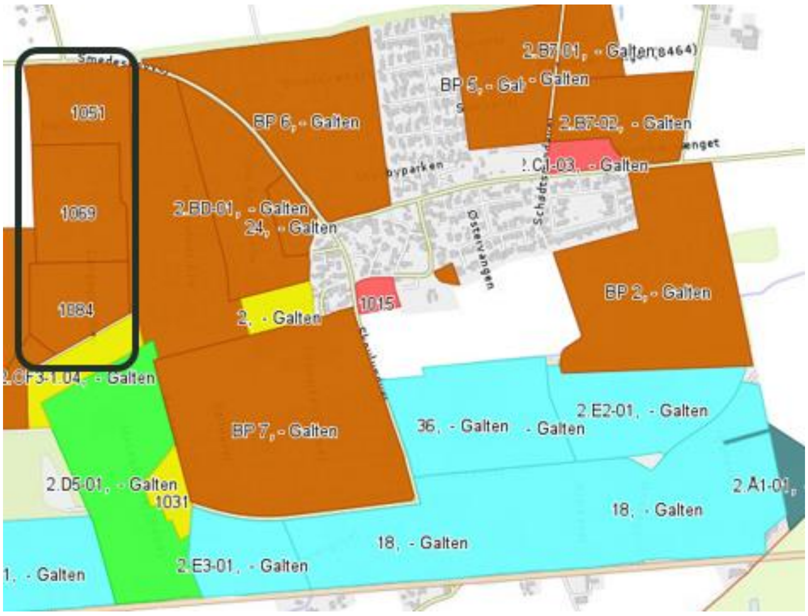


FIGURE 3.11 The under-planned residential area in the high-pressure zone [Skanderborg Kort, 2017]

This area consists of seemingly 44 constructed buildings and 31 planned buildings. This totals to 75 more households underway of being included to the water supply system. To calculate the possible water demand of the area in one year, the demand of a single household is needed. There are two options: either the average of 83.6 m^3/year (corresponding to 2.15 persons/household) suggested by DANVA (Vestergaard, 2017), or the value of 93.7 m^3/year calculated from the high-pressure zone data. The latter was decided as more appropriate, since it was generated from real data and more relevant to local situation. Therefore, the total consumption of the area would be: 93.7*75 = 7028 m^3/year, equal to approximately 5% of the total consumption registered at consumers in 2015 (140000 m^3), not a substantial amount. Moreover, when the layout of the distribution network was modelled in MIKE Urban, the information taken from the pipe system layer of the webGIS was updated with all the pipes laid out in this zone, hence it was assumed to already be a part of the supply system, not a future plan anymore.

It was then decided that future assessment would be more worthwhile if larger areas were involved. Therefore, an imagined scenario was devised involving the free area in northern Skovby (Figure 3.12). It is estimated to cover around 300 new houses, creating a demand of 93.7*300 = 28110 m^3/year (20% of the 140000 m^3 above). This area can be supplied by either the high-pressure or low-pressure zone distribution system or both. It will later be integrated in the MIKE Urban model as an outlet from the existing system. For detailed analysis of its impact, please refer to Section 3.2.2, part New Residential Area.

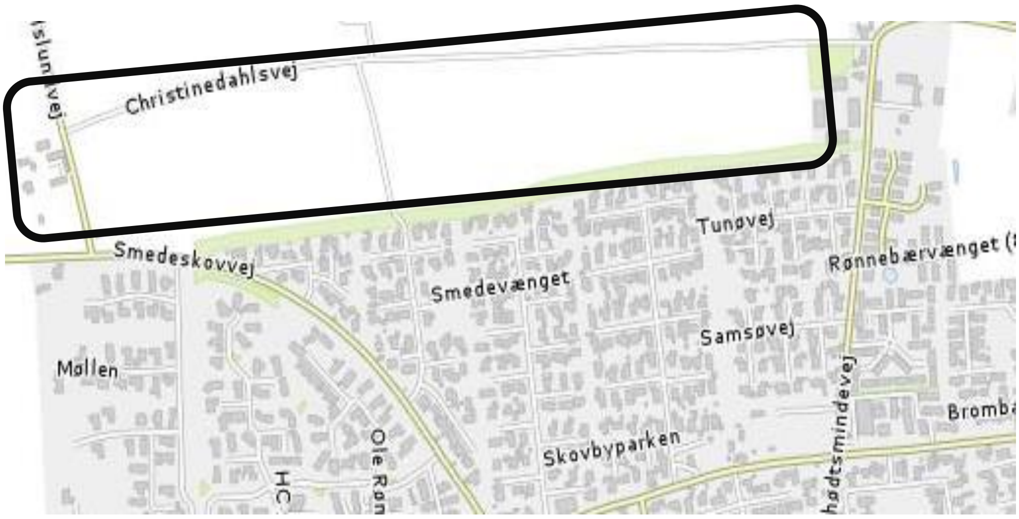


FIGURE 3.12 The possible future housing areas in northern Skovby [Skanderborg WebGIS, 2017]

3.2 Supply Network

3.2.1 Model through MIKE Urban

For modelling the current system MIKE URBAN – MIKE Powered by DHI Version 2014 was used.

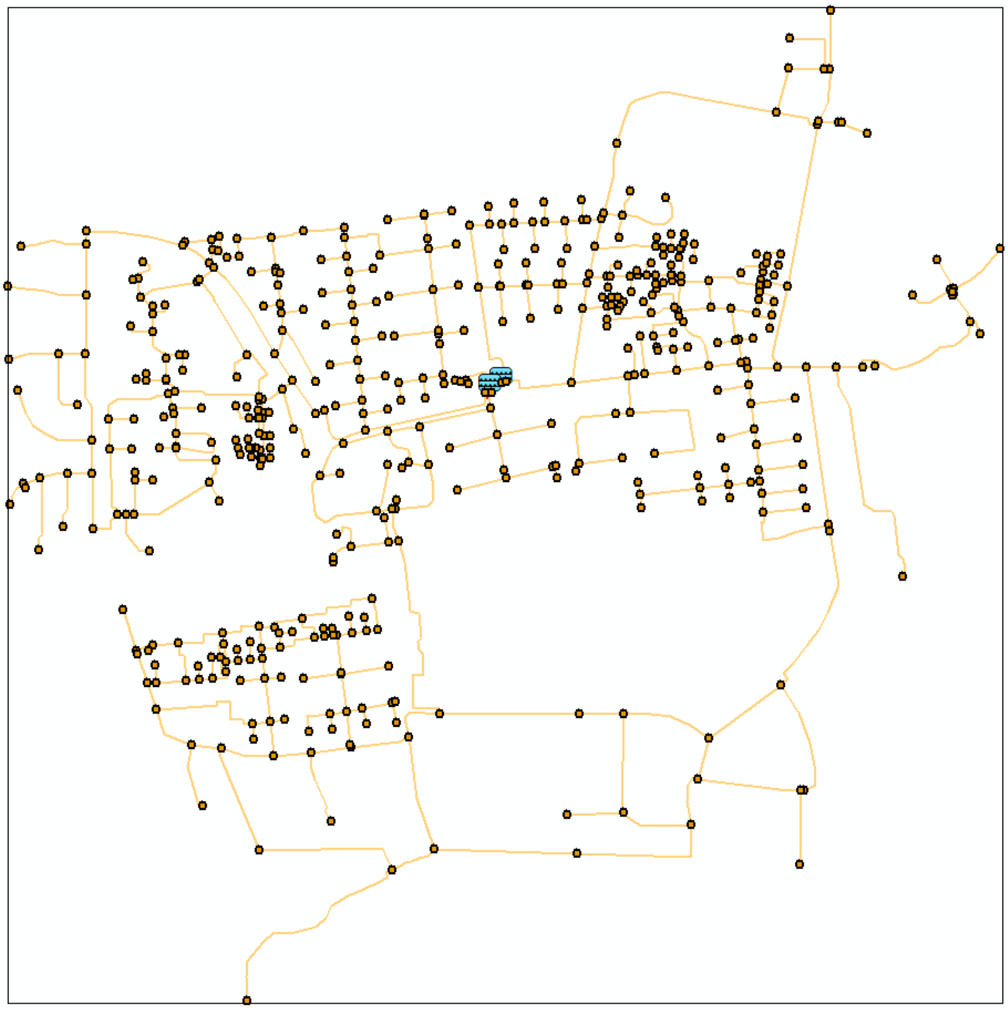


FIGURE 3.13 Picture of modelled system

The location of the pipes, their outer diameters, the location of the hydrants and other information related to the model was taken from the Skovby Water Supply Map [Skovby Map, 2017]. The pipes were assumed to be manufactured by Nordisk Wavin A/S, therefore the inner diameter was taken from them [Wavin, 2017]. Additional pipe diameters were taken from PlasticPipeShop Ltd. [Plasticpipeshop, 2017].

The simulation run time is limited to 14 days and the hourly factors for each hour within a day were used. This means that it simulates two entire weeks with the variation in consumption throughout the week. The first and eighth day would be a Sunday, the second and ninth day a Monday, and so on, ending with the seventh and fourteenth day as Saturday.

The model should be as accurate as possible but still there are some simplifications that were decided to be implemented.

Simplifications:

- Clean water tanks are represented as reservoirs with static water level instead of pumps
- Dead-end pipes with more sections with different diameters are assumed to have the highest diameter in all sections
- Total water consumption of each consumer category in 2015 was provided by the waterwork. From this and the number of consumers in each category in each zone, average demand for one consumer could be calculated in relation to their location and category. Due to the diversity of consumer types in low-pressure zone, the consumption of 1 hotel, 1 park and farms, were grouped under "household" for sake of simplification, which means that household demand of this zone would be much higher than the other zone. 10 institutions in this zone get a separate demand (each: 1906 l/d) due to reasons already stated in section 3.1.2 Consumer Types above. Average demand of a household in high pressure zone is 257 l/d while that in low-pressure zone is 425 l/d and industry (all in low-pressure zone) average demand is 708 l/d. Demands were grouped into nodes and the demand in each node was calculated with the average consumption and number of buildings corresponding to category.
- Hourly factors in the low-pressure zone not divided into industry hour factors and household hour factors. For a detailed cyclic profile see Appendix C: MikeUrban Section 1

Due to a lack of data, some assumptions had to be made. These are described as follows:

- Lowest pipe roughness is 0.005 mm, as default in MikeUrban 2014
- There are no local head losses within the system

3.2.2 Overall Processes and Critical Parts

Flow

During peak hours, where the consumption is the highest during a day, the flow is not very high. The flow in most of the system does not exceed 1 l/s, except for the main pipes and the pipe leading out of the low-pressure water works, which has the highest flow. The flow in the main pipes, as seen below during the morning peak hour on a Monday, range from 1 l/s further away from the water works to 3 l/s near the water works, with one pipe section higher. It reaches a flow slightly above 3 l/s. As written before, the pipe leading out of the low-pressure water works has the highest flow of all pipes. This pipe can reach up to 10 l/s in peak hours, as seen in Figure 3.14.

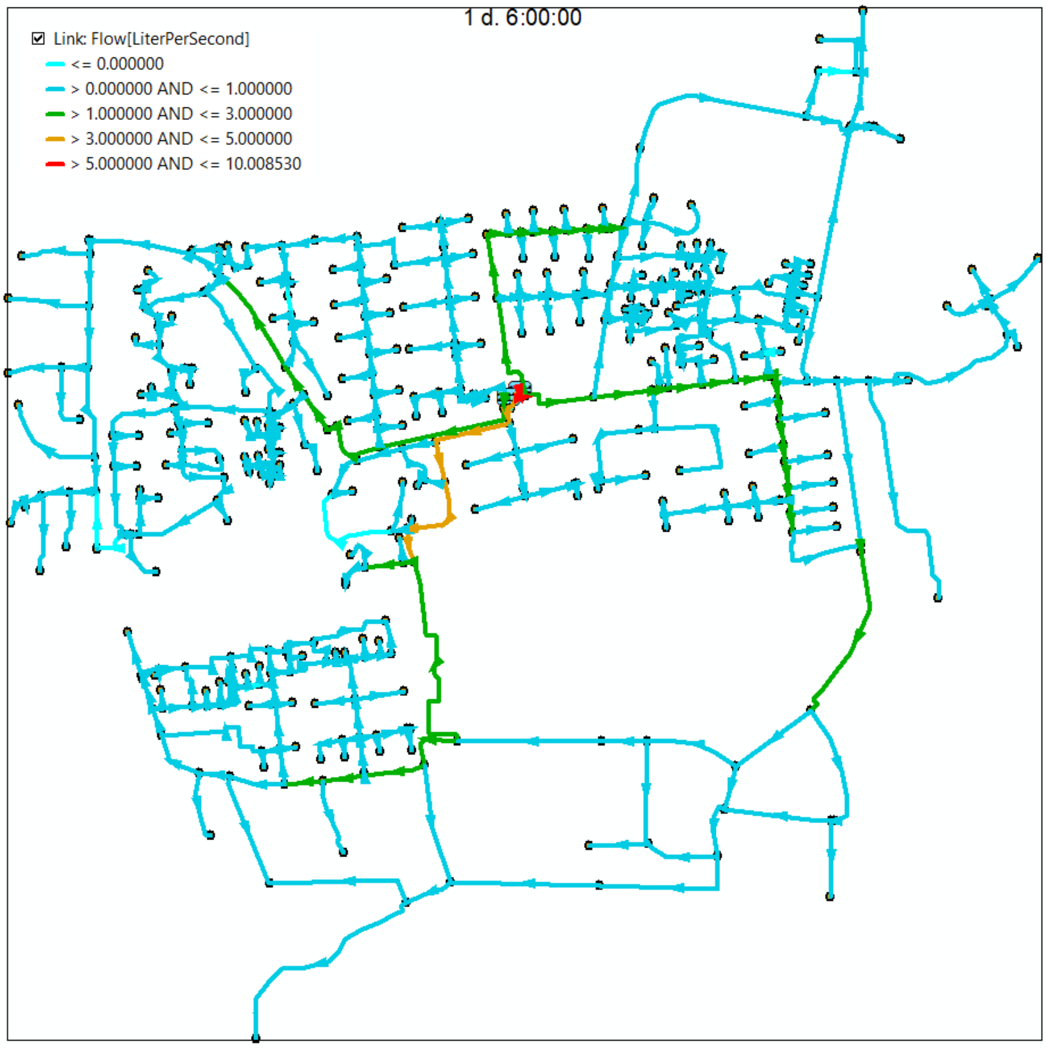


FIGURE 3.14 Flow during peak hour at Monday 6 am

Velocity

Similarly, to the flow, the velocity during the night with low consumption is very low, but it is also not high during the day.

The velocity doesn't exceed 0.3 m/s even in the peak hour. In the endings of the system the velocity is not higher than 0.05 m/s. Some of the main pipes have a higher velocity but compared to a typical system, with velocities around 1 m/s, they are still relatively slow. The velocity reaches up to 0.15 m/s, and in some pipes a bit higher. The highest velocity in the system can be found in the outgoing pipe of the low-pressure reservoir, in some main pipes and in some dead-end pipes. The high velocity in the dead-end pipes can occur because of some assumptions and simplifications that were made.

In conclusion, it can be said that the velocity in the entire system is very low. So, the system may be over dimensioned and that has an impact on the water quality, which will be discussed in further detail in the section "Water Age".

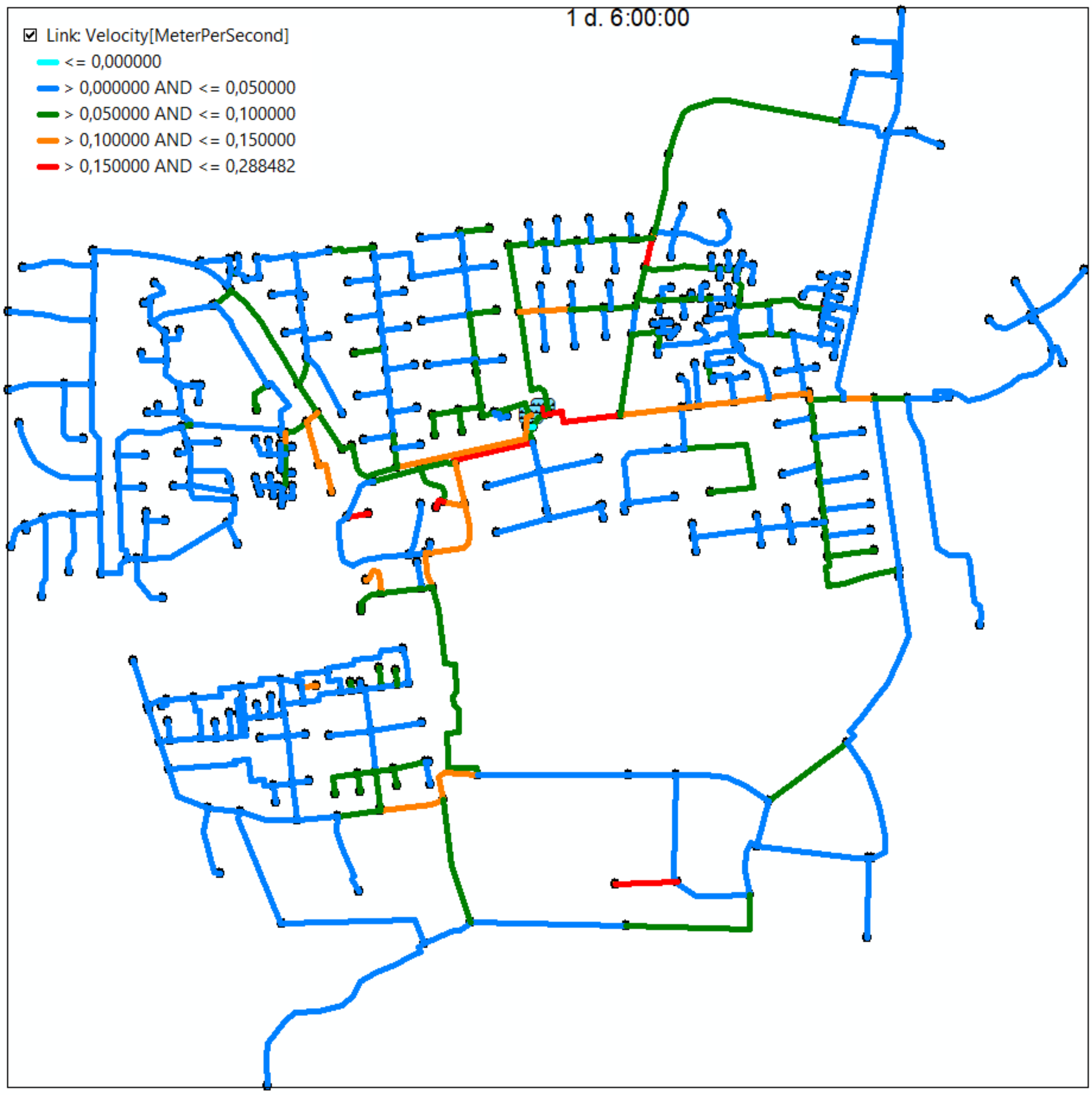


FIGURE 3.15 Velocity during peak hour at Monday 6 am

<u>Pressure</u>

The pressure depends on the pressure flowing out of the simplified reservoirs. The high-pressure zone has a supply of 87.5 m of pressure. The low-pressure zone is supplied by 81.25 m.

In general, it can be said that the pressure in the nodes doesn't vary much. It can also be said that the pressure during a normal consumption period is constant. All the nodes in the system also have the minimum amount of pressure necessary, which would be 20 metres of water column.

The pressure in the high-pressure zone is dominated by a pressure above 20 m and up to 35 m. In the beginning of the system the pressure is slightly higher. That can be explained because of the topography of Skovby, as towards the west, there is a higher elevation than in the rest of the city.

In the low-pressure zone, the nodes have a pressure around 35 m to 45 m of water column. As said before, the elevation here is lower and therefore the pressure is lower. In the north of the low-pressure zone are two nodes with a higher pressure than the rest of the system. The pressure here ranges from 45 up to 56 m. That is also because of the elevation. The nodes lie downhill with an elevation around 30 m. In comparison, the elevation of the other nodes ranges from 40 to 60 m, as seen in Figure 3.16 below.

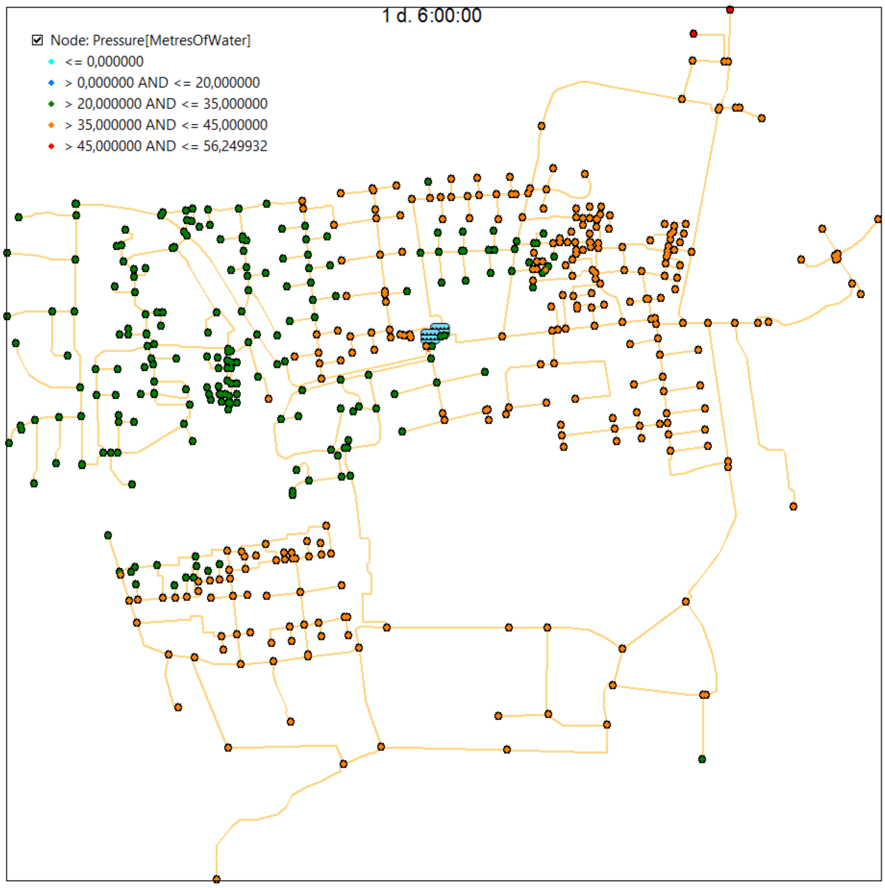


FIGURE 3.16 Pressure in nodes on Monday 6 am

<u>Water Age</u>

The water age after a simulation of 14 days is good even with the very low velocity through the entire system. The water age at the consumer points tends to be less than 24 hours old. At some spots, it may exceed 24 hours but is not older than 48 hours. Assuming that the age in the storage tanks is about 48 hours, there is a maximum of 72 to 96 hours (three to four days) of age at the consumer points, which is acceptable.

As seen in Figure 3.17 the system has three nodes connected with short pipes to the main supply system with a very high water age. Two of those nodes are located within the west of the high-pressure zone and one is located in the east of the low-pressure zone. In these areas, the water age is exactly as high as the simulation runs, that means that there is stagnant water. These points shouldn't be too critical because there is no demand. It would be better to do something about the pipes so that there is no stagnant water, but currently no consumer will receive this old water so it is not urgent.

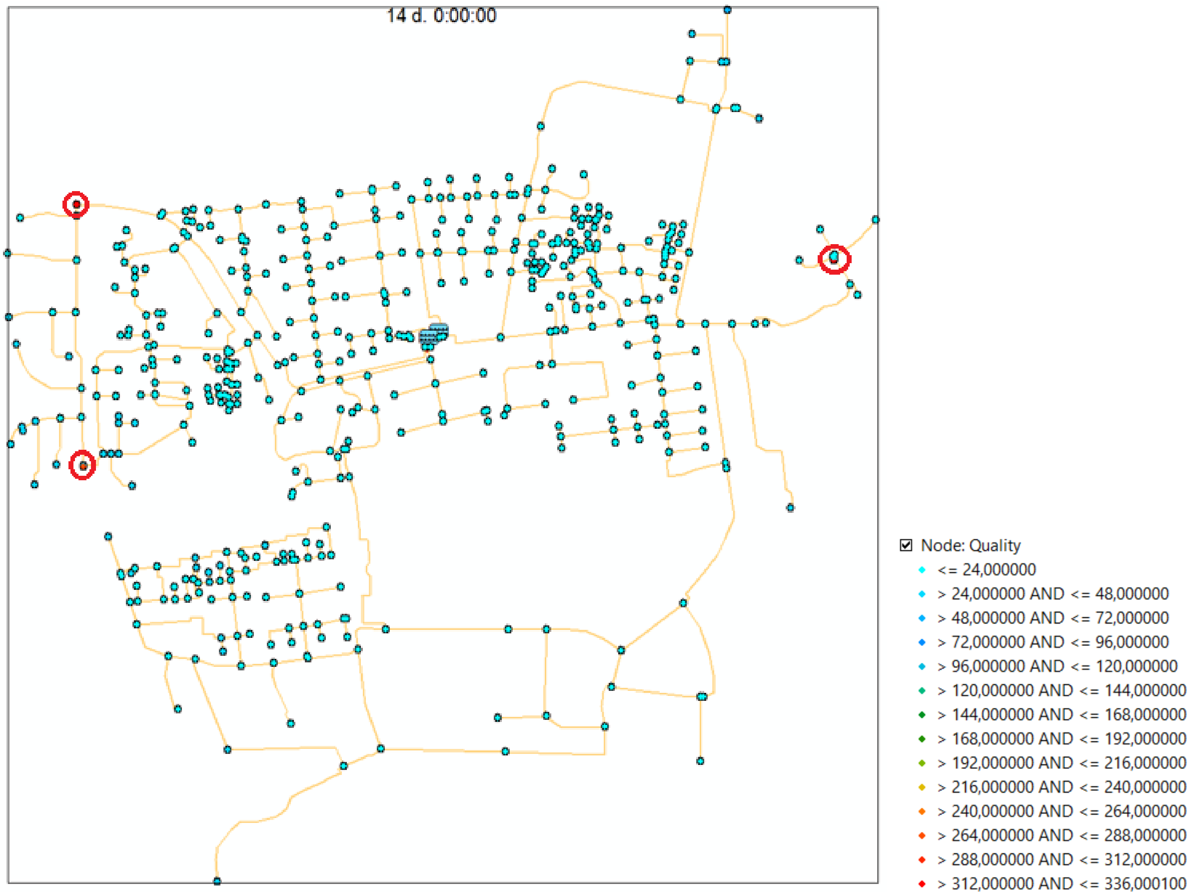


FIGURE 3.17 Water age in hours after a 14-day simulation

Fire Flow

In total, the system has 14 hydrants. Four located in the high-pressure zone and ten in the low-pressure zone. A typical pressure for a hydrant would be at least a pressure of 15 m [DHI 2014].

For a case of a fire within the system, it was assumed that the fire flow would be the highest, which is 25 l/s [Vestergaard, 2017]. The calculation also considers that only one hydrant at a time is active. For this simulation, it was found that four of the 14 hydrants have a negative pressure. That means that the risk of dirty water getting sucked into the system through leakages in pipes is present. These hydrants are therefore not usable with a fire flow that high. The hydrants are numbered 2, 3, 4 and 13 as seen in Figure 3.18.

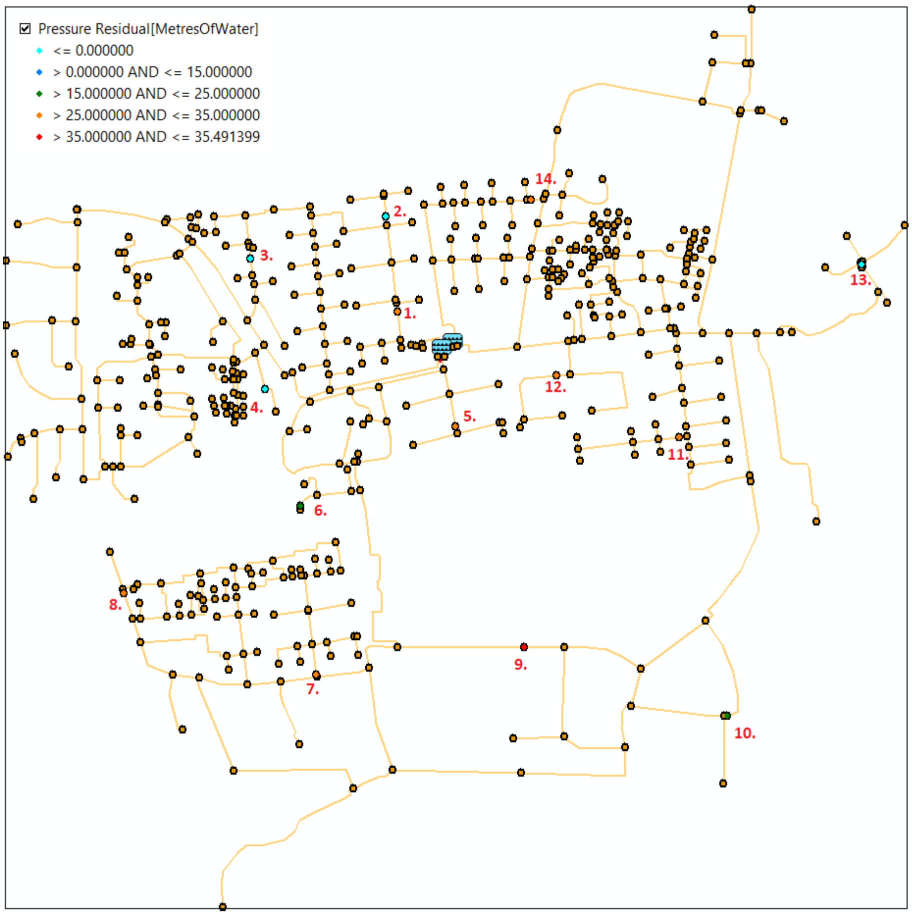


FIGURE 3.18 Fire Flow with 14 hydrants and 25 l/s simulating only one hydrant active

Because of some hydrants with negative pressure it was simulated a fire flow with a minimum of 14 l/s [Vestergaard, 2017]. The result is that only three out of 14 hydrants now have a negative pressure. Those are hydrants 3, 4 and 13. This can be explained that these three hydrants are connected to pipes with a very low diameter (50mm and 63mm). Because of the diameter, the velocity in the pipes is very high, resulting in a very high head loss, which explains the massive pressure drop.

Because of the negative pressure, these hydrants (2, 3, 4, 13) can be considered as critical points, which will be discussed in the next point.

Critical Parts

As written before the critical parts in the system are the hydrants with negative pressure during normal fire flow. Therefore, the system was simulated with different flows for these hydrants to see how high the flow can be without resulting in a negative pressure. It was also considered that nearby nodes will not have a negative pressure to avoid the risk of sucking in dirty water. The assumption is that 5 m [Vestergaard, 2017] of pressure in nearby nodes should be enough to avoid this risk. The pressure for hydrants to work is still said to be 15 m.

The results of the simulation with different flows are as following:

- Hydrant 2 usable with a fire flow of max. 16 l/s

- Hydrant 3 usable with a fire flow of max. 7 l/s

- Hydrant 4 usable with a fire flow of max. 9 l/s

- Hydrant 14 usable with a fire flow of max. 10 l/s

There is also the consideration to relocate some of the hydrants to avoid the high velocity in small pipes which will cause the pressure drop. It would be logical to replace at least hydrant 3 because this hydrant is connected to the smallest pipe with 50 mm outer diameter and has a huge impact on the surrounding. There is also a suitable alternative pipe with an outer diameter of 200 mm around 100 m south of the hydrant. The new hydrant will also be able to maintain a fire flow of 25 l/s without any negative pressure.

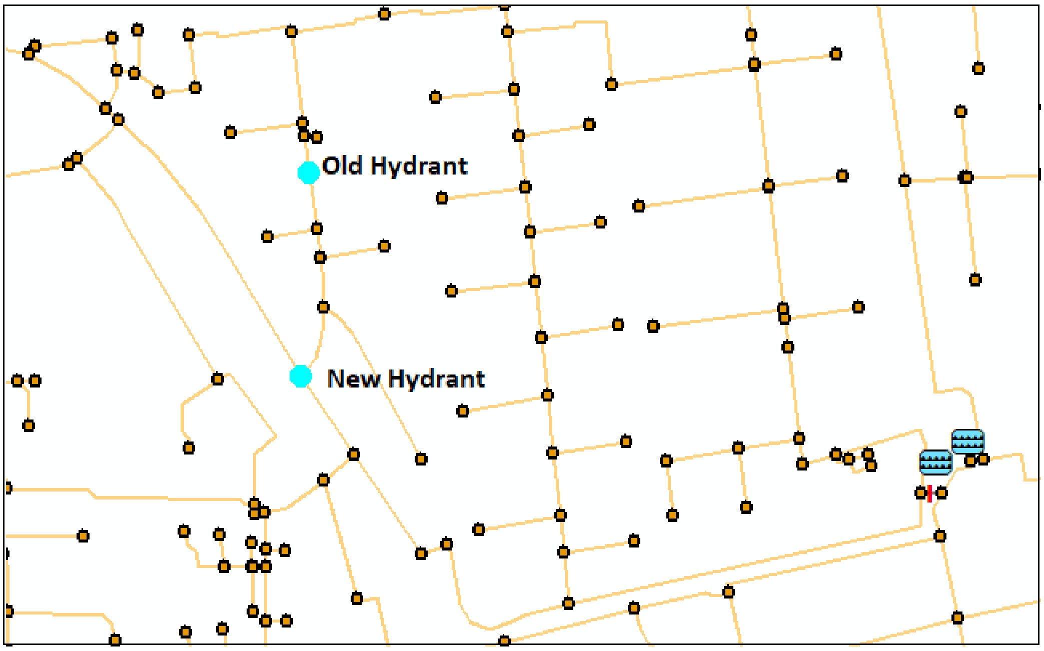


FIGURE 3.19 Location of current (old) and recommended (new) hydrant 3

<u>New Residential Area</u>

There is also a simulation that was run just for the purpose how the current system reacts if there would be a new residential area in the north of Skovby.

The new area is assumed to have 300 new houses that results in a demand of 0.9 l/s. It is also considered how the network reacts if the new area is connected to either the high- or low-pressure zone. The pipe diameter is assumed to have 200 mm outer diameter with an inner diameter of 182.6 mm. The pipe connected to the high-pressure zone is 350 m long and the one connected to the low-pressure zone 700 m. The connection to the existing system was chosen to be along existing streets.

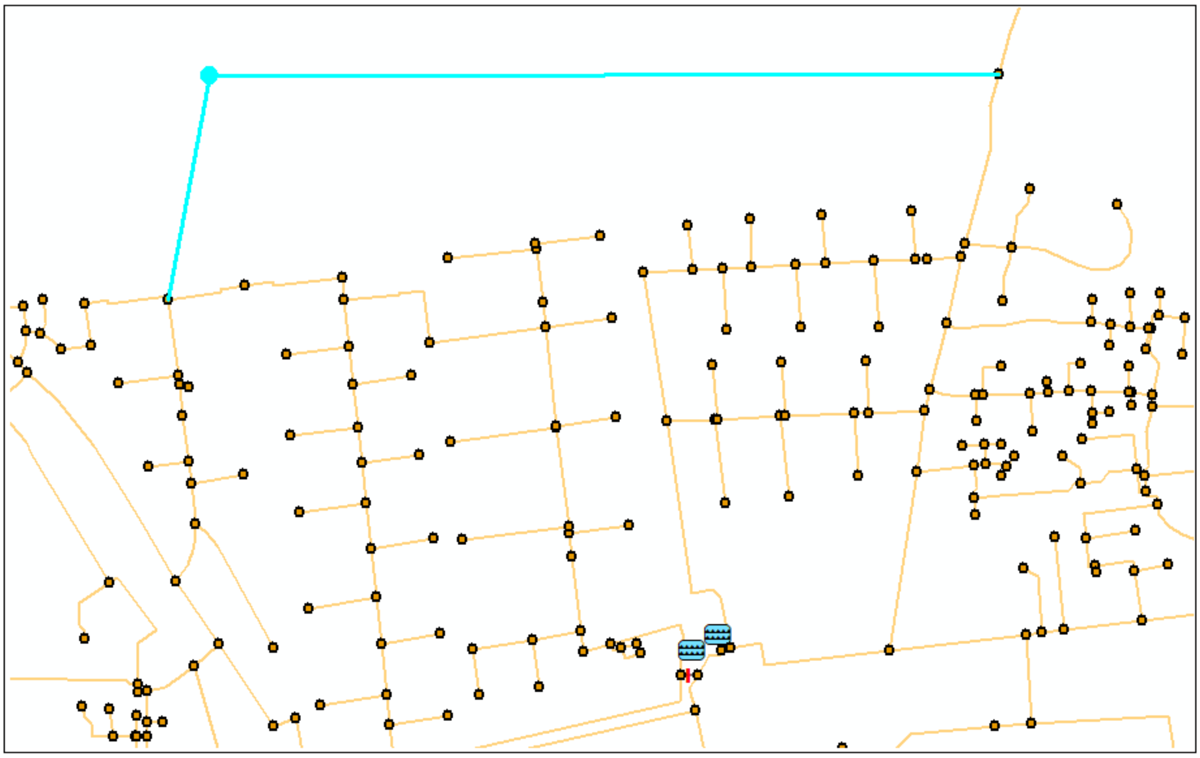


FIGURE 3.20 Location and connection of the new residential area

If connected to the high-pressure zone, it could be seen that the flow is slightly higher in the entire system during a peak hour. The main pipes leading to the new area would have a flow of 3.6 l/s instead of 2.5 l/s without the new area but this change isn't dramatically. The flow in the new pipe would be 1.99 l/s [Appendix C: MikeUrban Section 2]

The velocity would also be slightly higher but still not very significant. The change in the main pipes would be around 0.05 m/s (from 0.09 to 0.14 m/s) but this is still very low. The velocity in the pipes leading to the new residential area are very high compared to the situation without the new area. Here it can be seen that the velocity jumps from 0.02 m/s to 0.46 m/s as seen below in figure 3.21. It would be recommended to exchange the pipes if the new area would be connected to this point. In the new pipe, there would be a velocity of 0.08 m/s.

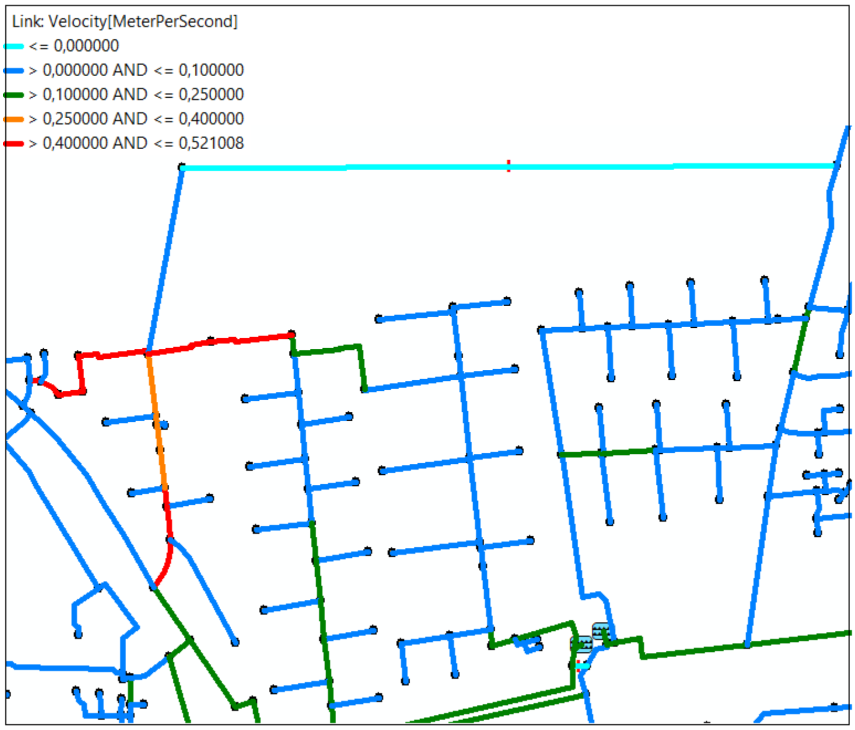


FIGURE 3.21 Location of critical pipes if connected to the high-pressure zone during peak hour at Monday 6 am

The pressure in the entire system doesn't change much and is still above the minimum of 20 m. The new area would have a pressure around 43 m [Appendix C: MikeUrban Section 3]

If connected to the low-pressure zone, it could be seen that the flow in the main pipes leading to the new area would rise a bit. The flow would jump from 2.26 l/s to 3.38 l/s which is not very high. Besides that, some pipes would have a change in flow direction. These pipes would be in the east of the existing system. The new pipe for the new area would also have a flow of 1.99 l/s [Appendix C: MikeUrban Section 4]

The velocity in a peak hour wouldn't change much. The pipe connected to the new pipe would have a higher velocity. It would rise from 0.08 m/s to 0.28 m/s [Appendix C: MikeUrban Section 5]

The pressure of the entire low-pressure zone would be almost the same. There is no significant change. The pressure in the new node would be around 38 m [Appendix C: MikeUrban Section 6]

With a new area in the north, it can be said that it is doable. The impact on the current system is not very high, but it is recommended to connect the new area through the low-pressure system because of the pipes with a high velocity in the high-pressure system. As said before, if connected to the high-pressure system these pipes should be replaced because there could be a problem if the new area would expand more and therefore the demand would rise. For the safety of the supply both pipes should be installed, so if one pipe must be closed, the area would still be supplied with water through the other pipe.

3.2.3 Sensitivity Assessment

Some assumptions mentioned earlier were changed to see what kind of impact these changes will have on the current system.

Assumption:

Roughness changed from to 0.005 mm to 0.025 (maximum for plastic - as default in MIKE Urban)

There was no significant change in flow, velocity or pressure found. The only difference here would be the fire flow. With a roughness coefficient of 0.025 instead of 0.005 hydrant 2, 4 and 13 need a different fire flow as seen below.

- Hydrant 2 usable with a fire flow of max. 15 l/s instead of 16 l/s
- Hydrant 4 usable with a fire flow of max. 8 l/s instead of 9 l/s
- Hydrant 13 usable with a fire flow of max. 9 l/s instead of 10 l/s

In conclusion, it can be said that the roughness doesn't have an enormous impact on the system. The reason for that is the very low velocity.

Assumption:

- Local head losses changed from 0 to 0.1
- There were no significant changes in flow, velocity, pressure of fire flow found. It can be said that the local head losses of 0.1 don't have any impact on the system that should be looked at.

3.2.4 Supply Security and Safety

To check if the system is still able to supply sufficient water to all areas it was decided to simulate various incidents like a maintenance in main pipes or a closed water works.

The results of three main aspects (flow, velocity and pressure) were considered.

The decided incidents are like following:

I. High-pressure waterwork not running

The only changes in flow occur in the high-pressure zone. Some pipes now have a slightly higher flow than before. The main pipes reach flows of 5 l/s instead of 3 l/s like before. Some pipes will have a change in flow direction (the light blue pipes in Figure 3.22). These are located north of the high-pressure waterworks, but this can be explained, because the pipes leading out of this waterwork are closed to simulate a shutdown of this waterwork.

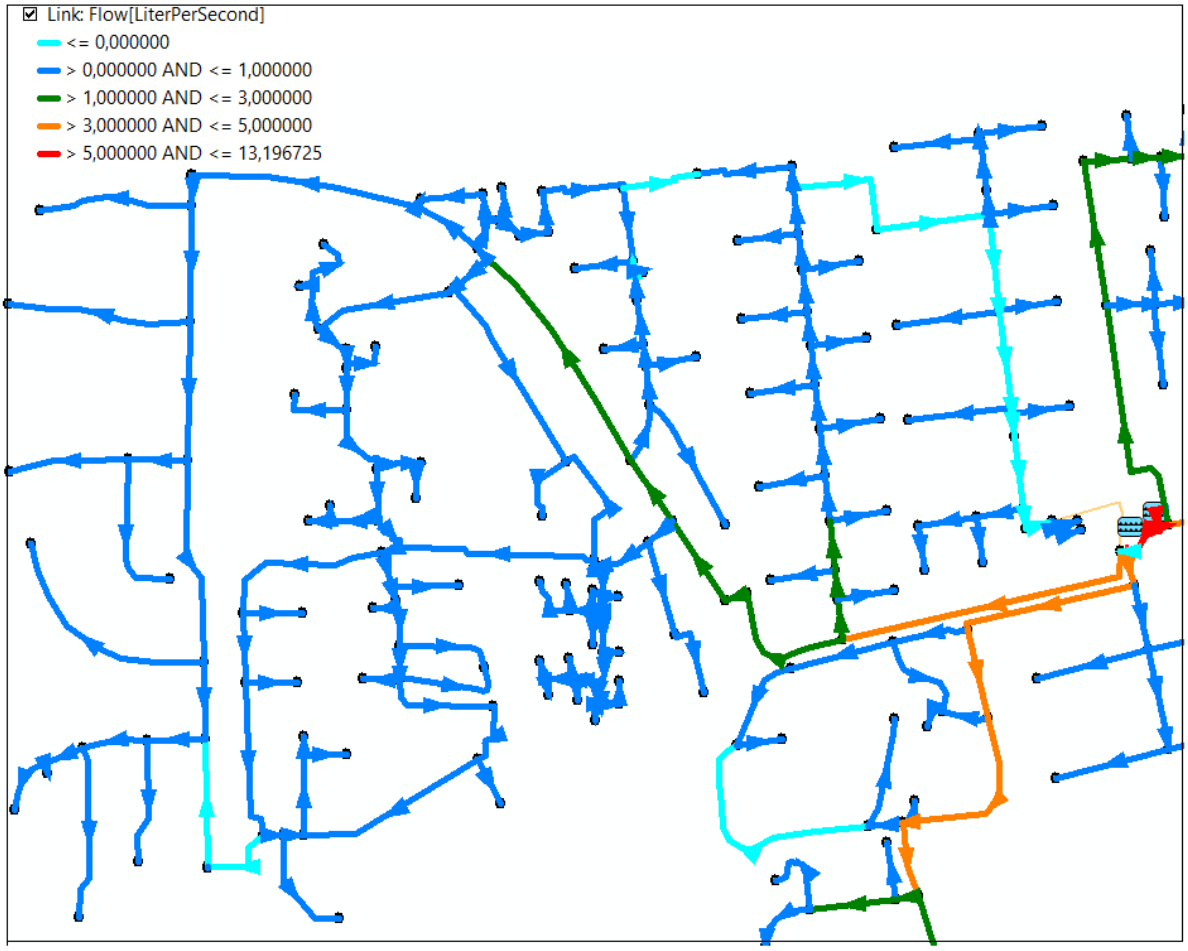


FIGURE 3.22 Flow after high-pressure waterworks shutdown in peak hour at Monday 6 am

The velocity is slightly higher but not significant.

The pressure is dropping because of the lower pressure coming out of the low-pressure waterworks. All nodes will have a lower pressure that is around the difference of the two waterworks, which is around 7 m of water column.

This is not very critical but now there are three nodes in the western part of the high-pressure zone (circled in red) which have a pressure slightly below the minimum of 20 m (around 18 m and 19 m). The consumer will still have water but it will not have the usual pressure and may run slower.

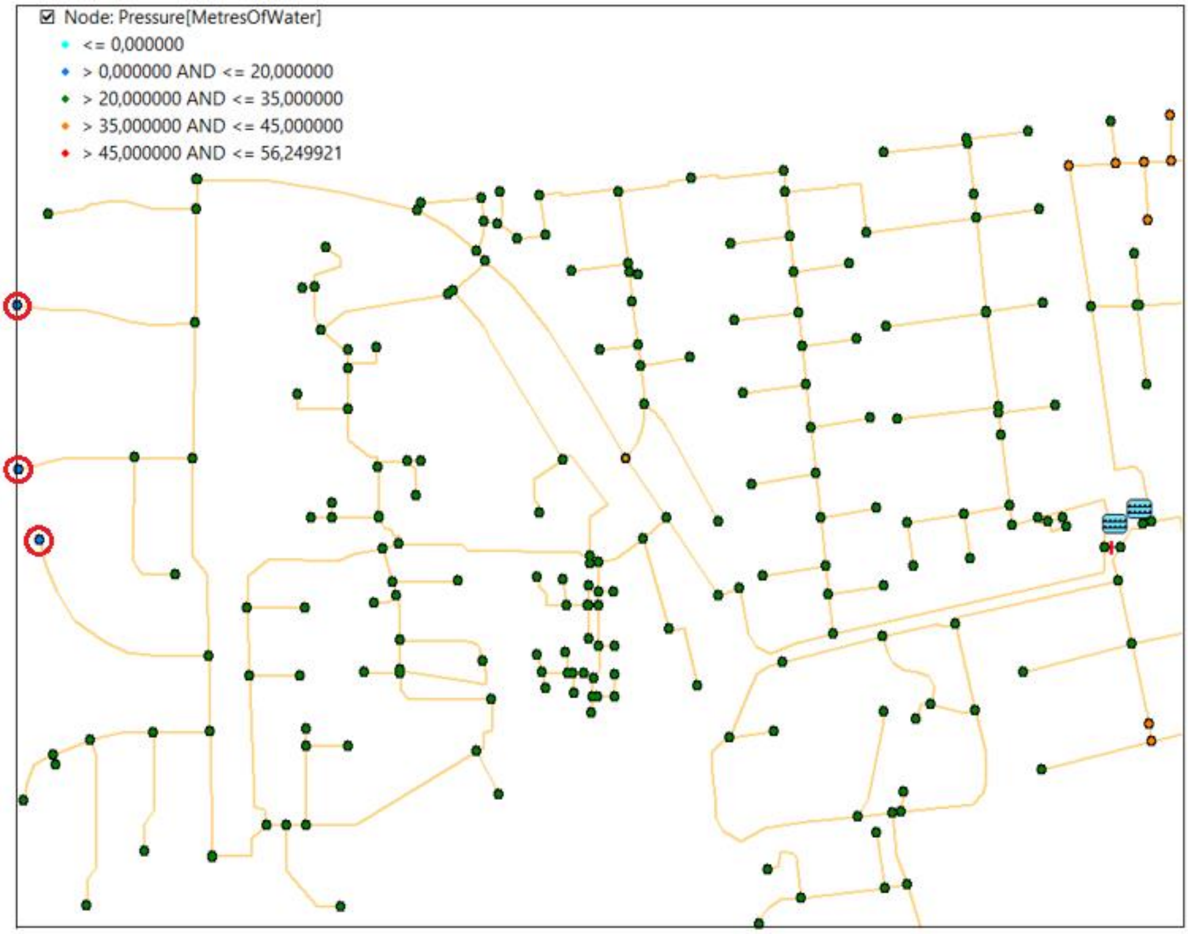


FIGURE 3.23 Pressure after high-pressure waterworks shutdown in peak hour at Monday 6 am

II. Low-pressure waterworks not running

There is no significant change in the water flow.

The velocity is slightly higher than a normal situation. The maximum is now around four times than in a normal situation. The highest velocity is located in the usually closed pipe connecting the high- and low-pressure zone. The velocity there is around 1.17 m/s.

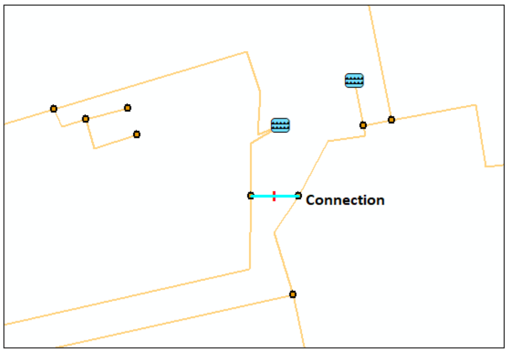


FIGURE 3.24 Location of the connection between both zones

Because of the higher pressure coming from the high-pressure zone the entire low-pressure system will have a higher pressure. Exactly the opposite as before. The maximum now is around 62 m because of the downhill topography in the north of Skovby.

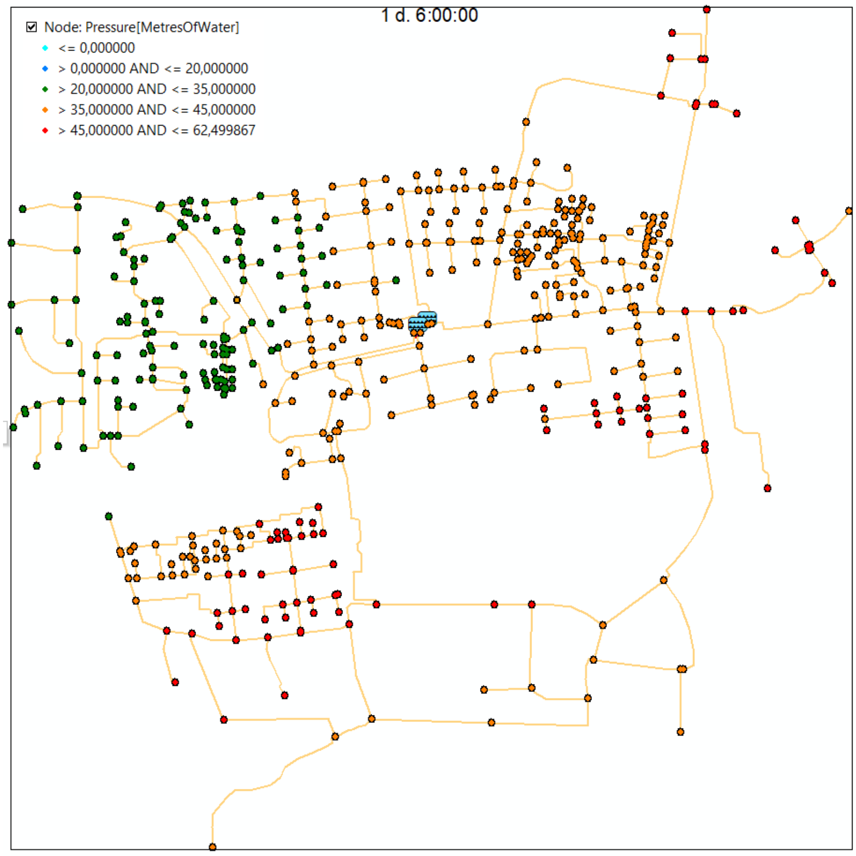


FIGURE 3.25 Pressure after low-pressure waterworks shutdown in a peak hour. Monday 6 am

III. Closed main pipe I

The closed pipe is located in the low-pressure zone and is slightly south of the waterworks.

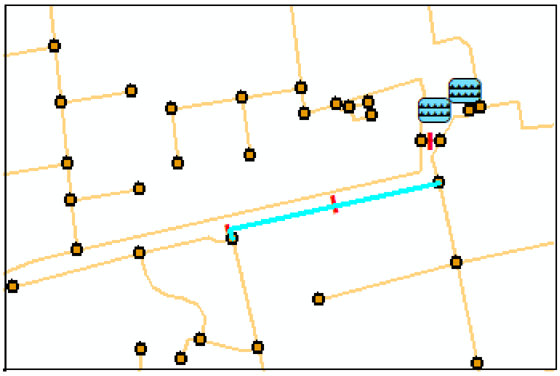


FIGURE 3.26 Location of the closed pipe

The flow is higher than before. Especially in the main pipe leading towards the cut off area, but it is not considered as significant.

The velocity is higher in the entire system than before. Also, like the flow, it is especially higher in the main pipe leading to the cut off area. From 0.31 m/s in the beginning to 0.23 m/s later in the system.

The pressure doesn't have any differences compared to the normal situation.

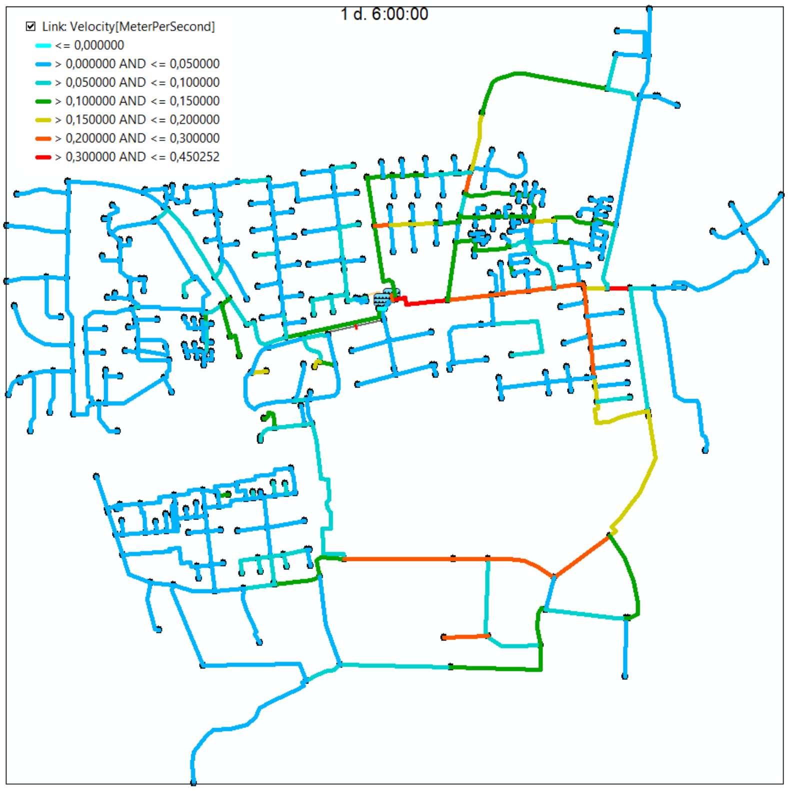


FIGURE 3.27 Velocity after closing a pipe during a peak hour. Monday 6 am

IV. Closed main pipe II

This closed pipe is the bigger pipe leading out of the high-pressure waterworks.

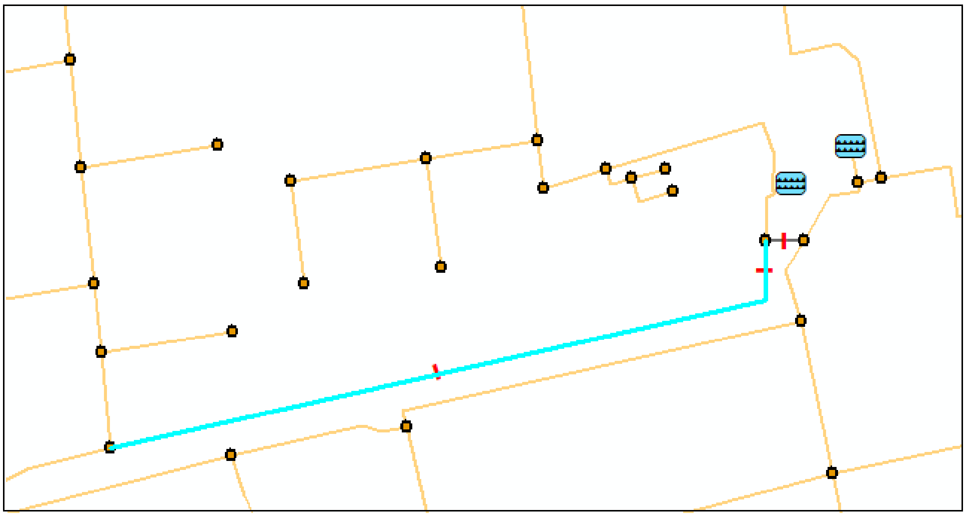


FIGURE 3.28 Location of the closed pipe

The water will now flow the upper way to the rest of the area. The flow is therefore higher than in the normal situation, but is it not very significant. The water runs up to 3.5 l/s.

In general, the velocity is slightly higher but not very critical till the water reaches a very small pipe. With an outer diameter of 50 mm the pipe is too small. The velocity in this pipe is up to 1.8 m/s during a peak hour as seen in Figure 3.29. Because of this speed there is also a high head loss resulting in this pipe.

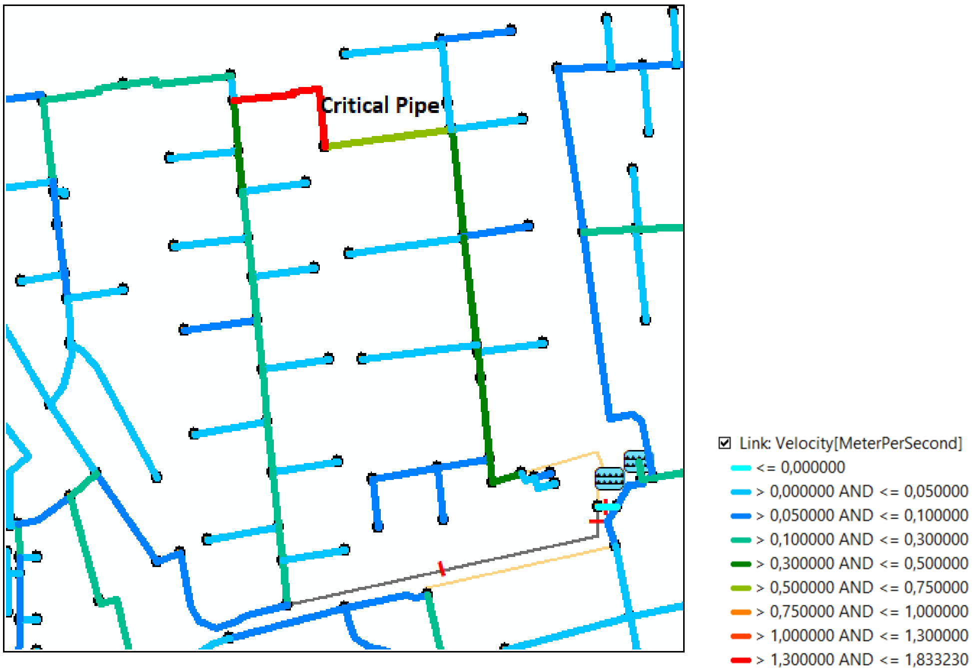


FIGURE 3.29 Location of the critical pipe

Because of the high velocity and resulting head losses in the small pipe the system loses a lot of pressure. The losses are too high to maintain a normal pressure above 20 m in the entire western area of Skovby (as can be seen by the blue nodes).

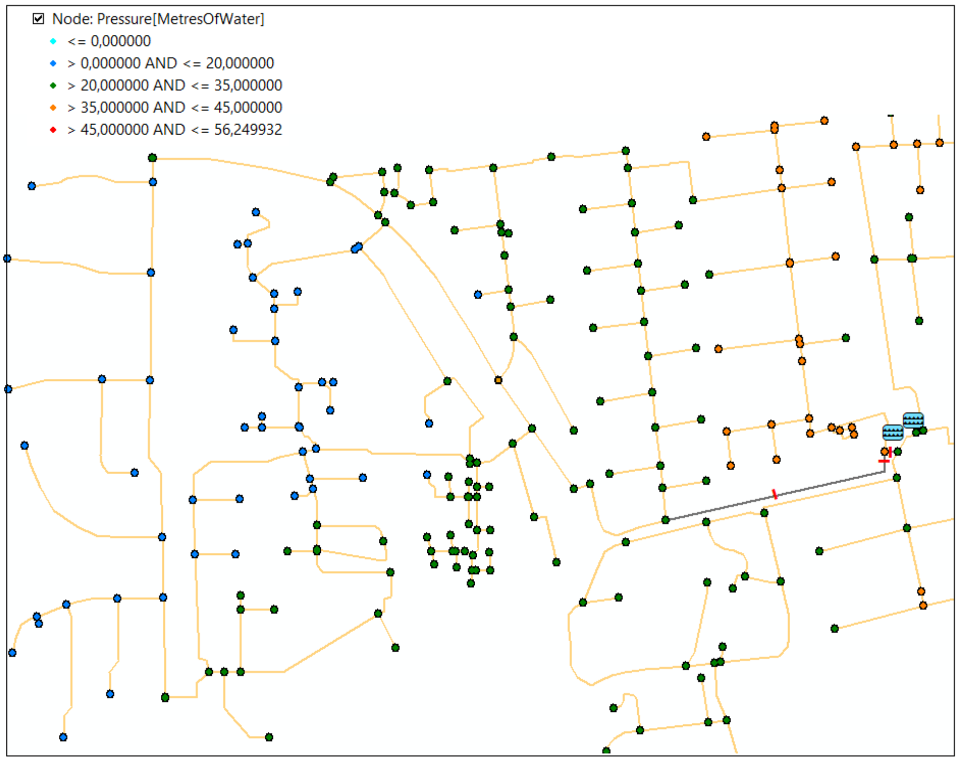


FIGURE 3.30 Pressure of the high-pressure zone after closing a main pipe in a peak hour. Monday 6 am

In conclusion, if this pipe needs to be repaired or closed in any case it should be repaired as fast as possible otherwise a lot consumers will not have sufficient pressure to secure a normal flow of the water. To avoid this kind of risk, another option would be to replace the small pipe mentioned above with a bigger pipe and eliminate the bottleneck in that way.

3.3 Conclusion

It can be said that the supply system, in its existing state, works fine. Despite the very low velocities (could be due to a possible over-dimensioning of the pipes), the water age is good. However, because of the over-dimensioning of the system, it's also possible to supply new areas in the future, but some bottlenecks may be created due to higher consumption. There aren't many critical points, besides the fire flow. According to the simulation, a normal fire flow between 14 to 25 l/s is currently unfavourable since this would cause negative pressure. Lastly, having to close certain pipe (such as main pipe II) can have a massive impact on the supply and therefore it should be repaired as soon as possible if such situation occurs.

4. ARSENIC AND PESTICIDE REMOVAL

In addition to investigating the current distribution system and water work, further consideration and possibilities of Arsenic (As) and pesticides (BAM) removal were also explored. A scenario was devised in which there is evidence of criteria-exceeding level of As and pesticides, each pollutant found in one of two hypothetical possible intake wells. This means that additional treatment is required, apart from the current treatment process of the water work, no matter from which well the extra water is to be abstracted from. Therefore, this chapter is going to consider possible treatment methods of As and BAM, based on the water quality matrix presented in Section 2.2.6 Groundwater Chemistry. Then, one of the contaminants would be concluded as more important to treat and a proposal as to how its treatment could be integrated in the existing process would be suggested.

4.1 Arsenic removal methods

According to the U.S. EPA, prior to choosing removal methods, investigation of the source water characteristics must be carried out, as certain parameters are crucial to the performance of the As removal process. Firstly, Arsenic species directly affect the removal efficiency. Generally, arsenate or As(V) is easier to remove than arsenite -As(III). So if the arsenite is dominant in influent water, it is recommended that a pre-oxidation stage takes place so that arsenite could oxidize into arsenate. Secondly, iron concentration is important since arsenate is removed by adsorbing to iron coagulants, precipitation of natural iron or adsorptive media. The more iron there is, the higher the efficiency of removing arsenate. Therefore, concentration of As and Fe changes what treatment method suits best, both in their quantity and in their relationship with each other:

- **High Fe levels (>0.3 mg/l) and high Fe: As ratio (>20:1)**: arsenic removal based on iron removal processes with enhanced oxidation, adsorption and co-precipitation.
- **Moderate Fe levels (>0.3 mg/l) and low Fe: As ratio (<20:1)**: a modified treatment method such as coagulation or filtration with iron salts added is necessary
- **Low Fe levels (<0.3 mg/l)**: Technologies such as adsorptive media, coagulation/filtration and ion exchange are most suitable.

Thirdly, pH value sometimes needs to be adjusted to increase the removal capacity of the adsorptive media. The following table show optimal (or acceptable values) of pH as guidelines for certain methods:

TABLE 4.1 pH optimal values for several As removal methods [EPA, 2005]

Method	Optimal level of pH	Acceptable level of pH
Iron-based media adsorption	< 7.0	7-8
Alumina-based media adsorption	5.5	-
With iron oxides via coagulation/filtration related processes	5.5-8	-
Ion exchange	Not impacted	-

4.1.1 *Common methods*

Having determined these preliminary properties, the most appropriate method can be chosen. Typically, other factors also must be examined, such as wastewater and sludge treatment technology, fulfilling requirement from regulations, testing and monitoring of the method before and during implementation, and financial considerations. However, the scope of this section only considers the theoretical and technical aspect of the methods.

The key concepts and respective chemicals and materials used for each method are given as an overview. The method which is found to be most suited to the scenario of this chapter will be discussed further in depth.

Popular As removal methods that have been widely tested and studied are divided into 4 categories:

- Precipitation processes: Adsorption co-precipitation with hydrolysing metals such as Al^{3+} and Fe^{3+} is most common, followed by filtration. Oxidation of arsenite to arsenate is recommended.
- Adsorptive processes: Water is led through a contact bed and arsenic is removed by chemical surface reactions. The media could be activated carbon, activated alumina, Fe/Mn oxide based or coated filter media
- Ion exchange processes: removes specifically negative-charged As(V) ions so if As(III) is present, it needs to be oxidized.
- Membrane processes: consists of nano-filtration, reverse osmosis and electrodialysis and is more effective at removing As(V) than As(III)

Now that all the basic knowledge has been presented, it is time to explore the specific case of this chapter.

4.1.2 *Relevant water quality matrix*

As stated above, determination of a background water quality is important. As shown in Section 2.2.6, boring number 5 (or DGU 88.1307) was found to have different water quality matrix and elevated level of As when compared to all the other wells. This means that this well belongs to another aquifer and this aquifer, even if able to provide enough yield to supply water, would complicate the treatment process because of the As. Therefore, this well is considered one of the two possible intake wells in the scenario mentioned above, which takes advantage of the few chemical analysis parameters provided in JUPITER database as the possible water quality matrix.

The following table does not only include parameters from boring 5 (B5) but also from other wells, since these are going to be needed for later calculations. The concentration of Fe for B5 was not found on Jupiter but was taken from an on-site test informed by Peder Maribo.

TABLE 4.2 Parameters from each boring related to As removal

Boring no.	Boring ID	Flow [m^3/h]	Fe [mg/l]	Mn [mg/l]	NH_3 [mg/l]	Others
B1	88.813	27	0.3	0.6	0.07	
B4	88.1394	33	0.417	0.413	0.02	
B5	88.1307	10	0.8	n/a (0.5)	0.645	pH: 7.84 As: 18 µg/l
Average (of only 3 wells above)		23.3	0.5	0.5		
B2	88.723	27	0.54	0.45	n/a	

Since no concentration of Mn was recorded for B5, a level of 0.5 mg/l was assumed for later calculation. B1 and B4 is connected to the new water work and B2 is connected to the old one. B5 is yet connected to either and is assumed in this report to be linked to the new facility, since this one has stronger aeration and larger filtration area.

These indicate high iron levels and high Fe: As ratio (here is around 40:1), meaning standard iron removal process with addition is enough to reduce As to drinking water level via adsorption and co-precipitation [EPA, 2005]. In optimized conditions, the rule of thumb says that removal of 1 mg/l of Fe would achieve removal of 50 µg/l of As. Here, with the criteria for Fe of 0.1 mg/l (for outgoing water from water work) [Maribo, 2017], at least 0.7 mg of Fe needs being removed, which would result in reduction of 35 µg As, more than enough to bring As concentration to acceptable level. However, this is a rough extrapolation, and given that this method is not 100 percent efficient, it is unlikely that absolutely all of the Arsenic will be removed.

4.1.3 Possible complications and solution

Nevertheless, said rule of thumb only applies if the As species in question is arsenate – As(V) and not arsenite – As(III). Since this water sample analysis does not indicate whether the arsenic found mostly consists of arsenate or arsenite compounds, it is assumed that most of the arsenic content is in the form of arsenite compounds because these are more water soluble. However, arsenate is easier to remove than arsenite, so one step of special oxidation (pre-oxidation) is needed in addition to the standard treatment of iron removal. Because aeration would only be able to oxidize Fe and Mg, not As, another oxidizing agent such as chlorine, permanganate, ozone and manganese dioxide media (greensand) should be added before the filtration. Arsenate then adsorbs to the iron hydroxide precipitates that eventually gets filtered out of the water. Optimal pH level for this treatment is less than 7.0 but a value in the range pH 7-8 is acceptable. This method is considerably effective, with a removal efficiency of approximately 80-90% [EPA, 2005].

However, according to the study “Predicting As removal during metal hydroxide precipitation” [McNeill, 1997], at a Fe level of only 0.8 mg/l (corresponding to ~0.014 mMol), As removal efficiency could only reach around 50% at most, as can be seen from Figure 4.1:

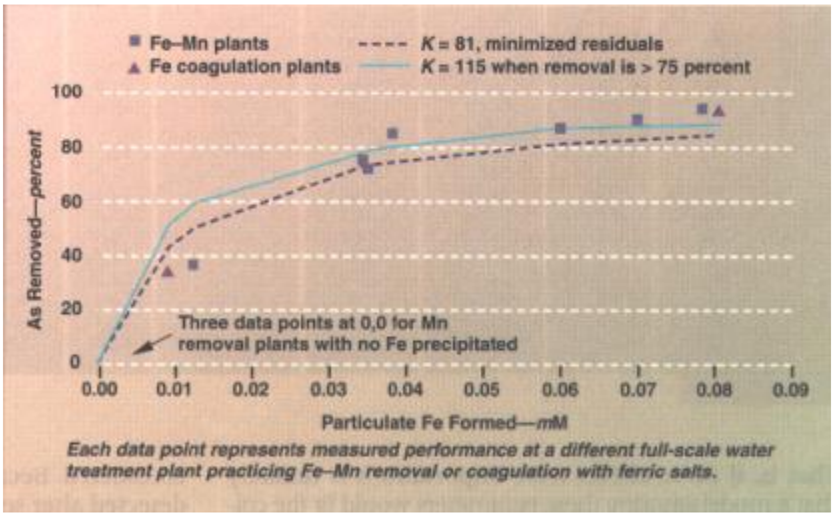


FIGURE 4.1 Total As removed as a function of the amount of Fe formed [McNeill, 1997]

This diagram was plotted according to real-life statistics from several different waterworks, with the Fe-Mn plants among them, meaning the plants employing the previously supposed sufficient method. Real-life plot implies a level of credibility difficult to ignore. Hence, it's likely that with just 0.8 mg of Fe, only ~9 µg of As would be removed, against all the thumb rule and assumptions given above by EPA.

On the safe side, suppose there is now 9 µg of As(V) left needed to be removed. At this point, there are two strategies to consider: either diluting the As level by mixing water from this boring with water from other wells or adding coagulant to aid the precipitation process.

With the diluting strategy, if the total amount of Fe after mixing is sufficient to remove As, this could be the simpler solution. From Figure 4.1, if 80% of removal is to be expected, at least around 0.035 mMol Fe/l is required. This concentration equals to 1.96 mg Fe/l. With the water from boring 5 already have 0.8 mg, there is still 1.16 mg needed. However, in order to reach this amount of Fe, water from all the remaining 3 borings has to be combined (since if only water from B1 and B4 - wells connected with the new waterwork - is mixed: 0.3+0.4=0.7, <1.16 mg Fe/l). Considering the situation that the 3 wells have already been split into 2 different treatment facilities, it is deemed impractical to try mixing all of the water together and then split them again. Hence, the scenario of mixing all water from 4 wells is quite unrealistic and unfavourable.

The other and perhaps more beneficial strategy to aid the treatment process would be adding a coagulant to help with the effective adsorption and precipitation. This coagulant could either be iron or aluminium salts, since they are both effective (with iron salts slightly better) [McNeill, 1997]. However, introducing aluminium to the water could possibly be quite troublesome since in the same study, not all Al added precipitates as $Al(OH)_3$, while As can only sorb to the precipitated. Hence, this would reduce the amount of As potential for removal and partly diminish the purpose of adding Al as a coagulant. On the other hand, all Fe added as coagulant would precipitate as iron hydroxides and attract all the As it could. Therefore, the iron salts were chosen as a more assured solution.

$FeCl_3$ was then regarded as highly convenient as the iron ion already has valence state of III, effectively cancelling out the need to oxidize the added iron and hence can be added directly to the contact basin or just before the filtration stage. Now the question remains of how much $FeCl_3$ is actually needed to remove the remaining As. From *Table 3 Summary of data used for simplified model* (refer to Appendix D: Arsenic Removal Section 1) of the study *"Predicting As removal during metal hydroxide precipitation"* by McNeill in 1997, a Fe concentration of 0.07 mMol would achieve 90% of As removal or in this case, reducing As level from 18 µg/l to 1.8 µg/l, well below the criteria. Since the water from B5 already has 0.014 mMol of Fe, there is need for 0.056 more, corresponding to 9.1 mg/l of $FeCl_3$. In total, the amount of Fe ions is 0.07*56 = 3.92 mg/l, rendering the Fe average value in Table 4.2 1.55mg/l instead of 0.5 mg/l.

However, adding a coagulant may have side effects. The more Fe is added, the more hydroxides and precipitation would be produced, raising the amount of substances retained by the filter. This would eventually lead to more frequent and possibly unfavourable backwash. Therefore, calculation of possible scenario of backwash intervals is essential.

Since the new water work is now connected to 3 different wells and water from each of which is pumped at alternate intervals, it would make sense to calculate the filter operation time between backwashes using average values of flow and concentration of Fe and Mn in the water from all 3 wells. Using the average values from Table 4.2, the amount of Fe and Mn retained over 1 m^2 filter is:

$$23.3\frac{m^3}{h} \times (1.55 + 0.5)\frac{gFe, Mn}{m^3} \times \frac{1\,kg}{1000\,g} \times \frac{1}{20.4\,m^2} = 0.0023\frac{kg}{h * m^2}$$

With the rule of thumb stating that backwash should take place whenever there is 0.5 kg Fe + Mn retained per m^2 of filter. Therefore, the amount of time between backwash should be: 0.5/0.0023 = 217 hours, or 217/24 = 9 days. This frequency of backwash is deemed not to frequent and unlikely of causing severe adverse effects on filter media. In short, employing $FeCl_3$ seems to be a reasonable solution.

Additionally, it is decided that chlorine could be the potential oxidant for oxidizing As(III) to As(V) because of its effectiveness [EPA, 2003]. However, As is not the only substance that oxidize using chlorine. The following compounds/substances are also competitive of oxidizing source: Fe, Mn, HS and NH_3. Hence, when determining the necessary dose of Cl_2 to be added, chlorine demand of these parameters need to be noted as well. Nevertheless, since these concentrations are not of extremely high level, it is likely that the chlorine dose would not be as high as creating needs for dechlorination later-on in the treatment process. A rough estimation of chlorine demand is presented in Appendix D: Arsenic Removal Section 2. Of course, bench-scale test is still necessary to determine the right dose.

Regarding $FeCl_3$, a small system such as in Figure 4.2 could be established to monitor the pace and dose added. A tank made of either fiberglass-reinforced polyester or rubber-lined steel should be used to store ferric chloride. Flow meters are used to pace the water flow and dosage. At the connection to the water line, an isolation valve and check valve are used. An inline mixer could be installed and after mixing with $FeCl_3$, the water is sent back to the main treatment line for filtration.

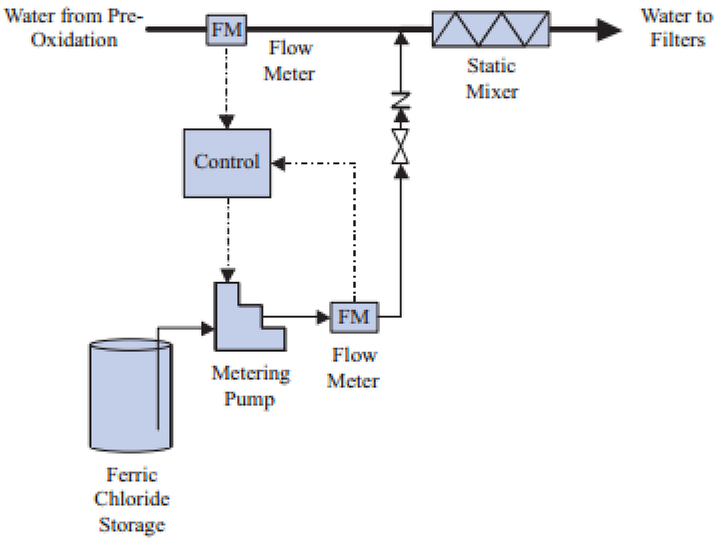


FIGURE 4.2 Ferric chloride addition flow diagram [source: Arsenic Treatment Technology Evaluation Handbook for Small Systems, EPA, 2003]

4.1.4 Possible sludge treatment and conclusion

When considering a treatment method, attention should also be paid to the waste generated. In this case, the waste is sludge from the settling basin and the filter backwash water. Because it contains high concentration of arsenic (and other contaminants), it would likely be hazardous and need to be treated properly before disposal or being directed to sewers. [EPA, 2005]. Since sludge is semi-liquid, the same methods of removing As for potable water can also be used to remove As from sludge liquid. In a study, the sludge samples were treated with activated alumina and iron-based media, in which the latter was found to have higher efficiency, generally around 70% and can reach 100% if the method for prior treating of groundwater was also based on iron removal processes. [MacPhee, 2001]. After this, As would mostly bind to particles in solid phase, further easing the process of cement based solidification and stabilization (s/s). This process involves mixing the sludge and cement in a binder system. The calcium in cement inhibits the leaching and immobilizes As, hence reducing contamination risk to the environment. The higher the Ca: As ratio, the lower the leaching. This s/s technique, when combined with lime and Ordinary Portland cement (OPC), successfully stabilizes and safely disposes of arsenic sludge. [Mandal, 2015].

To summarize, a pre-oxidation step is first added to the normal iron removal process. Here, oxidation takes place so that Fe, Mn and As can all be oxidized. Then the water is led to a contact basin where iron chloride is added, arsenic adsorption to ferric hydroxide occurs and subsequently to filters, where the coagulations and precipitation are removed, resulting in potable water. Lime may be added to maintain pH and control corrosion. The whole procedure requires some changes to the existing treatment line, as illustrated in Figure 4.3.

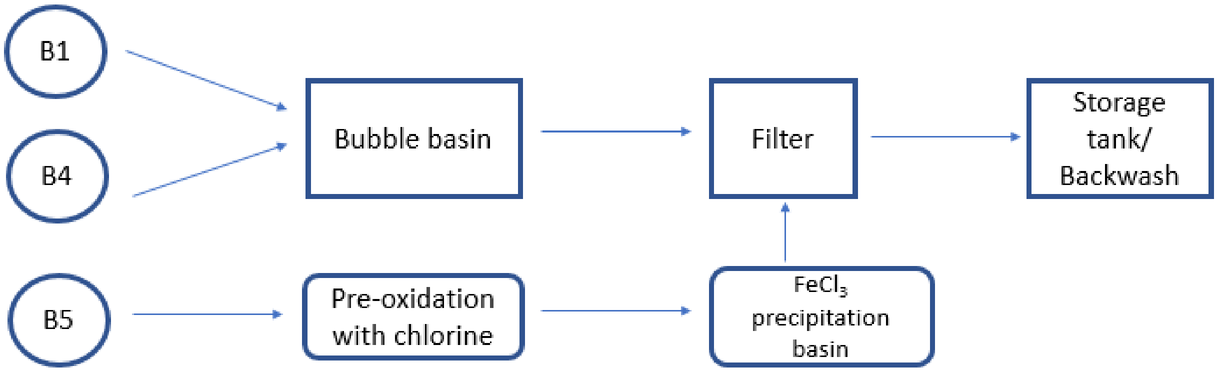


FIGURE 4.3 Example of integrating arsenic removal by iron removal process to current treatment process

The angular rectangles represent the existing treatment line and the rounded rectangles the added one for the arsenic water from boring 5. The arrows represent the flow of water from wells as raw water to storage tank as clean water.

If the removal efficiency is 90% according to calculation above, the outgoing As level would fulfil drinking water criteria. The sludge water should then be treated separately with adsorption by iron-based media and sent to sewers. The sludge solids could be sent to treatment with solidification and stabilization technique. Utmost effort should be made to ensure that high amount of As in the waste would not leach and contaminate soil or other groundwater aquifers.

4.2 Pesticide removal methods

Pesticide has also been a rising problem in drinking water sources in recent decades. Many studies and researches have also been conducted targeting at reducing pesticide or pesticide transformation and decompose by-products in water. In groundwater, pesticide would not exist in just one single compound or one kind of pesticide, rather there would be a combination. Usually, this combination is made up of decompose by-products from the original pesticide. A common term for this combination is BAM so pesticide in groundwater would also be referred to as BAM henceforth.

Since there is no exceeding level of BAM in any of the water samples of Skovby, it is assumed that a concentration of medium to low (0.5-1 µg/l, the criteria is 0.5 at the water work) would be appropriate (for comparison with situation for As above). Similar to treating As, there is a variety of methods to treat pesticide, and each has its own pros and cons. Only some popular ones would be listed. Most of the following description and statement are referred from the bachelor thesis: “Pesticide removal from water” by Bourgeois et al (2012).

4.2.1 Common methods

Activated carbon (AC) adsorption: This is the most popular method due to its low cost, ease of use and limited equipment required. The mechanism of purification is simply through adsorption thanks to its high area of porous surface. There are two forms of activated carbon: powdered and granular. Since powdered carbon are smaller in size and could potentially clog existing downstream machineries, granular carbon (GAC) is commonly more preferred.

Removal efficiency of BAM ranges from 50-90%, depending on the composition of original pesticide.

Chlorination: this is a quite common method as well, also due to its low cost and installation fees. The operation mechanism is through oxidization. Because of this, its efficiency also depends on possible existence of oxidizing competitive substances in the water. If there is, its removal of BAM would decrease. Overall a removal percentage of 60% was found. However, if excessive Chlorine is added, dechlorination is often required to mediate the level down to accepted Chlorine concentration and this could induce further unnecessary cost and complications.

UV photolysis: Mechanism employed is photolysis, or degradation using light. Since this process occur at low rate, hydrogen peroxide is often added to rapidly oxidize compounds to aid the photolysis. This method is beneficial in which it can facilitate complete oxidation of pollutants, or destruction of compounds, as opposed to a phase-transfer method (like that of activated carbon). If this happens, the ultimate end products would be carbon dioxide and other fully oxidized compounds. However, if full breakdown does not happen, a variety of by-products could occur and further complicating the water quality and treatment process. Experiments of the study shows removal efficiency ranging from 50 to 90%, depending on the pesticides considered. Considerable number of persistent by-products were found. Also, the more hydrogen peroxide added, the more positive the outcome. So, although being quite effective, there is risk of incomplete breakdown and thus creating more by-products and problems. Moreover, the seemingly certain need for hydrogen hydroxide is another source of expense.

Ozonation: this is also a popular method, being able to oxidize wide variety of metals and pollutants, but would always leave residue of O_3, which can be extracted using turbulent flow of water. Ozone must be created from oxygen in air and fed to the water. The average removal rate of pesticides is 40-60% and increases to 65% if a coagulant is added. H_2O_2 can also be added to help with oxidation but this can become very expensive very quickly. Overall, this seems like a rather costly method compared to others due to the equipment for creating ozone and possible necessary coagulant or hydrogen peroxide.

Membrane filtration: this seems to be the most effective method, with removal in the order of 80-100%, especially with ultrafiltration (thin film composite). Nanofiltration is effective against pesticides as original compounds, but not against by-products. However, the biggest disadvantage is the high maintaining cost. Membrane filtration needs close monitoring and maintenance to keep the decreasing efficiency from plummeting too quickly.

Out of all the above methods, activated carbon appears to be the most suitable with medium to low concentration of BAM. The operation doesn't need close monitoring or expertise, there is less possible complications, the efficiency is relatively high and the operation cost is rather low.

4.2.2 Activated carbon and possible complications

There was a Danish study by Clausen et al in 2003 [Clausen, 2003.] performed on both BAM and atrazine (an original pesticide). A realistic bench-scale experiment was set up and it showed a considerable head loss in the top 2 cm of the adsorber tank due to iron precipitation. This can be easily solved by manually removing the top layer, effectively reduce the need and expenses for backwash, which is beneficial, since backwashing can cause saturated carbon particles from below to migrate to the top and facilitate early breakthrough of pesticide.

More importantly, the bench scale resulted in adsorbing capacity of 48-57 µg BAM/g AC and 43-66 µg atrazine/g AC with a maximum effluent concentration of 0.1 µg/l (or the criteria for BAM in Danish context). A lower maximum outlet of 0.01 µg/l reduces the capacity to 20-34 µg BAM/g AC and 15-27 µg atrazine/g AC. This order is still relatively high anyway and seem to ensure sufficient removal.

However, diffusion of pesticides into AC appears to be a slow process, in which equilibrium was not reached until after 2-3 weeks. Additionally, no major difference was shown between the removal of BAM and atrazine, which means AC can efficiently remove both, no matter the substance is pesticide or by-products. [Clausen, 2003]

As mentioned above, there are two AC materials: powdered AC and granular AC (abbreviated as PAC and GAC, respectively). According to Water Research Foundation, PAC has lower initial cost and flexibility in dosage but high operating cost, impossibility of regeneration and production of large amount of sludge are the inevitable disadvantages. On the other hand, despite also having high initial and operating cost, GAC can be regenerated, thus used for a longer period and is more economical for larger water treatment systems and continuous control of chemical contamination.

In a treatment line, GAC could be added post-filter (line a in Figure 4.4), where GAC unit is located after the traditional filtration process or replace the filter altogether and act as an adsorber-filter tank (line b in Figure 4.4).

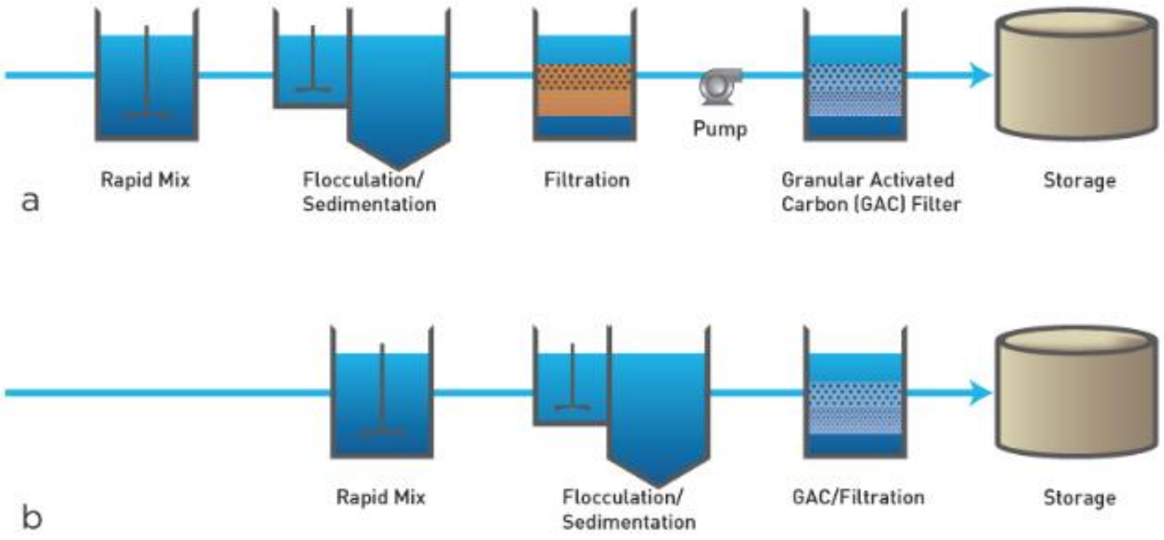


FIGURE 4.4 Integration of GAC to treatment line: (a) Post-filtration adsorption; (b) Filtration-adsorption [Water Research Foundation, 2008]

In the case of Skovby, if a water source with elevated BAM is discovered and brought into production, establishing a separate treatment line for the pesticide-polluted water is recommended to minimize the risk of spreading the contaminants to other water sources. If this strategy is ever regarded as impractical or impossible, a GAC filter could then be added at the end of existing treatment line (as line a in Figure 4.4), although this means that the BAM would be present in all of the preceding facilities that contacts with more clean water from existing sources.

However, AC adsorption capacity is always affected by natural organic compounds in the raw water as these and BAM compete for AC's adsorption area. High concentration of organic compounds could hinder and reduce the filter capacity. This reduction is called carbon-fouling and is irreversible. It happens when humic substances adhere to carbon surface in lower filter depth. [Baldauf, 1993] Therefore, it is important that TOC (total organic carbon) level of the candidate water be examined carefully when considering this type of treatment method. This parameter is not shown to have elevated concentration in all water samples from Skovby, so it is assumed that the considered scenario doesn't pose this problem either.

Additionally, regeneration of GAC seems to be a costly deal since according to Water Research Foundation: "Depending on the economics, facilities may have on-site or off-site regeneration systems or may waste spent carbon and replace it with new". Furthermore, on-site regeneration facilities often require obtainment of air permits. Of course, regeneration always is a more environmentally sustainable choice, but if it proves to be too expensive to handle, it could be considerably problematic.

4.3 Cost comparison and conclusion

Since cost is considerably important when making decisions, the following is a rather rough estimation of operation cost of As and pesticide removal taken from various studies. Wherever possible, information from Denmark is used, otherwise data from other countries were referred, unit adapted to the situation of Skovby (mgd - million gallons per day to m^3/d) and the currency converted to DKK. Please note that this only include operation cost and not initial capital cost for instalment of new equipment or system.

As removal would employ pre-oxidation using chlorine, ferric chloride as coagulant and already include lime treatment. Pesticide removal would employ activated carbon filter.

TABLE 4.3. Comparison of possible costs for As and pesticide removal

Substance/system	Cost in original source	Cost converted to situation of this report
Chlorine for pre-oxidation of As removal [EPA, 2003]	USD 1100/y for flow rate of 0.04 mgd (~151 m^3/d, supposing water from this boring is pumped for 10 h/d at most)	DKK 0.05/m^3
Ferric chloride for As removal (cost for lime treatment included) [EPA, 2003]	USD 800/y for flow rate of 0.04 mgd	DKK 0.04/m^3
Sum for As removal		**DKK 0.09/m^3**

Activated carbon filter for pesticide removal [Eurowater, 2017]		**DKK 0.15-0.2/m^3**

As can be seen, the operation cost for pesticide removal is higher than of As removal. Although this is a rough comparison, it still weighs in favour for As removal over the other alternative.

From sections 4.1 and 4.2, it seems that pesticide is easier to treat as there is virtually no side effects and further complication, as opposed to the situation with As removal. However, pesticide is a complex problem in and of itself since there is a wide range of compounds and by-products that can exist under the “pesticide” category. On the other hand, arsenic is just one substance and even though As(III) is troublesome, once it’s oxidized to As(V), it can be removed quite easily. Furthermore, if all side effects of As treatment is tackled, the effluent level can be assured, as all the calculations above have shown. Too frequent backwash does not seem to be a potential problem either.

Certainly, in this report, it seems that the problem with As was investigated much more deeply than pesticide. The investigator did at first assume As to be easier to treat and hence, paid that issue more attention, which over time unravelled and proved to be significantly more complex than it had let on. Nevertheless, while pesticide in this case is more or less an imaginary problem (no water sample shows elevated of BAM in reality in Skovby), Arsenic is a real one and requires careful consideration. Also, pesticide removal seems to be a problem that only theoretical investigation based on hypothetical situation (such have been done for this report) does not give enough credibility.

Due to the scope and extent that the As removal scheme was examined compared to pesticide removal in this report, combined with the recommendation that As removal additional equipment could be added on the side of the existing treatment line while pesticide removal should be separated completely, it is suggested and concluded that the water work should choose the source with arsenic over that with BAM. The scheme for As treatment has also been planned out more or less while the other is still hidden with possible further complications (GAC regeneration, spent GAC disposal, contamination to other water sources) and probably would need a real-life bench-scale test to determine feasibility.

5. CONCLUSION

This report investigated the water supply system in Skovby, belonging to the Skanderborg municipality and supplying water to about 1300 consumers. The water is supplied via three active intake wells from a glacial meltwater sand aquifer. The geological profile was investigated, and it was found that the aquifer is confined by a clay layer, so likely is well protected. The three intake wells are DGU 88.813, DGU 88.723, and DGU 88.1394 and were estimated to require 0.55km^2, 0.47km^2 and 0.12km^2 of recharge area respectively. The water quality in each of these wells was investigated, as well as in well DGU 88.1307 which is no longer used for supply. In general, the water quality was determined to be in typical ranges and suitable for drinking water, apart from an excess of CO_2, iron, manganese and ammonium. These issues are easily treated by aeration and filtration. The current treatment facilities incorporate aeration and filtration, but in the old waterwork it is using a free fall aerator and in the new waterwork an aeration basin. Both of these treat the water to a satisfactory level. The waterworks were also studied to see if they could cope with a higher water demand, and was found that this was possible. The risks involved with the waterworks are not high, as in the case of a pump breaking the remaining pumps can be used, algae growth is not a major concern, and bacterial growth is unlikely.

The water consumption was also studied. There are two pressure zones in the area, and the consumption for each was estimated as approximately two-thirds of water consumption from the low-pressure zone and the remainder from the high-pressure zone. The variation in consumption between the zones was investigated, and found that there are differences in daily use due to the consumer type (household vs industry). The daily and hourly factors were also determined. In the future there will be more residential area, which is expected to increase consumption but have no huge effect on the variations. The future assessment was decided to be performed as if an extra 300 houses were built, increasing demand by 28110m^3. The supply network was modelled by MIKE Urban, and found that there are currently low flows and velocities in the network. This is possible due to over dimensioning of the pipes, although the water age is still fine. This may cause issues in the future when providing water for new areas as some bottlenecks would be created. In the event of fire flow, it could cause a negative pressure which is not ideal, so it should be considered to relocate one of the hydrants or lower the fire flow. If certain pipes had to be closed there would be a large impact on the supply, so it is worth noting that in some cases (such as main pipe II) these need to be monitored carefully as a breakage here would be very disruptive.

The water supply was also investigated in terms of the complications that could occur with pesticides and arsenic. Both of these contaminants were studied to see which is more important to treat. It was found that pesticides seem easier to treat but need real-life conditions and statistics for more ascertain testing and is more expensive to remove than arsenic. Thus Arsenic removal was studied further, and additional treatment based on Fe removal was devised. It is recommended that if the waterwork were ever to include water source with either As or pesticide into production, bench-scale testing and more thorough investigation based on real conditions take place.

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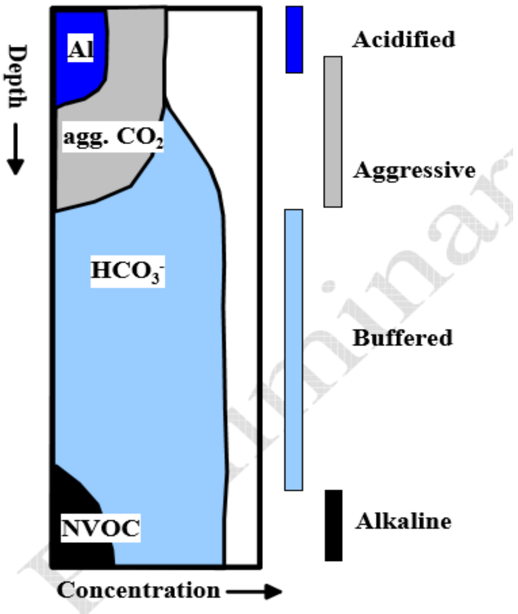
APPENDIX A. WATER QUALITY

1

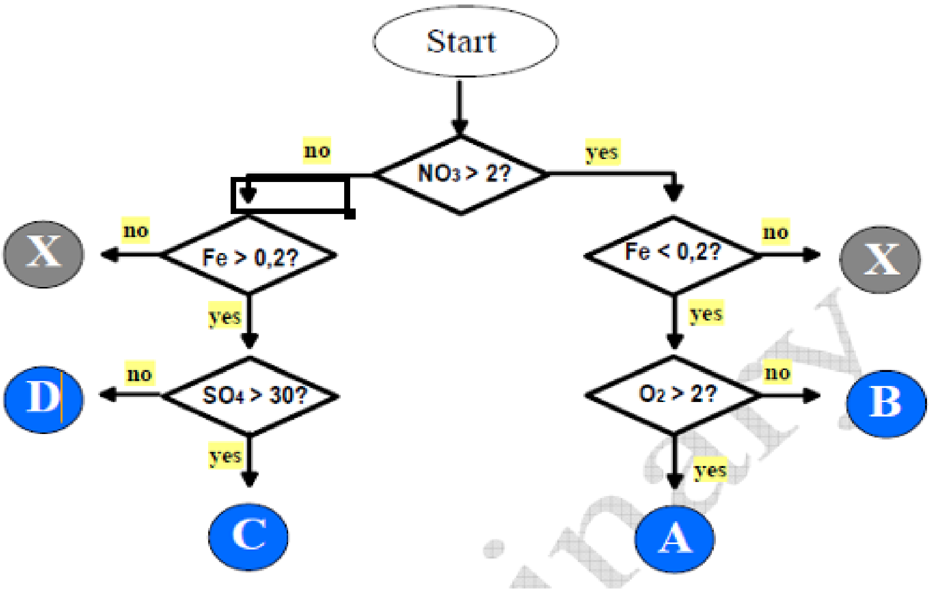
The pH indicates the acid-base type of the water. This image shows the different types.



The average pH of the three wells was found to be 7.6. . The type in this case is buffered, meaning that it resists changes in pH as small amounts of acid or base are added

2

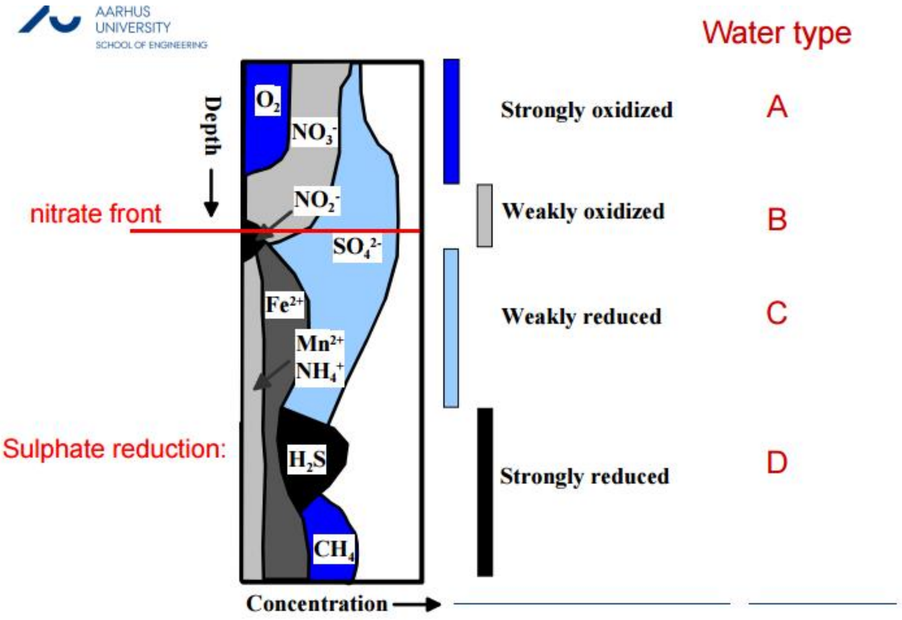
This image represents the algorithm used to see the redox type.



The final result was (based on the concentrations of nitrate, iron, and sulfate)that this water is redox type C, with no conflicts

3

Here the redox types are shown with their chemical composition.



As is said in point 2, the Skovby water type is c, so weakly reduced.

4

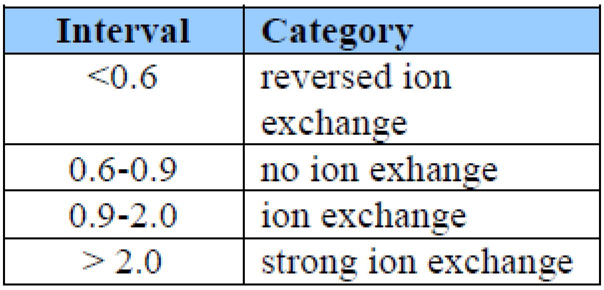
The formula of the ion exchange

$$I = \frac{\frac{Na}{23}}{\frac{Cl}{35.5}} = 1.54\left(\frac{Na}{Cl}\right)$$

The ion exchange for this water was 0,8 .

5

Table that shows the category for each interval of ion exchange value.

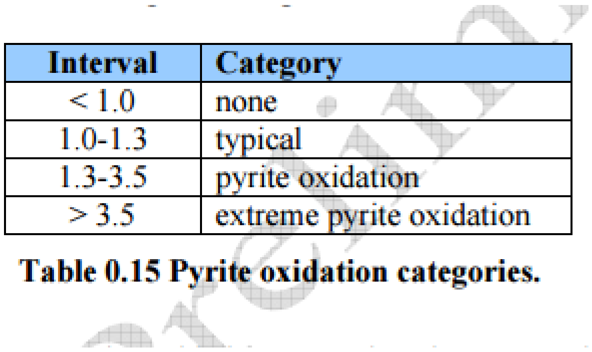


Interval	Category
<0.6	reversed ion exchange
0.6-0.9	no ion exhange
0.9-2.0	ion exchange
> 2.0	strong ion exchange

The value 0,8 means there´s no ion exchange in the water.

6

Table that shows the category for each interval of degree of weathering



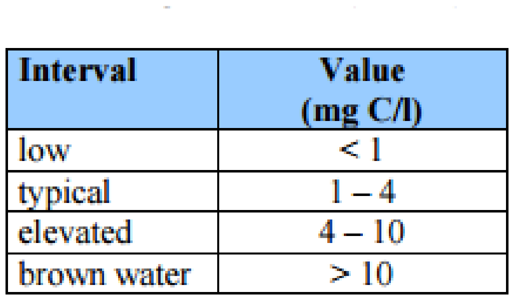
Interval	Category
< 1.0	none
1.0-1.3	typical
1.3-3.5	pyrite oxidation
> 3.5	extreme pyrite oxidation

Table 0.15 Pyrite oxidation categories.

The value obtained was 1,38 so there´s a little bit of pyrite oxidation but it´s really close to the typical value.

7

Table to see if the NVOC content is high, low or typical.



Interval	Value (mg C/l)
low	< 1
typical	1 – 4
elevated	4 – 10
brown water	> 10

The NVOC content was 1,23 mgC/L which is a typical value.

.

8

Danish drinking water criteria

Major constituents

Parameter	Formula	Units	Water-works	Property Line	Tap
calcium	Ca^{2+}	mg/l	-	-	-
magnesium	Mg^{2+}	mg/l	50	50	50
total hardness	-	dH°	-	-	-
sodium	Na^+	mg/l	175	175	175
potassium	K^+	mg/l	10	10	10
ammonium	NH_4^+	mg/l	0.05	0.05	0.05
iron	Fe	mg/l	0.1	0.2	0.2
manganese	Mn	mg/l	0.02	0.05	0.05
hydrogen carbonate	HCO_3^-	mg/l	-	-	-
chloride	Cl^-	mg/l	250	250	250
sulfate	SO_4^{2-}	mg/l	250	250	250
nitrate	NO_3^-	mg/l	50	50	50
nitrite	NO_2^-	mg/l	0.01	0.1	0.1
total phosphorous	P	mg P/l	0.15	0.15	0.15
fluoride	F^-	mg/l	1.5	1.5	1.5
total dissolved solids	-	mg/l	1500	1500	1500
NVOC	-	mg C/l	4	4	4

Gasses

Parameter	Formula	Units	Water-works	Property Line	Tap
oxygen	O_2	mg/l	-	5	-
aggressive carbon dioxide	CO_2	mg/l	2	2	2
hydrogen sulfide	H_2S	mg/l	0.05	0.05	0.05
methane	CH_4	mg/l	0.01	0.01	0.01
chlorine	Cl_2	mg Cl/l	-	-	-

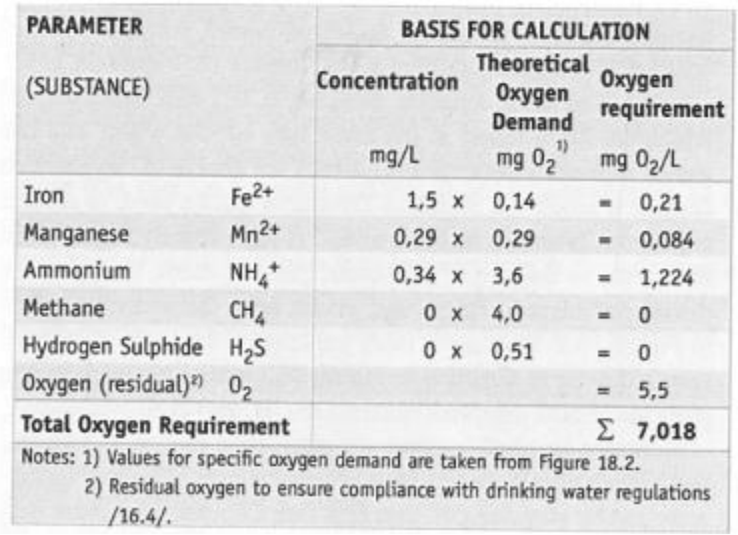
Inorganic trace elements

Parameter	Formula	Units	Water-works	Property Line	Tap
aluminum	Al	µg/l	-	100	200
antimony	Sb	µg/l	-	2	5
arsenic	As	µg/l	-	5	10
barium	Ba	µg/l	-	700	700
lead	Pb	µg/l	-	5	10
bor	B	µg/l	-	1000	1000
cadmium	Cd	µg/l	-	2	5
chrom, total	Cr	µg/l	-	20	50
copper	Cu	µg/l	-	100	2000
mercury	Hg	µg/l	-	1	1
nickel	Ni	µg/l	-	20	20
selenium	Se	µg/l	-	10	10
zinc	Zn	µg/l	-	100	3000
zinc	Zn	µg/l	-	-	5000

APPENDIX B. TREATMENT FACILITIES

1

Example of how to calculate theoretical oxygen demand



PARAMETER (SUBSTANCE)		BASIS FOR CALCULATION				
		Concentration mg/L		Theoretical Oxygen Demand mg O_2 1)		Oxygen requirement mg O_2/L
Iron	Fe^{2+}	1,5	x	0,14	=	0,21
Manganese	Mn^{2+}	0,29	x	0,29	=	0,084
Ammonium	NH_4^+	0,34	x	3,6	=	1,224
Methane	CH_4	0	x	4,0	=	0
Hydrogen Sulphide	H_2S	0	x	0,51	=	0
Oxygen (residual)2)	O_2				=	5,5
Total Oxygen Requirement					Σ	**7,018**

Notes: 1) Values for specific oxygen demand are taken from Figure 18.2.
2) Residual oxygen to ensure compliance with drinking water regulations /16.4/.

In the case of Skovby the oxygen demand was 0,5 mg O_2/L. The way of calculating it was the same as in the table above.

2

Different types of aeration methods

Name	**Step/ cascade**	**Packed tower**	**Bubble**	**Pressure filter**	**INKA**
Type	water in air	water in air	air in water	air in water	fluidized
Water movement	gravity drop	gravity drop	gravity flow	pumped	gravity flow
Air movement	room ventilation	blower	compressor	compressor	compressor
Current	cross-current	counter current	cross-current	cross-current	co-current

In the old waterworks they use freefall and bubble basin in the new ones. Here there are some of the characteristics of each method.

3

Mn, Fe and NH4 removal

$$2Mn^{2+}{}_{(aq)} + O_2 + 2H_2O \rightarrow 2MnO_2 + 4H^+$$

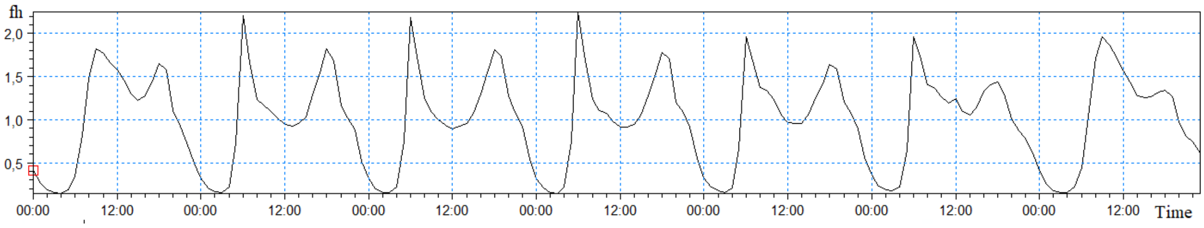
$$4Fe^{2+}{}_{(aq)} + O_2 + 6H_2O \rightarrow 4FeOOH_{(s)} + 8H^+$$

$$NH_4^+ + 2O_2 \rightarrow NO_3^- + H_2O + 2H^+$$

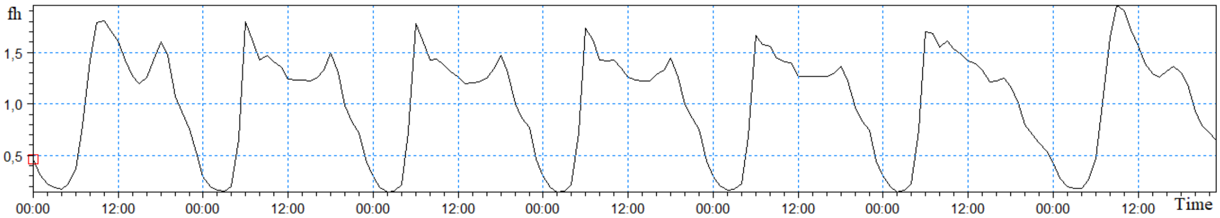
Here there the reactions that explain how the parameters that are exceeding the criteria are removed.

APPENDIX C. MIKE URBAN

1

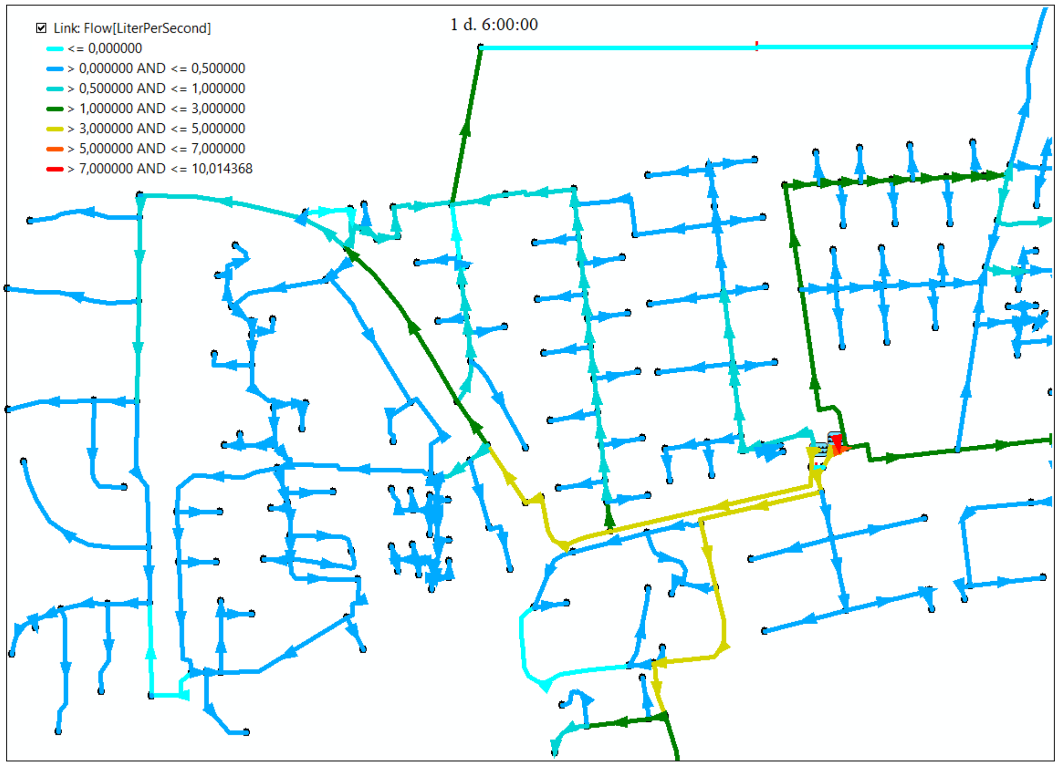


Cyclic profile of used hour factor for one week in high-pressure zone starting with Sunday



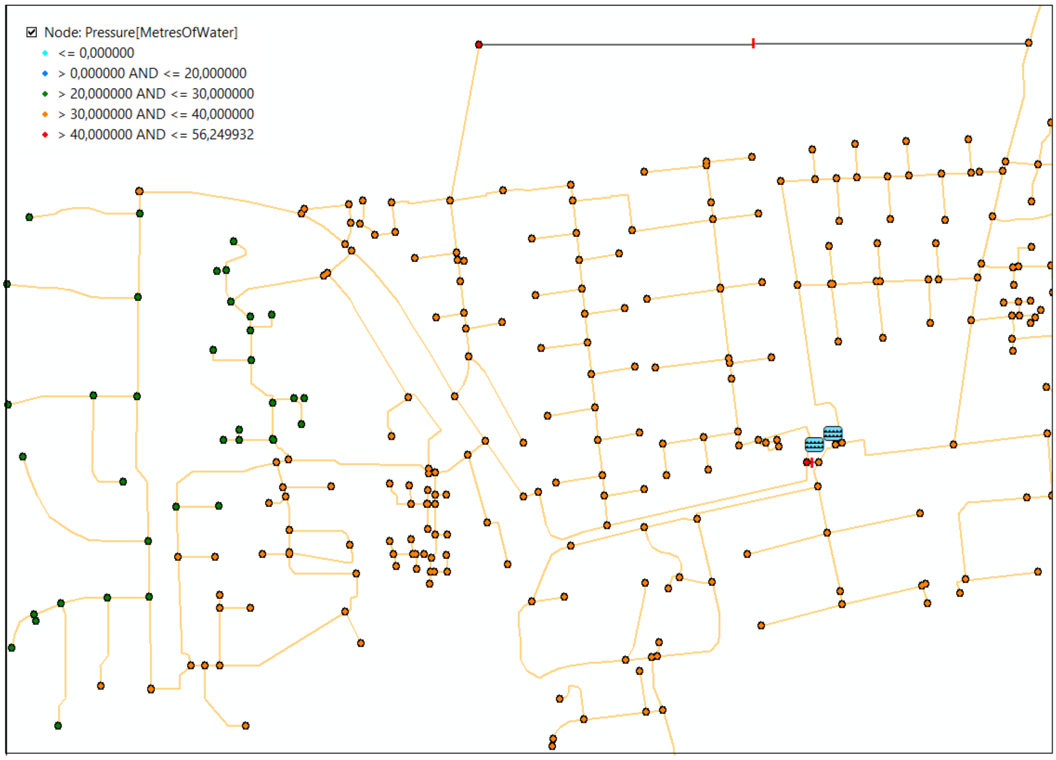
Cyclic profile of used hour factor for one week in low-pressure zone starting with Sunday

2



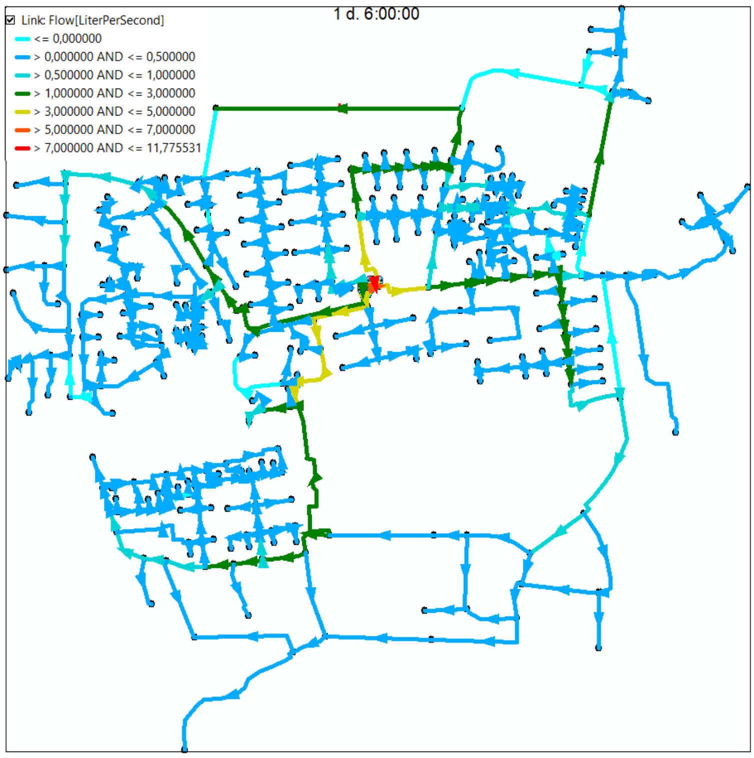
Flow during peak hour (Monday 6 am) in high-pressure zone with new residential area

3



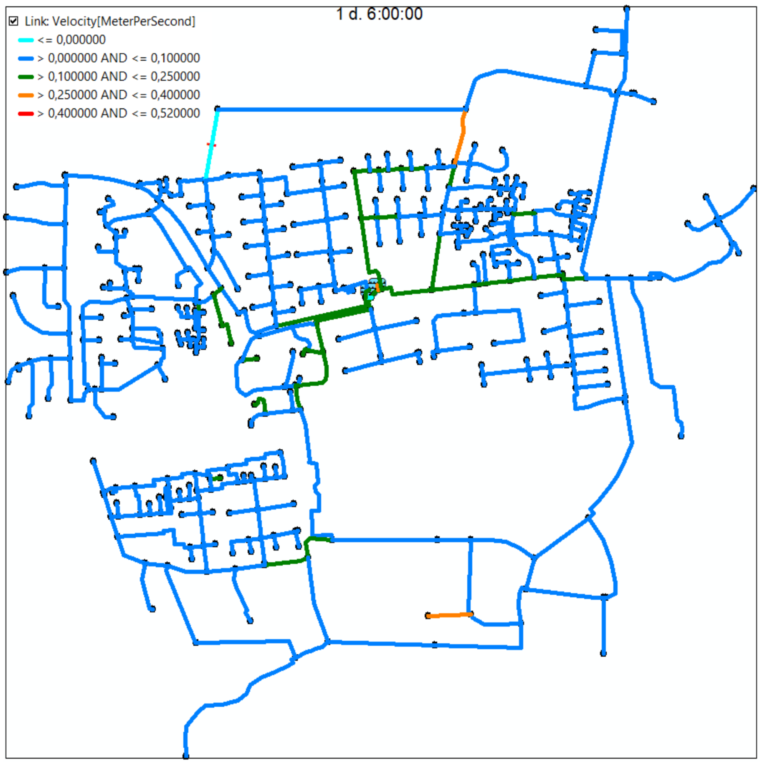
Pressure in high-pressure zone with new residential area

4



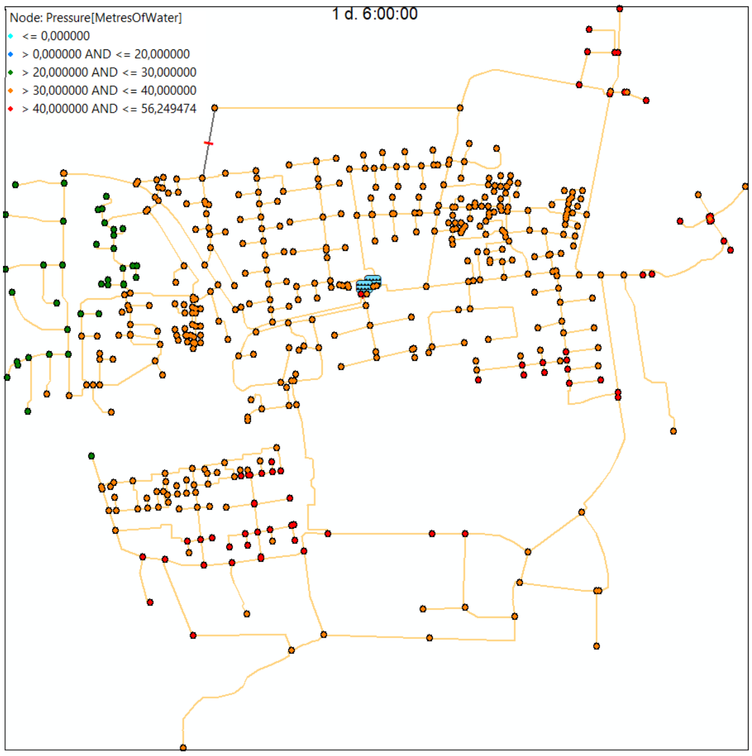
Flow during peak hour (Monday 6 am) if new residential area is connected to low-pressure zone

5



Velocity during peak hour (Monday 6 am) if new residential area is connected to low-pressure zone

6

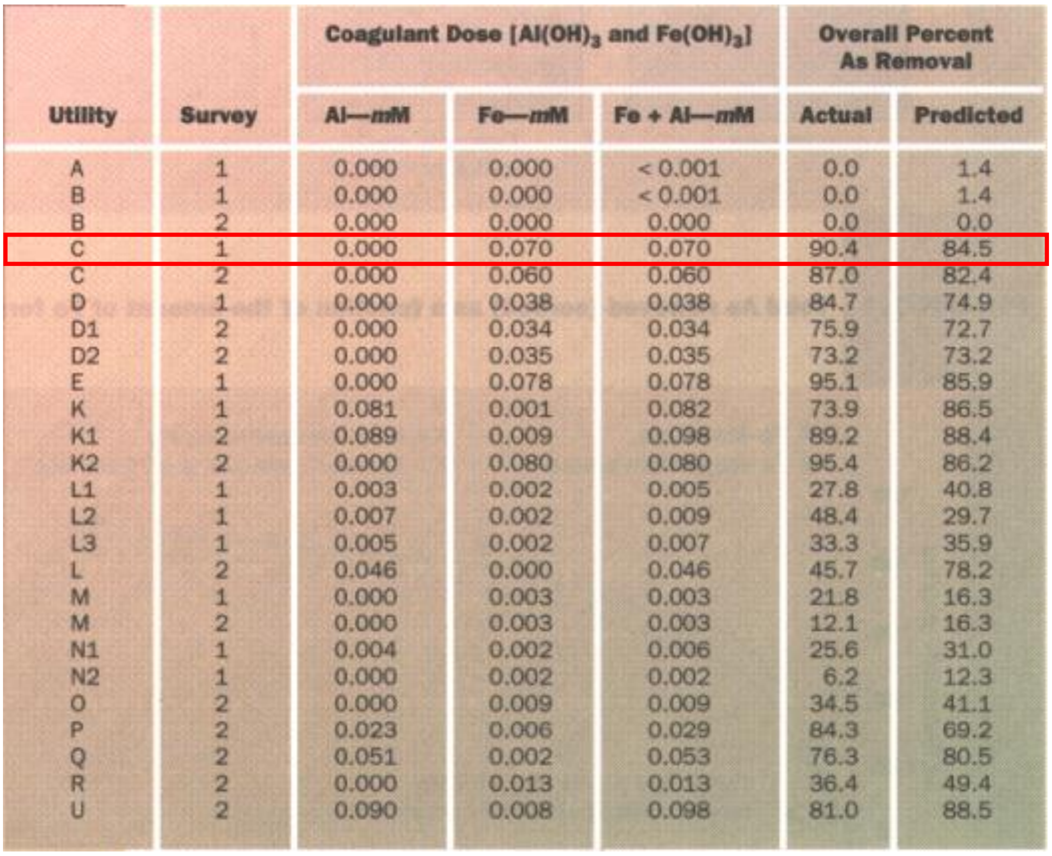


Pressure if new residential area is connected to low-pressure zone

APPENDIX D. ARSENIC REMOVAL

1.

Table 3 Summary of data used for simplified model from study *"Predicting As removal during metal hydroxide precipitation"* by McNeill in 1997:



Utility	Survey	Coagulant Dose [Al(OH)$_3$ and Fe(OH)$_3$]			Overall Percent As Removal	
		Al—mM	Fe—mM	Fe + Al—mM	Actual	Predicted
A	1	0.000	0.000	< 0.001	0.0	1.4
B	1	0.000	0.000	< 0.001	0.0	1.4
B	2	0.000	0.000	0.000	0.0	0.0
C	1	0.000	0.070	0.070	90.4	84.5
C	2	0.000	0.060	0.060	87.0	82.4
D	1	0.000	0.038	0.038	84.7	74.9
D1	2	0.000	0.034	0.034	75.9	72.7
D2	2	0.000	0.035	0.035	73.2	73.2
E	1	0.000	0.078	0.078	95.1	85.9
K	1	0.081	0.001	0.082	73.9	86.5
K1	2	0.089	0.009	0.098	89.2	88.4
K2	2	0.000	0.080	0.080	95.4	86.2
L1	1	0.003	0.002	0.005	27.8	40.8
L2	1	0.007	0.002	0.009	48.4	29.7
L3	1	0.005	0.002	0.007	33.3	35.9
L	2	0.046	0.000	0.046	45.7	78.2
M	1	0.000	0.003	0.003	21.8	16.3
M	2	0.000	0.003	0.003	12.1	16.3
N1	1	0.004	0.002	0.006	25.6	31.0
N2	1	0.000	0.002	0.002	6.2	12.3
O	2	0.000	0.009	0.009	34.5	41.1
P	2	0.023	0.006	0.029	84.3	69.2
Q	2	0.051	0.002	0.053	76.3	80.5
R	2	0.000	0.013	0.013	36.4	49.4
U	2	0.090	0.008	0.098	81.0	88.5

In this study values and treatment efficiency from various water works (utilities) were recorded to make a model predicting the percentage of As removed (the model is Figure 4.1 in the report). All the water works involved in this study are named as A through U as seen above. The value 0.07 mMol of Fe was taken as reference from the first survey of utility C, this has a As removal efficiency of 90%.

2.

Chlorine demand calculation

The following estimation of chlorine demand for pre-oxidation is based on "*Arsenic Treatment Technology Evaluation Handbook for Small Systems*" by the U.S. EPA, section 2.0 Arsenic mitigation strategies, 2.2 Pre-oxidation Processes, 2.2.1 Chlorine. All information can be directly referred to this handbook, except for the reaction equation of chlorine and ammonia, which was taken from the book Principles of Water Treatment by Howe, Kerry et al 2012, published by MWH Global, a water and natural resources firm.

Chlorine demand can be roughly calculated from stoichiometric ratio for oxidation of Fe(II), Mn(II), HS and Ammonia. The relevant reaction equations and the ratios calculated from them are:

Equations	Stoichiometric ratio
With As(III): $H_3AsO_3 + OCl^- \Rightarrow H_2AsO_4^- + H^+ + Cl^-$	0.95 mgCl^2/mg As(III)
With Fe(II): $2Fe^{2+} + OCl^- + 5H_2O \Rightarrow 2Fe(OH)_3 + Cl^- + 4H^+$	0.64 mgCl^2/mg Fe(II)
With Mn(II): $Mn^{2+} + OCl^- + H_2O \Rightarrow MnO_2 + Cl^- + 2H^+$	1.29 mgCl^2/mg Mn(II)
With HS^-: $HS^- + OCl^- \Rightarrow S^0 + Cl^- + OH^-$	2.21 mgCl^2/mg HS^-
With NH_3: $2NH_3 + 3HOCl \Rightarrow N_2 + 3H_2O + 3HCl$	6.26 mgCl^2/mg NH_3

From the ratio and the specific concentration of substances in Boring 5 water, the chlorine demand is calculated:

Parameters	Concentration (mg/l)	Theoretical Cl_2 demand (mg) for 1 mg of substance	Actual Cl_2 demand (mg/l)
As(III)	0.018	0.95	0.0171
Fe(II)	0.8	0.64	0.512
Mn(II)	0.5	1.29	0.645
HS	n/a (0)	2.21	0
NH_3	0.645	6.26	4
		Sum	5.2

Suppose 5.2 mg Cl_2/l is the ultimate chlorine demand in equation (1):

$$\delta_{Cl_2} = 3 \cdot D_{Cl_2}$$

Where:

δ_{Cl_2} = Chlorine Dose (mg/L as Cl_2), and

D_{Cl_2} = Ultimate Chlorine Demand (mg/L as Cl_2).

Then, the chlorine dose should be: 5.2*3 = 15.6 mg/l. So roughly the dose should be around this value. However, it is recommended in the handbook that simple bench testing is needed to be sure of the instantaneous and ultimate chlorine demand of the water.