

# An MS Excel tool for water distribution network design in environmental engineering education

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## Abstract

An MS Excel tool for teaching hydraulic analysis of water distribution networks (WDNs) was developed. The tool is able to perform both steady-state and extended period simulations of a WDN. The tool does not require installation and offers a portable solution to WDN design problem in undergraduate environmental engineering studies.

## KEYWORDS

environmental engineering, extended period simulation, steady-state analysis, water distribution networks

## 1 | INTRODUCTION

A water distribution network (WDN) is composed of sets of pipes, tanks and/or reservoirs, and other elements such as pumps, emitters as well as fittings. The elements in a WDN are arranged in a manner that is engineered to supply safe and clean water to cities. WDNs contain open and closed loops of pipes, which require sophisticated algorithms to calculate hydraulic equilibrium that exists in the network at a given time [6].

Since its first introduction, Microsoft Excel has been used extensively for education [2]. Over the past years, a great number of papers have been dedicated to the possible use of MS Excel in a number of engineering fields including

references [9,13,14,18]. MS Excel is usually available at least with a student license in most students' computers. It includes a simplified programming language (Visual Basic for Applications [VBA]), and for this reason, several researchers pointed out that MS Excel (and VBA) is the most suitable tool for engineering education within the scope of Bologna Process [17,19]. Considering complicated nature of WDN analysis, MS Excel is a perfect tool for teaching hydraulics of WDNs, and several researchers applied Excel-based solutions to the problem of WDN analysis [4,15,16].

Several methods have been proposed for hydraulic analysis of WDN including Hardy–Cross method [6,12], linear theory method [11,25], Newton–Raphson method [20], global gradient algorithm [23], co-tree flows method [7], and

**Abbreviations:**  $A$ , network topology matrix for links and nodes;  $b$ , base flow of a demand pattern;  $B$ , network topology matrix for links and fixed-head nodes;  $C$ , Hazen–Williams coefficient;  $D$ , internal diameter of a specified link;  $f$ , friction factor in a specified link;  $F$ , column vector of link flows in the network;  $g$ , gravitational acceleration;  $h$ , column vector of total headlosses in the network;  $h_f$ , friction loss in a specified link;  $h_L$ , total headloss in a specified link;  $h_m$ , minor loss at a specified element;  $H$ , column vector of heads at nodes;  $H_f$ , column vector of known-heads in the network;  $H_T$ , total head of water at a specified point;  $I$ , identity matrix;  $k$ , number of known-head nodes in the network;  $K$ , minor loss coefficient of a single element;  $L$ , length of a specified link;  $m$ , flow multiplier of a demand pattern;  $M$ , flow exponent matrix;  $n$ , number of unknown-head nodes in the network;  $N$ , cross-sectional area of a water tank;  $p$ , number of links in the network;  $P$ , pressure at a specified point;  $q$ , column vector of water demands at the nodes;  $Q$ , flowrate in a specified link;  $R$ , total loss resistance in a link;  $R_1$ , friction loss resistance in a link;  $R_2$ , minor loss resistance in a link;  $v$ , flow velocity at a specified link;  $x$ , water level in a tank;  $y$ , flowrate in the link connected to a water tank;  $y_1$ , flow exponent depending on the headloss method;  $z$ , elevation of a specified point;  $\rho$ , density of water;  $\mu$ , dynamic viscosity of water;  $\nu$ , kinematic viscosity of water.

fixed-point iteration method [24]. All of these methods aim to solve mass and energy balance equations written for a WDN with given topology, and perform steady-state analysis of the network. As a result of advances in computational technology, however, the design and operation of WDNs usually require time-dependent simulations of a WDN. In recent years, several software packages such as EPANET2 and WaterGEMS adapted extended period simulations for integrating the time variable to the hydraulic analysis to overcome the difficulty of solving the network under transient conditions. An extended period simulation of a WDN is a collection of steady-state solutions with nodal demands and water level in tanks varying with time. Demir et al. [6] reported a modified Hardy–Cross algorithm that integrates the time variable into the hydraulic analysis to obtain an extended period simulation of the given network.

Recently, Niazkar and Afzali [15,16] reported the use of MS Excel as an interface to a MATLAB code for WDN analysis. Although MATLAB could provide much faster solutions than MS Excel VBA, the software requires an installed copy of MATLAB, which may not be available to all students. Considering the fact that most of students who own a personal computer also own at least a student license of MS Excel and that MS Excel VBA is recognized as the most suitable tool for engineering education by a great number of researchers/teachers, a VBA program could be more helpful for teaching WDN analysis. Besides, several tools for teaching WDN hydraulics [4,15,16,25] perform only steady-state analysis of a given WDN. Therefore, new tools are required for teaching extended period simulations of WDNs.

This paper presents an educational MS Excel tool for teaching WDN design and operation in environmental engineering. The tool is called YTUSU, and is able to handle both steady-state and extended period simulations of a given WDN with time-dependent demand patterns as well as user-defined pumps and working schedules. A user-friendly interface is provided in MS Excel sheets that work in conjunction with the associated subroutines and functions in Visual Basic for Applications (VBA). Also, the solution to a sample WDN is presented to assess the tool's performance. The tool is prepared as open-source so that anyone with experience in WDNs and VBA programming can improve it.

## 2 | BACKGROUND

### 2.1 | Definitions

A WDN is composed of a set of circular pipes interconnected to each other in a manner that forms loops or branched parts. Each pipe, called a *link*, has its own length, roughness, and diameter. Besides, several parameters are attributed to pipes including pipe material and sum of minor loss coefficients. A

pipe is a link that defines the connectivity of nodal points in the network. Therefore, a “starting node” and an “ending node” are also attributed to a link. A node is a junction point of a set of links and is defined by its own elevation, hydraulic grade, and operating pressure.

A reservoir is defined by its constant hydraulic grade during the simulation while a tank has other attributes including cross-sectional area as well as water level and hydraulic grade that changes with time. A pump is a kind of link that includes a pump installed over the course of the pipe. Therefore, the pumps are treated similar to pipes.

The elevation, the pressure head, and the velocity head correspond to the terms in Bernoulli's trinomial and together sum up to total head of water at a specified point in a link as follows:

$$H_T = \frac{P}{\rho g} + \frac{v^2}{2g} + z \quad (1)$$

where  $H_T$  is the total head of water at a specified point in the link ( $\text{m H}_2\text{O}$ ),  $P$  is the pressure ( $\text{N} \cdot \text{m}^{-2}$ ),  $\rho$  is the density of water ( $\text{kg} \cdot \text{m}^{-3}$ ),  $g$  is the gravitational acceleration ( $9.81 \text{ m} \cdot \text{s}^{-2}$ ),  $v$  is the flow velocity in the link ( $\text{m} \cdot \text{s}^{-1}$ ), and  $z$  is the elevation of the specified point in the link ( $\text{m}$ ).

### 2.2 | Hydraulics

The first and the most important step in solving a WDN is calculating friction headloss through a pipe. The most commonly used method for this purpose is the well-known Darcy–Weisbach formula, which is given by

$$h_f = \frac{fL v^2}{D 2g} = \frac{8fLQ^2}{\pi^2 g D^5} \quad (2)$$

where  $h_f$  is the friction headloss through pipe ( $\text{m}$ ),  $f$  is friction factor (dimensionless),  $L$  is the pipe length ( $\text{m}$ ),  $D$  is the internal diameter of the pipe ( $\text{m}$ ), and  $Q$  is the flowrate through pipe ( $\text{m}^3 \cdot \text{s}^{-1}$ ). A great number of formulae have been proposed to calculate friction factor ( $f$ ) and several of them [1,3,5,8,10,21,22] were incorporated in YTUSU. Well-known Hagen–Poiseuille formula was used to calculate friction factor in laminar flow. In transient conditions, the friction factor is calculated by third-order interpolation from Moody's diagram. The friction factor is a function of Reynolds number ( $Re$ ) which is calculated using the following formula:

$$Re = \frac{\rho v D}{\mu} = \frac{4\rho Q}{\pi \mu D} \quad (3)$$

where  $\mu$  is the dynamic viscosity of water ( $\text{kg} \cdot \text{m}^{-1} \cdot \text{s}^{-1}$ ).

As an option, the friction headloss through a pipe can also be calculated by the well-known Hazen–Williams formula:

$$h_f = \frac{10.65LQ^{1.852}}{C^{1.852}D^{4.8704}} \quad (4)$$

where  $C$  is the Hazen–Williams coefficient (dimensionless).

In addition to friction headloss through a pipe, local headlosses also take place due to a number of fittings on the pipe. Local headloss through a pipe, called minor losses, is calculated by the equivalent pipe length method as follows:

$$h_m = \frac{v^2}{2g} \sum_i K_i = \frac{8Q^2}{\pi^2 g D^4} \sum_i K_i \quad (5)$$

where  $h_m$  is minor headloss through a pipe (m H<sub>2</sub>O), and  $K_i$  is the minor loss coefficient of the  $i^{\text{th}}$  fitting on the pipe (dimensionless).

Finally, the total headloss through a pipe can be calculated as the sum of friction and minor headlosses as follows:

$$h_L = R_1 Q^{y_1} + R_2 Q^2 \quad (6)$$

where  $h_L$  is the total headloss through a pipe (m H<sub>2</sub>O),  $R_1$  and  $R_2$  are resistance coefficients of the pipe due to friction and fittings, and  $y_1$  is the exponent depending on the method of headloss calculation employed ( $y_1 = 2$  for Darcy–Weisbach,  $y_1 = 1.852$  for Hazen–Williams).

### 2.3 | Steady-state simulation

In 1988, Todini and Pilati [23] proposed a method, lately known as global gradient algorithm (GGA), which is a fast and robust method for WDN analysis. Although co-tree flows method was reported to be 15–82% faster than GGA [7], GGA is widely accepted and incorporated in many software packages for WDN analysis. The MS Excel tool implemented in this study, YTUSU, employs the global gradient algorithm.

Global gradient algorithm starts by assuming a WDN of  $n$  nodes with unknown heads,  $k$  nodes with known heads, and  $p$  links with unknown flowrates that connects all nodes within the network. The method aims finding the  $n$  unknown heads (pressures) and  $p$  unknown flowrates (flows) iteratively.

For a given network topology, the nodal flows (demands) are expressed by mass conservation equation as

$$q = A^T F \quad (7)$$

where  $q$  ( $n \times 1$ ) is the vector of nodal demands,  $A^T$  ( $n \times p$ ) is transpose of  $A$ , and is a matrix that defines the system topology, and  $F$  ( $p \times 1$ ) is the vector of link flowrates.

Element of  $A^T$  in  $i^{\text{th}}$  row and  $j^{\text{th}}$  column is 1 if the  $i^{\text{th}}$  node is the ending node of the  $j^{\text{th}}$  link,  $-1$  if the  $i^{\text{th}}$  node is the starting node of the  $j^{\text{th}}$  link, and 0 if the  $i^{\text{th}}$  node is not connected to the  $j^{\text{th}}$  link.

For the network topology given by the topology matrix  $A^T$ , the energy conservation equation is written as

$$h = RF^y \quad (8)$$

where  $h$  ( $p \times 1$ ) is the vector of total headlosses in links,  $y$  is the exponent depending on the method of friction headloss calculation, and  $R$  ( $p \times p$ ) is the diagonal link resistance matrix. The flowrates in pipes (entries of  $F$ ) are assumed to be positive if the flow is from starting node to ending node, and negative if it is from ending node to starting node. Simultaneous solution of Equations (7) and (8) require at least one known nodal head within the network. Integrating the  $k$  nodes with known heads, one gets

$$RF^y + AH = -BH_f \quad (9.a)$$

where  $H$  ( $n \times 1$ ) is the vector of unknown nodal heads,  $B$  ( $p \times k$ ) is another topology matrix that is similar to  $A^T$  and that defines connections of  $k$  known-head nodes, and  $H_f$  ( $k \times 1$ ) is the vector of nodal heads of  $k$  known-head nodes. For linearizing purposes and taking the flow direction into consideration, the element in  $i^{\text{th}}$  row and  $j^{\text{th}}$  column of resistance matrix ( $r_{ij}$ ) is defined so that

$$r_{ij} = \begin{cases} 0 & \Leftrightarrow i \neq j \\ R_{1,i} F^{y_1-1} + R_{2,i} F & \Leftrightarrow i = j \end{cases} \quad (9.b)$$

and Equation (9.a) takes the form of

$$RF + AH = -BH_f \quad (9.c)$$

Equations (7) and (9.c) form a linear system of equations as follows:

$$\begin{bmatrix} R & A \\ A^T & 0 \end{bmatrix} \begin{bmatrix} F \\ H \end{bmatrix} = \begin{bmatrix} -BH_f \\ q \end{bmatrix} \quad (10)$$

Starting with an initial vector of  $F$  of arbitrarily selected link flowrates, the flowrates and pressures within the network are calculated iteratively as follows:

$$H^{i+1} = -[A^T M^{-1} R^{-1} A]^{-1} \{A^T M^{-1} [F^i + R^{-1} BH_f] + [q - A^T F^i]\} \quad (11.a)$$

$$F^{i+1} = [I - M^{-1}] F^i - M^{-1} R^{-1} [AH^{i+1} + BH_f] \quad (11.b)$$

Here,  $i$  is the iteration counter starting from zero,  $I (p \times p)$  is identity matrix, and  $M (p \times p)$  is a diagonal matrix. The diagonal elements of  $M$  are equal to  $y_1$ .

The tool randomly selects positive numbers between 0 and 1 for pipe flows in the network, that is, the tool assumes initial flowrates to be from starting node to ending node, and starts iteration for solving the system of equations based on unknown nodal heads and pipe flows.

## 2.4 | Extended period simulation

Extended period simulation is a collection of steady-state solutions at various times with varying nodal demands and pipe flows as well as varying water level in tanks. Several software packages like EPANET2 and WaterGEMS adapt this approach for simulating WDNs over time.

The global gradient algorithm by Todini and Pilati [23] was used for steady-state hydraulic simulation of the network. Then, steady-state solutions of the given network at various times are combined to form an extended period simulation.

The nodal demands within a WDN are the flows out of the system that are assumed to be concentrated at nodes. In operation, nodal demands change with time. In YTUSU, the nodal demands are assumed to be the sum of water demands for various purposes. The water demand at the  $i^{\text{th}}$  node at time  $t$  is calculated by

$$q_i(t) = \sum_j m_j(t) b_{ij} \quad (12)$$

where  $q_i(t)$  is the water demand at the  $i^{\text{th}}$  node at time  $t$ ,  $m_j(t)$  is the demand multiplier at time  $t$ , and  $b_{ij}$  is the base flow associated with the  $j^{\text{th}}$  demand pattern at the  $i^{\text{th}}$  node.

A number of user-defined demand patterns including residential, commercial, and industrial patterns are employed to calculate total water demand at a node at time  $t$ . With the nodal demands defined as a function of time, the total demand and the pipe flows within the network also changes with time. Therefore, water tanks are used for equalization purposes. The rate of change of water level in a water tank can be expressed by a first-order differential equation of the form

$$\frac{d_{xi}}{dt} = \frac{1}{N_i} \sum_j y_{ij} \quad (13)$$

where  $x_i$  is the water level in  $i^{\text{th}}$  tank,  $N_i$  is the cross-sectional area of the  $i^{\text{th}}$  tank,  $y_{ij}$  is the flowrate in  $j^{\text{th}}$  pipe connected to the  $i^{\text{th}}$  tank. The flowrate is assumed to be positive if entering to a tank and negative if leaving. During extended period simulation, Equation (13) is discretized and solved using flowrates calculated at each steady-state solution of the network.

## 3 | IMPLEMENTATION AND TESTING

### 3.1 | Implementation

The tool was implemented in MS Excel 2010 Visual Basic for Applications (VBA 7.0). It offers a multi-language support. Currently, available languages are English and Turkish. Also, a user's guide is prepared within the VBA code for developers to append a new language.

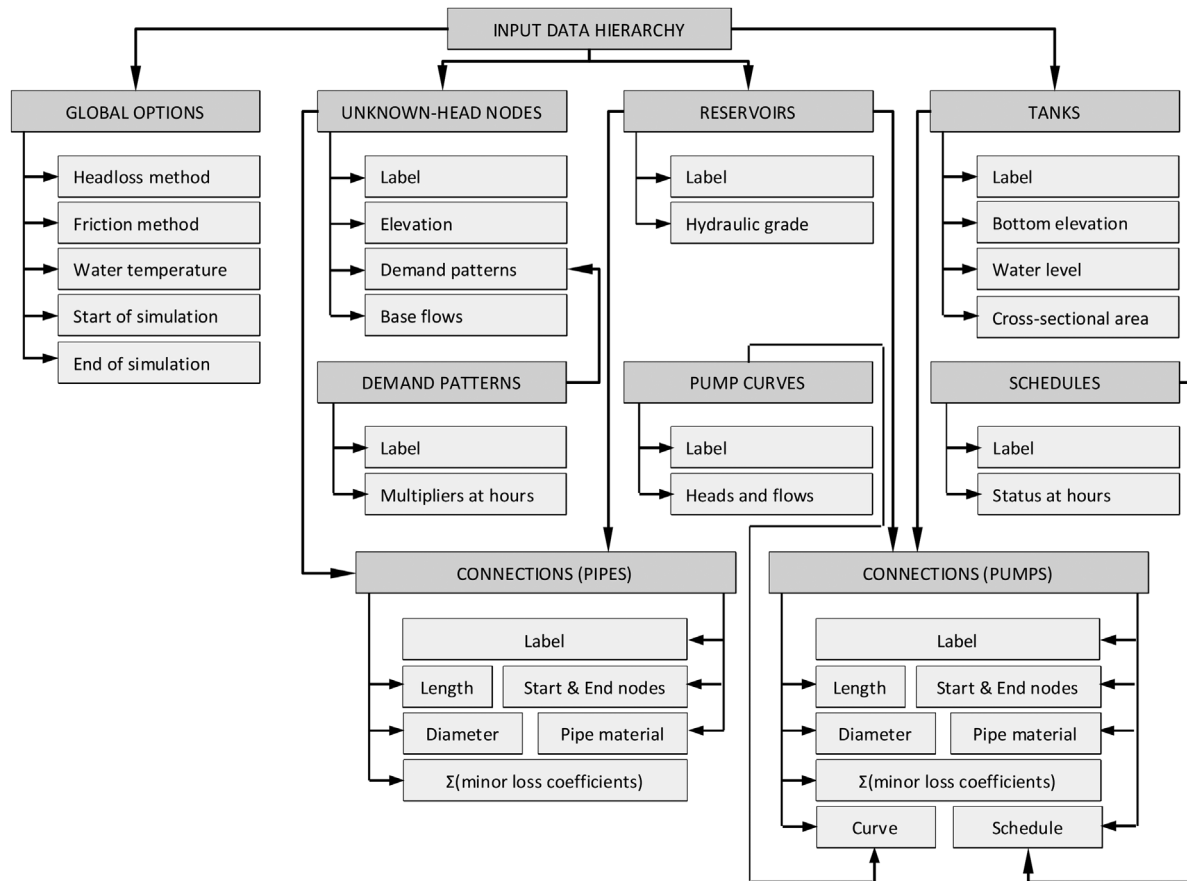
Figure 1 shows data hierarchy to handle a WDN. Of these, global options, unknown-head nodes, pipes, and at least one of reservoirs and tanks are compulsory for solving the network while patterns, pump curves, and pump working schedules are optional since built-in objects are already defined under these categories.

YTUSU consists of nine sheets for user inputs, a "Results" sheet, and a hidden sheet for storing lists of available headloss and friction methods, available pipe materials (for roughness heights and Hazen–William coefficients), and language options. Density and dynamic viscosity of water at temperatures between 0°C and 30°C are also stored on the hidden sheet. Figure 2 shows some of the sheets available.

The welcoming sheet is the "Patterns" sheet on which user can define any number of demand patterns. A demand pattern is defined by its unique label and flow multipliers at hours of a day. After defining the patterns, the user is asked to define pump curves and pump working schedules on "Curves" and "Schedules" sheets, respectively. A pump curve is defined by its unique label and three points, namely shutoff, operating, and maximum operating points, while a pump working schedule with its unique label defines when a pump starts and stops during a day.

After defining patterns and pump-related inputs, the user is required to define at least one reservoir or water tank on "Reservoirs" and "Tanks" sheets, respectively. For defining a reservoir, a unique label and a constant hydraulic grade is enough while a tank is defined by its unique label, cross-sectional area, bottom elevation, and initial water level in the tank.

A separate sheet "Nodes" is reserved for defining the unknown-head nodes in the network. A node is defined by its unique label, its elevation, as well as a number of demand patterns and associated base flows. The "Pipes" sheet is reserved for defining the connectivity within the network. A pipe has a unique label, a starting and ending nodes, a length, material, a diameter, and a minor loss coefficient. Since pumps are treated similar to pipes, the pumps are defined with the same properties through the "Pumps" sheet. For a pump, the tool requires two extra inputs as the pump curve, and the pump's working schedule. Upon completing the network definitions, the user can make a preliminary analysis of the



**FIGURE 1** Data hierarchy for defining a water distribution network

network for warnings and errors through the “Solver” sheet. Finally, the user selects simulation options including water temperature, headloss method, friction method, as well as starting hour and duration of the simulation. Figure 3 shows the solver and results sheets. The user can filter results at simulation times of his/her choice.

The VBA code can be classified under three main categories, namely error handling procedures, topology setting procedures, and calculation procedure. The error handling procedures are developed to minimize any kind of runtime errors due to wrong entries by the user. The tool handles the errors in two steps. The first step of error handling codes was implemented in object classes of the respective sheets in *Worksheet\_Activate*, *Worksheet\_Change*, and *Worksheet\_Deactivate* event procedures. These procedures prevent most of the errors originating from user inputs to minimize possible runtime errors. The second step of error handling is accomplished before setting the network topology to ensure no errors are raised in runtime due to user inputs.

The procedures for setting up the network topology are run before the calculation procedures. The main purpose of these procedures is to read the user input from the cells. They

store user inputs in appropriate network element variables and set up the necessary forms of matrices required for the solution algorithm.

The calculation procedure is implemented in a standard module in the main subroutine of the tool. It applies GGA using the outputs from topology setting procedures (topology and nodal demand matrices, etc.). The calculation procedure is also responsible for reporting the network flows and pressures after the simulation is complete. The time variable is implemented in the main subroutine as an outer loop to the GGA algorithm.

### 3.2 | Testing

The WDN of an imaginary town with an industrial zone, sparse commercial zones, and medium-to-high-density residential zones was designed using YTUSU as a sample network. The network consists of open and closed loops, and is composed of one reservoir, one water tank (service tank), 13 unknown-head nodes, 16 pipes (secondary pipes are excluded from the simulation), and a main transmission line equipped with a pump. A schematic view of the sample network is shown on Figure 4.

	A	B	C	D	E	F	G	H	I	
1	<b>PUMPS</b>									
2	LABEL	PUMP CURVE	SCHEDULE	NODES		LENGTH	<i>Yeni ekle..</i>	DIAM.	ΣK	
3				START	END	(m)	MATERIAL	(mm)		
4	PUMP-1	STANDARD	12-SAAAT	RES-1	TANK-1	300	Paslanmaz çelik	500	0	
5		STANDARD								
6		CRV-1							(a)	

	A	B	C	D	E	F	G	H	I	J	K	
1	<b>UNKNOWN-HEAD NODES</b>											
2	LABEL	ELEVATION	PATTERN	BASE FLOW	PATTERN	BASE FLOW	PATTERN	BASE FLOW	PATTERN	BASE FLOW	PATTERN	
3		(m)		(L/s)		(L/s)		(L/s)		(L/s)		(L/s)
4	A	30	CONSTANT	10								
5	B	30	CONSTANT	10								
6	C	30	RESIDENTIAL	10								
7	D	30	INDUSTRIAL	10								
8	E	30	COMMERCIAL	10								
9	F	30	INSTITUTIONAL	10								
10			HOSPITAL									
11			CONSTANT	10							(b)	

	A	B	C	D	E	F	G	H	I	J	
1	<b>PIPES</b>										
2	LABEL	NODES		LENGTH	<i>Add new..</i>	DIAM.	ΣK				
3		START	END	(m)	MATERIAL	(mm)					
4	P1	TANK-1	A	150	Paslanmaz çelik	200	0				
5	P2	RES-1		50	Paslanmaz çelik	150	0				
6	P3	TANK-1		50	Paslanmaz çelik	100	0				
7	P4	A		20	Paslanmaz çelik	150	0				
8	P5	B		50	Paslanmaz çelik	100	0				
9	P6	C		20	Paslanmaz çelik	200	0				
10	P7	D		50	Paslanmaz çelik	200	0				
11	P8	E		20	Paslanmaz çelik	200	0				

**FIGURE 2** Sheet view of YTUSU. (a) Pumps sheet, (b) nodes sheet, and (c) pipes sheet

The reservoir is the clean-water storage tank of a water treatment plant with a constant hydraulic grade of 212.00 m and is connected to the service tank with a 3-km main transmission line (stainless steel).

For the sake of simplicity here, following assumptions are made for the sample network:

- In order to reduce the number of variables to be considered in the design procedure, estimation of the most economically feasible diameter of the main transmission line was skipped and the diameter was assumed to be 550 mm.
- Although the pumping stations are designed so that a number of pumps are operated in parallel to meet the maximum flowrates, it is assumed that the pumping station contains only one pump.
- The location of the water tank is already selected. The bottom elevation of the water tank is 279.00 m. The water level in the tank is limited to 6.00 m by legislations and it is not allowed to drop below 2.00 m to provide water for fire-fighting purposes.
- The operating pressures within the network are limited between 30.00 m H<sub>2</sub>O and 80.00 m H<sub>2</sub>O by legislations.
- The velocities within the pipes are limited between 0.5 m/s and 1.5 m/s under worst-case conditions.
- Water is required in the town to meet residential, commercial, and industrial demands as well as to meet water consumption in schools, hospitals, and other purposes such as park and garden maintenance. Daily patterns for the town are available as in Figure 5. Also, water demands at nodal points are available (Table 1).
- The water level in service tank is 3.12 m at the beginning of the simulation, which is 0:00 AM. The duration of the extended period simulation is taken as 1 week (168 hr).
- Effects of fire flows on operating pressures are neglected. Fire-flow analyses are excluded from the design procedure.
- The network is defined by the major pipelines in the system, that is, the system is skeletonized. Also, the pipe lengths are known from Figure 4.

With these simplifying assumptions, the design problem involves determining the pipe diameters, selecting a suitable pump for the system, and dimensioning the water tank for sustaining the operating pressures at nearly constant levels. An optimal solution to this problem can be obtained in three steps.

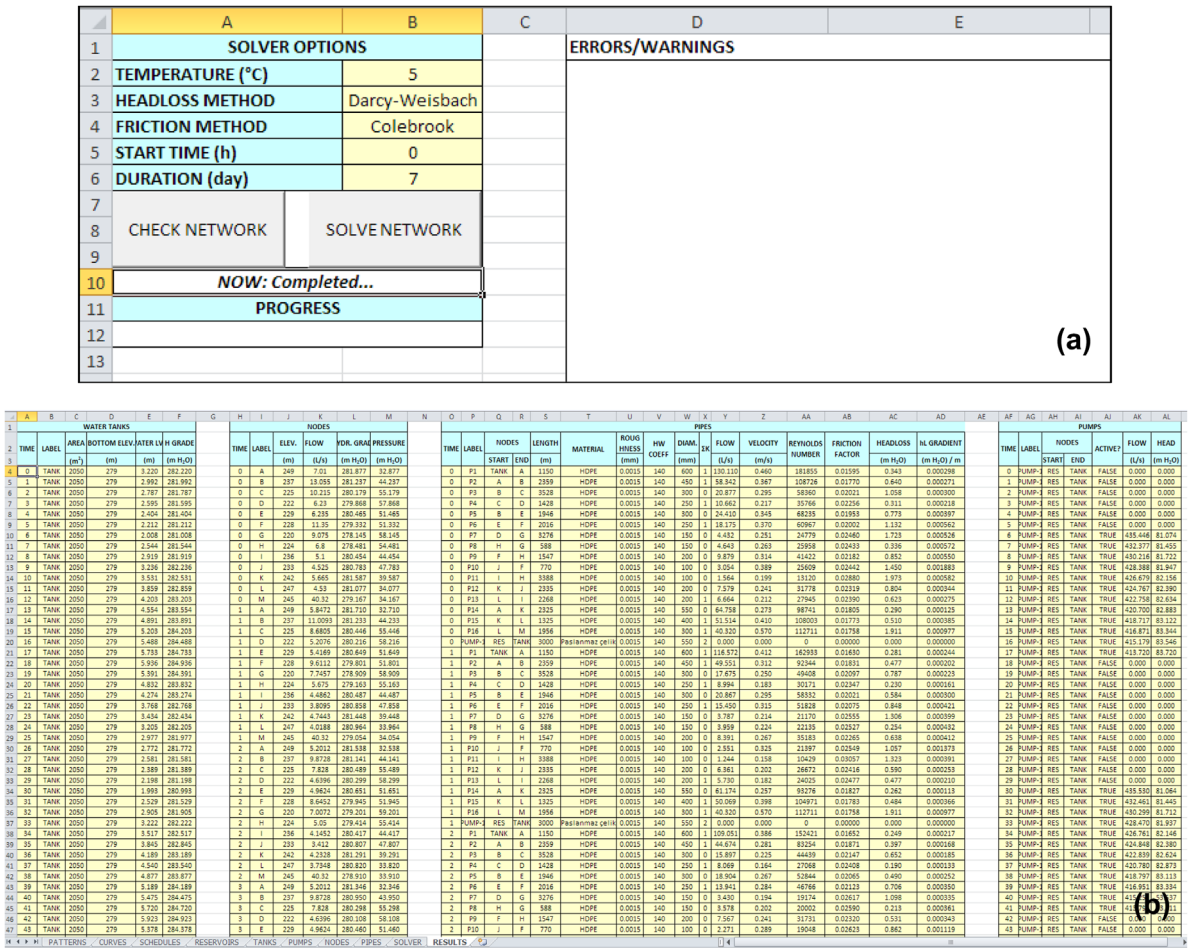


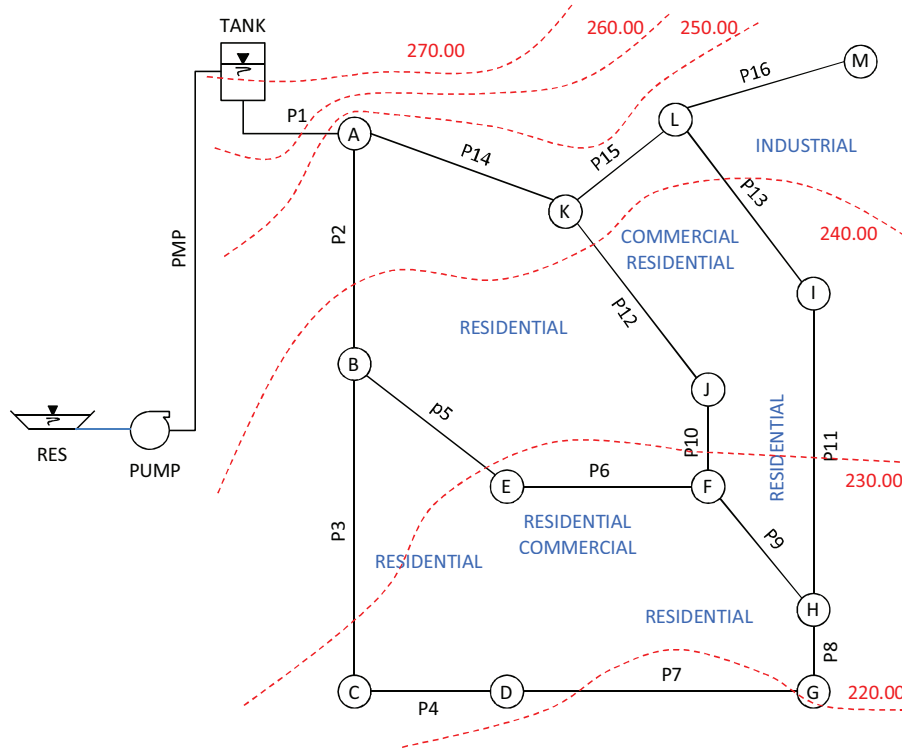
FIGURE 3 The sheet view of (a) solver options and (b) results sheet

The first step is to determine minimum pipe diameters that will keep the operating pressures within the desired range of 30.00–80.00 m H<sub>2</sub>O all the time. For this purpose, steady-state analyses of the network must be performed. For determining the worst-case conditions, hourly water demands based on demand patterns and associated base flows at nodes (Table 1) as well as pattern multipliers (Figure 5) are calculated. The maximum and minimum hourly water demands on the maximum day are calculated as 317.5 L/s at 07:00 PM and 108.5 L/s 02:00 AM, respectively, which points out that worst-case conditions prevail around 07:00 PM. Thus, starting with a random set of pipe diameters, the network was simulated a few times to obtain minimum pipe diameters. The procedure is trivial and one could get the best results after a few trials. All the network pipes are assumed to be of high-density polyethylene (HDPE) with associated minor loss coefficients given in Table 2. After a few steady-state simulation of the network, the minimum pipe diameters shown in Table 2 were obtained that satisfies operating pressure limitations under the worst-case conditions. It is worth to note that, for determining minimum pipe diameters, the water level in the tank must be set to its minimum

allowable value to represent the worst-case conditions, which is 2.00 m.

The second step of the solution is to find out a suitable pump that is capable of pumping daily water demand from the reservoir to the water tank. By finding a suitable pump in this context, the minimum pump curve is referred. For this purpose, the network must be simulated at least 2 days. The procedure is trivial and one could easily select a minimum pump curve after a few trials. Since the suitable pump is the one that is capable of pumping daily water demand, an assumption of 12 hr of operation a day for the pump means that the pump must have the capacity of two times daily water demand. The procedure starts with assuming that the pump is operational at least 06:00 AM to 06:00 PM. With this assumption and a random cross-sectional area of the water tank, it is possible to define a suitable pump as the one that will lead to equal water levels in the water tank at the same hours of any 2 days. After trial and error, the minimum three-point pump curve was found out as follows: Shut-off head = 110 m H<sub>2</sub>O, maximum operating flow = 3,200 m<sup>3</sup>/hr, design flow = 1,600 m<sup>3</sup>/hr, and design head = 80 m H<sub>2</sub>O.

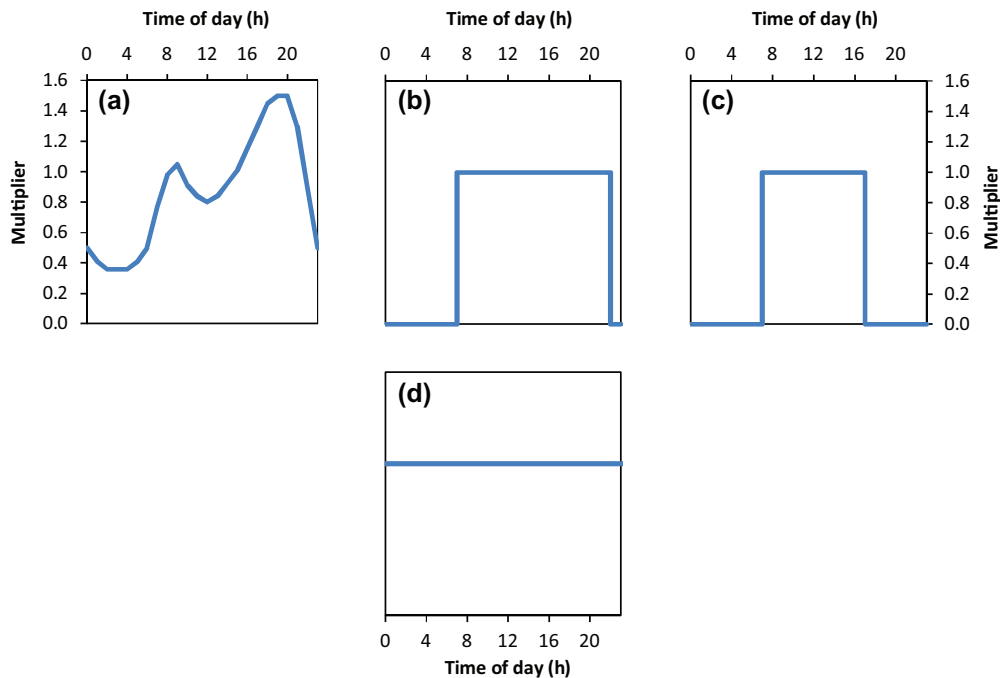




**FIGURE 4** Sample water distribution network

With pipe diameters and a suitable pump selected, the final step of the design procedure is to find optimum cross-sectional area of the water tank. For this purpose, extended period simulations of the network with selected pipe diameters and pump curve must be performed during a period of at least 1 day.

In each trial, a various cross-sectional area is used and the change of water level during the day is plotted. The cross-sectional area can be selected from plotting as the minimum cross-sectional area of the water tank that limits the water level between 2.00 m and 6.00 m. For the sample problem, the



**FIGURE 5** Multipliers for pattern data. (a) Residential; (b) commercial; (c) school; (d) hospital, industrial, and other demand patterns



**TABLE 1** Nodal elevations and demands

Label	Elevation (m)	Water demand patterns and base flows (L/s) <sup>a,b</sup>
A	249.00	D_R(12.92) + D_S(1.14) + D_O(0.57)
B	237.00	D_R(22.73) + D_S(1.14) + D_H(1.14) + D_O(0.57)
C	225.00	D_R(17.05) + D_S(1.14) + D_H(1.14) + D_O(0.57)
D	222.00	D_R(11.36) + D_C(3.41) + D_O(0.57)
E	229.00	D_R(9.09) + D_C(5.68) + D_S(2.27) + D_H(1.14) + D_O(0.57)
F	228.00	D_R(19.32) + D_C(1.14) + D_S(1.14) + D_H(1.14) + D_O(0.57)
G	220.00	D_R(14.77) + D_S(1.14) + D_H(1.14) + D_O(0.57)
H	224.00	D_R(12.50) + D_S(1.14) + D_O(0.57)
I	236.00	D_R(6.82) + D_C(9.09) + D_H(1.14) + D_O(0.57)
J	233.00	D_R(7.95) + D_C(11.36) + D_O(0.57)
K	242.00	D_R(10.23) + D_C(6.82) + D_S(1.14) + D_O(0.57)
L	247.00	D_R(5.68) + D_H(1.14) + D_O(0.57)
M	245.00	D_I(39.77)

<sup>a</sup>D\_R: residential, D\_C: commercial, D\_S: schools, D\_H: hospital, D\_I: industrial, D\_O: other.

<sup>b</sup>Values in parentheses represent base flows.

change of water level with time for various cross-sections of the water tank is shown in Figure 6. Clearly, the minimum cross-sectional area that meets the minimum and maximum allowable water levels in the tank is 2050 m<sup>2</sup>.

A final note on the three-step solution procedure is that since the head gained in the pump and the difference of hydraulic grades in the water tank and reservoir are interdependent, one can assume that the second and the third steps must be solved iteratively, especially if randomly selected cross-sectional area of the tank in the second step is far from the optimum value. However, this problem can be overcome with intuition and good engineering practice.

After the design procedure, the network was simulated for a period of 7 days. Expected minimum and maximum velocities in network pipes are shown in Table 3. Turkish legislations on WDNs clearly state that the velocities in network pipes be between 0.5 m/s and 1.5 m/s at design flowrates, which represents the worst-case conditions. In the current problem, the worst-case conditions are considered to happen around 07:00 PM. It is obvious in Table 3 that the pipe diameters are sufficient to meet the design criteria. On the other hand, minimum velocities were calculated below the legal limit. However, the velocity limits are for the highest water demand in the network and lower velocities are

**TABLE 2** Pipes and pumps in the network

Label	Connection <sup>a</sup>	Length (m)	Diameter (mm) <sup>b</sup>	ΣK	Material
P1	TANK—A	1,150	600	1.2	HDPE
P2	A—B	2,359	450	0.5	HDPE
P3	B—C	3,528	300	0.4	HDPE
P4	C—D	1,428	250	0.6	HDPE
P5	B—E	1,946	300	0.4	HDPE
P6	E—F	2,016	250	0.6	HDPE
P7	D—G	3,276	150	0.2	HDPE
P8	H—G	588	150	0.2	HDPE
P9	F—H	1,547	200	0.2	HDPE
P10	J—F	770	100	0.2	HDPE
P11	I—H	3,388	100	0.1	HDPE
P12	K—J	2,335	200	0.4	HDPE
P13	L—I	2,268	200	0.6	HDPE
P14	A—K	2,325	550	0.3	HDPE
P15	K—L	1,325	400	0.8	HDPE
P16	L—M	1,956	300	0.6	HDPE
PMP <sup>c</sup>	RES—TANK	3,000	550	2.0	Stainless steel

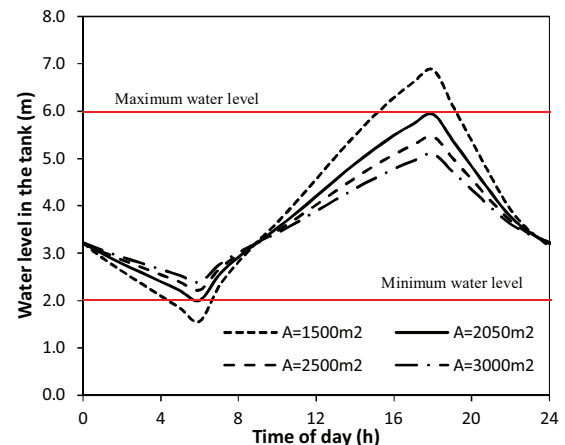
<sup>a</sup>Flow is assumed to be from starting node to ending node.

<sup>b</sup>Determined by several steady-state analyses under worst-case conditions to find minimum pipe diameters to sustain desired operating pressures.

<sup>c</sup>Diameter is chosen so that the velocity in the pipe is around 1 m/s.

expected during night hours when the demand is negligibly low compared to daytime demands.

The most important criteria for the design of a WDN are operating pressures. The operating pressures within the network must be within the allowable range all the time. Maximum and minimum operating pressures within the network were calculated during a simulation period of 7 days

**FIGURE 6** Change of water level during the day for various cross-sectional areas of the tank

**TABLE 3** Maximum and minimum velocities in pipes during simulation

Label	Minimum		Maximum	
	Velocity (m/s)	Day, hour	Velocity (m/s)	Day, hour
P1	0.386	6, 03:00 AM	1.125	2, 07:00 PM
P2	0.281	3, 03:00 AM	1.059	7, 07:00 PM
P3	0.225	3, 03:00 AM	0.853	1, 08:00 PM
P4	0.164	3, 03:00 AM	0.672	1, 08:00 PM
P5	0.267	3, 03:00 AM	1.024	7, 07:00 PM
P6	0.284	3, 03:00 AM	1.047	7, 07:00 PM
P7	0.194	3, 03:00 AM	0.679	1, 08:00 PM
P8	0.202	7, 04:00 AM	0.670	2, 07:00 PM
P9	0.241	6, 04:00 AM	0.845	4, 08:00 PM
P10	0.289	6, 04:00 AM	0.888	4, 08:00 PM
P11	0.135	3, 03:00 AM	0.586	5, 07:00 PM
P12	0.181	6, 04:00 AM	0.981	4, 08:00 PM
P13	0.166	3, 03:00 AM	0.815	5, 07:00 PM
P14	0.249	7, 02:00 AM	0.546	4, 08:00 PM
P15	0.392	3, 03:00 AM	0.606	5, 07:00 PM
P16	0.570	2, 05:00 AM	0.570	2, 02:00 PM

and are shown in Table 4. It is obvious that operating pressures at nodes were always between the allowable range.

### 3.3 | Future work

YTUSU has been developed for the sole purpose of teaching the basics of WDN hydraulics, design, and operation to environmental engineering students. Currently, the tool is able to handle both steady-state and extended period

simulations of a WDN composed of a set of unknown-head nodes, reservoirs, tanks, pumps, and pipes laid out in a looped or branched manner.

Authors think that adding following features should be a priority for future improvements of the tool.

- Integrating various kinds of valves such as isolation valves and pressure regulating valves in the solution algorithm to extend the tool's capabilities.

**TABLE 4** Maximum and minimum operating pressures during simulation

Label	Elevation (m)	Minimum		Maximum	
		Pressure (m H <sub>2</sub> O)	Day, hour	Pressure (m H <sub>2</sub> O)	Day, hour
A	249.00	31.60	7, 06:00 AM	34.27	1, 06:00 PM
B	237.00	40.69	7, 08:00 PM	44.45	1, 11:00 PM
C	225.00	45.62	7, 08:00 PM	55.49	1, 02:00 AM
D	222.00	46.28	7, 08:00 PM	58.30	1, 02:00 AM
E	229.00	43.25	7, 08:00 PM	51.68	1, 11:00 PM
F	228.00	36.94	7, 08:00 PM	51.95	1, 02:00 AM
G	220.00	38.20	7, 08:00 PM	59.20	1, 02:00 AM
H	224.00	35.97	7, 08:00 PM	55.41	1, 02:00 AM
I	236.00	37.14	7, 08:00 PM	44.67	1, 11:00 PM
J	233.00	38.21	7, 08:00 PM	48.00	1, 11:00 PM
K	242.00	37.94	7, 07:00 AM	40.30	1, 04:00 PM
L	247.00	32.12	7, 07:00 AM	34.41	1, 03:00 PM
M	245.00	32.21	7, 07:00 AM	34.50	1, 03:00 PM

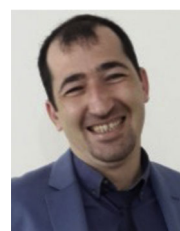
- Integrating procedures for calculating water age and performing water quality analyses.

## 4 | CONCLUSIONS

An MS Excel tool, called YTUSU (abbreviated from the Turkish name Yıldız Technical University Water), for designing WDNs is implemented using Visual Basic for Applications. Turkish and English language supports are built-in features and instructions were included in the tool for appending a new language. The tool is open-source and can be used for academic purposes. It employs global gradient algorithm for solving a given WDN. Data input and output are accomplished via Excel sheets. Also, a sample WDN design using the tool is presented in this paper. Considering the fact that most students who own a personal computer also owns a licensed copy of MS Excel, the tool provides advantages over several other software packages including it is open-source, it does not require installation, it works on a computer with MS Excel installed, and system requirements are minimal. The tool can be used for not only teaching design courses pertaining to WDNs but also helping the students better understand principles of design and operation without time-consuming calculations. The tool can be downloaded from academic web pages of authors ([www.yarbis.yildiz.edu.tr/seldemir](http://www.yarbis.yildiz.edu.tr/seldemir)).

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