PUBLIC WORKS

PRACTICAL MANUAL FOR FACULTY OF THE CIVIL ENGINEER

Made by Department of Sanitary and Environmental Engineering in Budapest University of Technology and Economics

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FOREWORD

The aim of the subject is that the students get acquainted with the dimension of the sanitary public works. They need to carry out 3 public works conceptual plans which are the water supply, sewage, storm water system on the given design area. The design methods are showed according the European Unions and Hungarians directives. The base data of the design is given in the worksheet. The design area is on the distributed map A4 and A3 format. The maps are found on the ftp site of the department (ftp://152.66.18.2/English). The method of the design follows the next steps:

1. Collection of the data
2. Determination of the loads
3. Determination of horizontal path of the public works
4. Determination of vertical path of the public works
5. Hydraulically design of the network
6. Hydraulically analysis of the network
7. The showing of the finally parameters and the calculated result (thematic layout maps, longitudinal sections)
8. Technical description of the network

Tasks to be completed in the given-in design work

- In a file folder with front page:
- The file folder contains the followings:
    o Work-sheet,
    o Table of contents,
    o Attachments,
    o Technical specification and detailed calculations,
    o Layout maps and longitudinal sections for designing

During the design process the student should consult with the practice lesson teacher at least 5 times, which is proved by the signature of the practice lesson teacher.

1. DETERMINATION OF DESIGNING WATER DEMAND (1ST PRACTICAL LESSON)

Every design task start the determination of the loads. The water demand depend on the type of the area, custom of the inhabitants. We need distinguish the design area. Is it a real area with water supply or new area without water supply? In first case we can determine the water demand from the real measured consumption and production. The calculation of peak water demand is important for the designing. We need distinguish the design area. It is a real area with water supply or new area without water supply. In first case we can determine the water demand from the real measured consumption and production. In this project we don't know the real consumption so we

use the values of the directives. This data can be find in the datasheet. The load has three parts in this task:

- inhabitant,
- public institution and
- industrial water demands
- firefighting water (this is given).

DOMESTIC WATER DEMAND

The domestic water demands are the consumption of the inhabitants. It is a distributed load in most case not a concentrate. In this task we calculated for a bigger area, or for the blocks of the parcels.
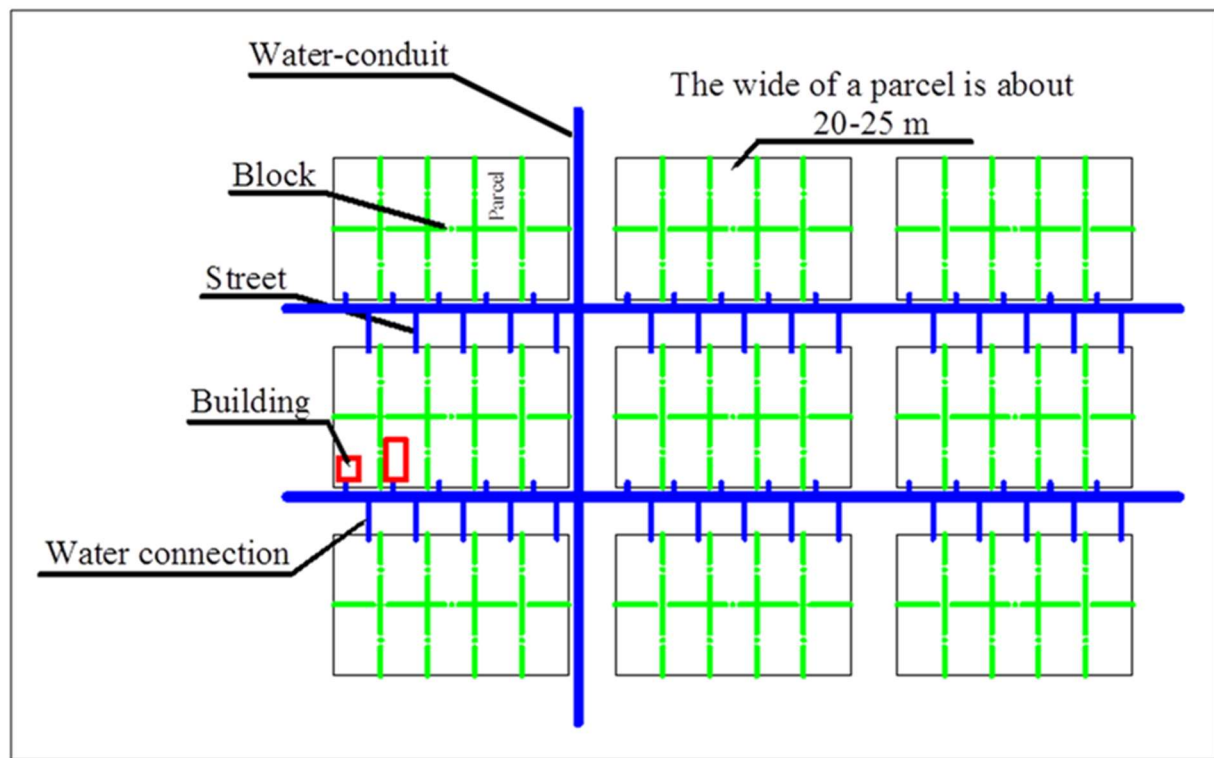
Used definitions:

Figure 1.: Determination of the number of the connection pipe

Count of Connection: We have got some blocks in our designing area, the block is a border of grounds/building sites. The buildings are on these grounds. Every building has a water connection or binding in. The distance of the grounds is 15-20 meter between themselves. The connections join on the one hand to the ground, on the other hand to the water distribution pipe. The distance of connection depends on the type of settlement. This distance between two connections is larger in a village than in a city. So we have to determine the count of connection (Fig. 1).

Dweller density: shows us the number people who joins to the binding-in. For example so many people live in a flat in average.

Number of people: This number is calculated from dweller density, and from count of connections (multiply)

Specific water demand (q) is a feature value of the consumption on the actually area. This value depends on the habit of dweller. This value is higher in a rich settlement, than in a poor one.

Average daily water demand (Q_{avg}): This value applies to actually designing area. This value is yearly average daily water demand of an area.

Seasonal ratio (β) shows us the difference between the daily average and the daily peak water demand. Because we have higher water demand on summer, than all-year.

Peak water demand (Q_{max}): is highest water demand of people in a year. We can calculate this value from average water demand, and from seasonal ratio. (multiply)

Loss coefficient: Every system has loss so has the water supply system some water loss too. This loss is 10-30 percent in the Hungarian water system. So we have to put this loss to the peak water demand. This loss comes from break of pipe, from leak, from cleaning of system, disinfection of the system. This value is pertain of system.

Designing peak water demand (Q_{design}): is increased value of peak water demand by the loss. We use this value to calculate the needed pipe size.

Count of Connections (piece)	dweller density (person/binding-in)	people of designing area (head)	specific water demand (l/person/day)	average water demand (m3/day)	seasonal ratio $\beta\,(-)$	peak water demand (m3/day)	loss coefficient	designing peak water demand (m3/day)
600	3	1800	150	270	1.4	378	1.1	**415.8**

Table 1. Demand calculation of the inhabitants

PUBLIC INSTITUTIONS WATER DEMAND

We have a public institution on our planning area. We have to choose a block on our planning area, whereon we say that is a public institution. The public institution block has just one connection. We haven't got to add this blocks connection to the dweller connection. We assume there is a grammar school. So There are some workers and lot of students. The student and the teacher have got different water demand. In this part of task, we haven't got to define the people number, because it is available. The calculating of water demand is like the dweller demand.

Profession of people	Number of people	specific water demand (l/person/day)	average water demand (m3/day)	seasonal ratio	peak water demand (m3/day)	loss coefficient	designing peak water demand (m3/day)
worker	60	150	9	1.2	10.8	1.1	**11.88**
student	450	130	58.5	1.2	70.2	1.1	**77.22**

Table 2. Water demand calculation of the public institution

THE INDUSTRIAL WATER DEMAND

We have an industrial area in our planning area. We have to choose a block on our planning area, whereon we say that is a industry. The industry block has just one connection. We haven't got to add this blocks connection to the dweller connection. The industry has two water demands one side technological demand other side the social water demand. The technological water is used to product fabricating. We calculate the social water demand as the dweller. We don't have to define the worker number because it is available. We assume in this task that the technological water demand doesn't vary in the year. It is fix. There are some branches of industry, where the seasonal ratio differs from 1.00 for example the cannery. The seasonal ratio of technological water demand is 1.00 in this task.

Technological water demand (m3/day)	Loss coefficient	designing peak water demand (m3/day)
1000	1.1	**1100**

Table 3. Industrial technical water demand

Number of People	specific water demand (l/person/day)	average water demand (m3/day)	seasonal ratio	peak water demand (m3/day)	loss coefficient	designing peak water demand (m3/day)
400	150	60	1.2	72	1.1	**79.2**

Table 4. Industrial social water demand

The full water demand of the design area: *1672.22* m3/d

2. PATH OF THE WATER SUPPLY NETWORK (2ND PRACTICAL LESSON)

The water supply system was built by transit, main, distribution, and connection pipe. Topology of the system can be branch or loop. In most case the main and distribution pipes create loops. The loops give higher safety level for the supply than it created from branches.

We design only one main-loop in our area. First we have to draw our one-loop main conduits line. The conduits must be laid down on the area of common use. We don't lay it on the private area, or below the buildings. The main loop must touch the consumption centers and the concentrated big consumer as the public institution and industrial area. It must connect to the given inlet point, which will be a source in the hydraulic model.

3. TOPOLOGICAL MODEL OF THE WATER SUPPLY NETWORK (2ND PRACTICAL LESSON)

We have to divide the created main ring into some parts, about 7-8 parts if we want to create the hydraulic model. These parts join with nodes. We set the nodes at the breakpoint (but it isn't necessary), and at the branch joint, and at the inlet node. (Fig. 2). We have to take on the nodes at every great consumer (for example the industry, the public institution), because they have concentrated consumption in our model. The water demands of the big consumers influence significantly the hydraulic state.
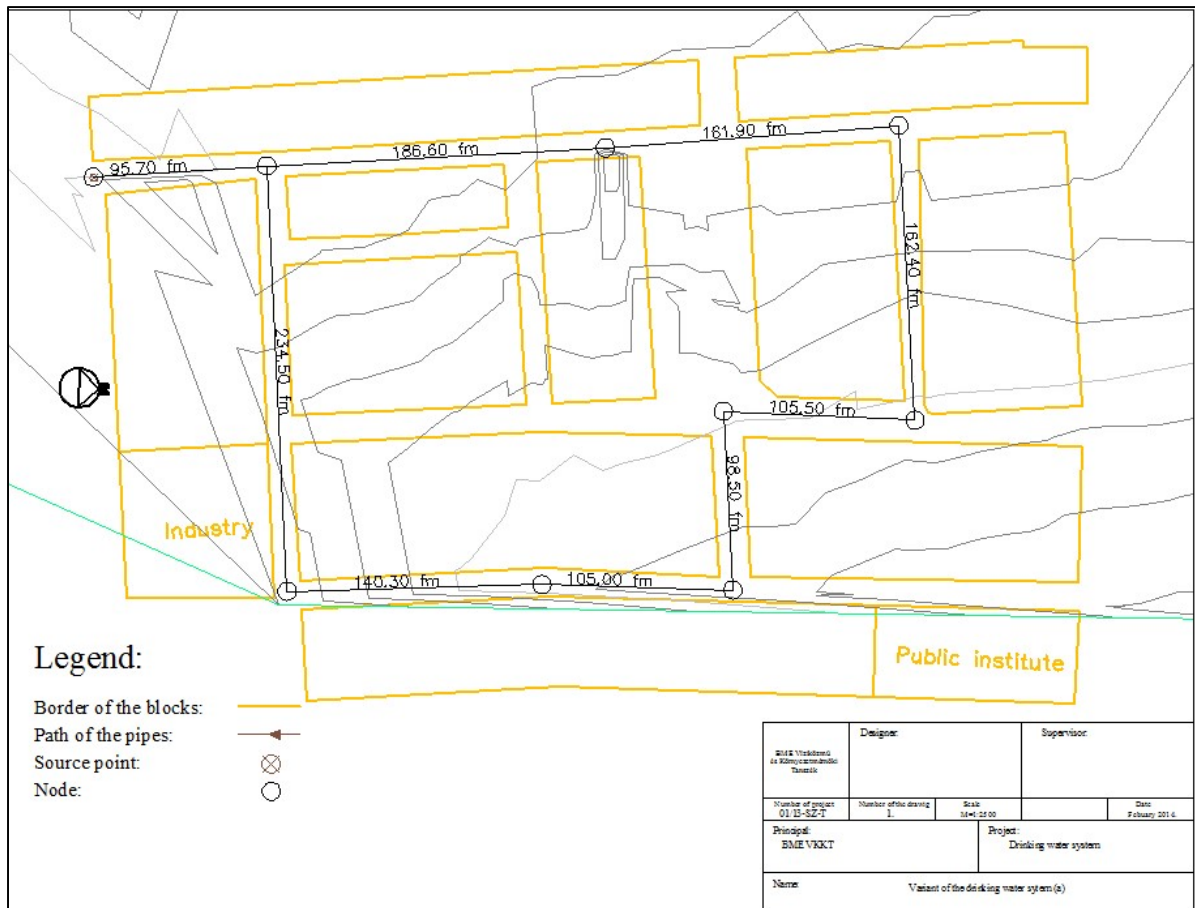
Figure 2. The path of the drinking water system (The main-loop)

We consider the communal consumption an uniform distributed water consumption on our main loop. We have to order this uniform distribution consumption to the nodes. We carry out this distribution on the basis of conduits length or the area which belongs to the current node. In this case we distribute the consumption (415.8 m³/d) according to the length in our example. You can see this calculation in the table 5. We need to measure the length of the branches to the calculation. We assume that the water demand on the branch is uniformly. So we can order the half water demand to the one and the other half to the other node (Table.6).

Branches	Lenght (m)	Q branches (m3/d)	Q node 1	Q node 2
1-2	95	30.6	15.3	15.3
2-3	235	75.7	37.8	37.8
3-4	140	45.1	22.5	22.5
4-5	105	33.8	16.9	16.9
5-6	99	31.9	15.9	15.9
6-7	106	34.1	17.1	17.1
7-8	162	52.2	26.1	26.1
8-9	162	52.2	26.1	26.1
9-2	187	60.2	30.1	30.1
sum	1291	415.8		

Table 5. The order of the dweller consumption to the links

Node	Branch									Node (m3/d)
	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-2	
1	15.3									15.3
2	15.3	37.8							30.1	83.3
3		37.8	22.5							60.4
4			22.5	16.9						39.5
5				16.9	15.9					32.9
6					15.9	17.1				33.0
7						17.1	26.1			43.2
8							26.1	26.1		52.2
9								26.1	30.1	56.2

Table 6. The distribution of the dweller consumption to the nodes

Node	Qdweller m3/d	Qpublic ins. m3/d	Qind. Soc. m3/d	Qpeople m3/d	Qind. Tech. m3/d	Qfire l/min
1	15.3			15.3		
2	83.3			83.3		
3	60.4		200	260.4	1100	
4	39.5			39.5		
5	32.9	300		332.9		
6	33.0			33.0		
7	43.2			43.2		600
8	52.2			52.2		
9	56.2			56.2		
szum	416	300	200	916	1100	
				2016		

Table 5. The full daily water consumption at the nodes.

4. HYDRAULICAL MODEL OF THE WATER SUPPLY NETWORK (3RD PRACTICAL LESSON)

The water system has typical states, for example maximum and minimum consumption, fire event, burst in a water pipe. This event influences the behaviour of the water system (pressure, water velocity). The different states have different pressure value and flow quantity and transporting direction. The typical states are:

- Maximum demand
- Minimum demand
- Average demand (4,17%)+ fire water
- Pipe failure

We design our system for the standard state, which is the hourly peak demand in this case. (In some case the firefighting state is the standard). We assume that, the maximum hourly water

demand is 8 percent of the daily water demand (table 6). (The water consumption value is changing during the day). So we multiply the daily people water demand by 8 per cent. We have to compute the nodes consumption in the typical states (Table 6, 7, 8.).

Node	Q people m3/d	Qind tech m3/d	Q fire l/perc	minimal consumption (1 %)			
				Q people	Qind. tech	Qfire	Qcsp
				l/s			
1	15.3	0	0	0.04			0.04
2	83.3	0	0	0.23			0.23
3	260.4	1100	0	0.72			0.72
4	39.5	0	0	0.11			0.11
5	332.9	0	0	0.92			0.92
6	33.0	0	0	0.09			0.09
7	43.2	0	600	0.12			0.12
8	52.2	0	0	0.14			0.14
9	56.2	0	0	0.16	0.00		0.16
Qcons							2.54

Table 6. The minimum hourly water consumption at the nodes.

Node	Q people m3/d	Qind tech m3/d	Q fire l/perc	peak consumption (8 %)			
				Q people	Qind. tech	Qfire	Qcsp
				l/s			
1	15.3	0	0	0.34			0.34
2	83.3	0	0	1.85			1.85
3	260.4	1100	0	5.78			5.78
4	39.5	0	0	0.88			0.88
5	332.9	0	0	7.39			7.39
6	33.0	0	0	0.73			0.73
7	43.2	0	600	0.96			0.96
8	52.2	0	0	1.16			1.16
9	56.2	0	0	1.25	0.00		1.25
Qcons							20.33

Table 7. The maximum hourly water consumption at the nodes.

	Q people	Qind tech	Q fire	check (4.17 %)+fire			
				Q people	Qind. tech	Qfire	Qcsp
Node	m3/d	m3/d	l/perc	l/s			
1	15.3	0	0	0.18			0.18
2	83.3	0	0	0.96			0.96
3	260.4	1100	0	3.01			3.01
4	39.5	0	0	0.46			0.46
5	332.9	0	0	3.85			3.85
6	33.0	0	0	0.38			0.38
7	43.2	0	600	0.50		10.00	10.50
8	52.2	0	0	0.60			0.60
9	56.2	0	0	0.65	0.00		0.65
Qcons							20.60

Table 8. The average hourly water and firefighting consumption at the nodes.

When we distributed the consumption upon the nodes we can determine the hydraulic parameters in the ring-mains. The hydraulic parameters are the next:

- pressure at the nodes
- flow quantity in the branches
- flow direction in the branches
- water velocity in the branhes

5. INITIAL WATER TRANSORT IN THE WATER SUPPLY NETWORK (3ʳᵈ PRACTICAL LESSON)

The determination of the water transport and its direction are complicate in a water supply network. The cause of this thing is the looped network. The flow direction is not clear in the loop. We use the first Kirchoff's law to determine the initial water flow.

First Kirchoff's law (node balance): The sum of the waters quantities, what flows in the node and out the node that is null.

We put on a flow direction as we wish, but we have one regulation the first Kirchoff's law the low of node. So we get the flow quantity (fig. 3). We can make branches from the loop. (We cut a conduit between two nodes. So we can calculate the directions of the flows. The flow of the cut branch is null)

Branch	Qin (l/s)	Node	Qnode (l/s)	branches	Qbranch (l/s)
	20.33	1	0.34	1-2	20.00
1-2	20.00	2	1.85	2-3	9.07
2-3	9.07	3	5.78	3-4	3.29
3-4	3.29	4	0.88	4-5	2.42
4-5	2.42	5	7.39	5-6	-4.98
5-6	-4.98	6	0.73	6-7	-5.71
6-7	-5.71	7	0.96	7-8	-6.67
7-8	-6.67	8	1.16	8-9	-7.83
8-9	-7.83	9	1.25	9-2	-9.07
9-2	-9.07				

Table 9. Initial flow in peak flow
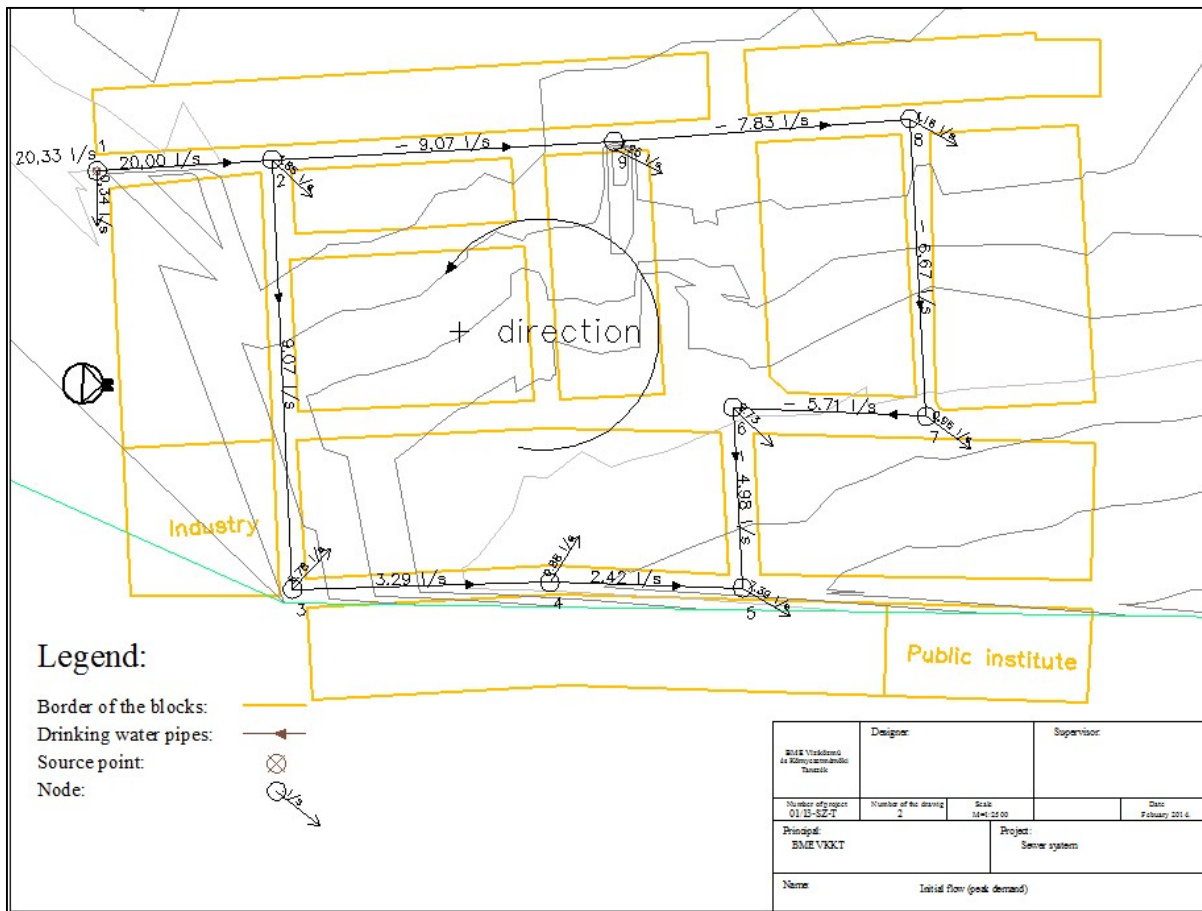
Figure 3. The initial water flow and direction

We set the initial diameter of conduits. We assume the velocity is 1 m/s in the pipes. We use the next equation:

$$Q = v * A$$

The minimum diameter is 100 mm in distribution system according the general rules (firefighting). If a node has two direction input, it can be 80 mm to.

6. REAL HYDRAULIC PARAMETERS IN THE WATER SUPPLY NETWORK (4TH PRACTICAL LESSON)

The initial water flow is not the real flow. We need to calculate the real flow. We use the second Kirchoff's low.

Second Kirchoff's law: We take on a circuity direction in our loop. The sums of the signed flow pressure loss must be null along the loop.

The formula of flow pressure loss is in the next:

$$hv = \lambda * \frac{L}{D} * \frac{v^2}{2 * g}$$

$$v = Q/A$$

$$hv = L * \frac{16 * \lambda}{D^5 * \pi^2 * 2 * g} * Q^2$$

$$hv = L * c * Q^2$$

$$hv = C * Q^2$$

Where:

D: Diameter of pipe (m)
L: lenght of conduit (m)
v : velocity (m/s)
g : 9,81 m/s2
hv: local loss (in the pipe)
Q: water flow (m3/s)

The „c" values, what belongs to different pipe diameter you can see on *table 9*.

λ= 0,033

Diameter	c
(mm)	
100	272,667353531
150	35,906811988
200	8,520854798
250	2,792113700
300	1,122087875
400	0,266276712
500	0,087253553
600	0,035065246
800	0,008321147
1000	0,002726674
1200	0,001095789
1400	0,000506982
1600	0,000260036
1800	0,000144301
2000	0,000085209

Table 9: *„c" values*

We analyse our loop that is it adequate for the second Kirchoff's law. We define the failure of the sum of the pressure flow loss. If it is not null then we correct our flows quantity (Table 10). Then we calculate the flows pressure loss again, until the failure will null, this is the Cross method (Table 11, 12). Finally we get the right flow direction, and quantity (Table 13). The result of the calculation is showed by table 14. We can see the right flow direction, the local flows pressure loss and the water velocity in the branches.

1. iteration

Branches	Lenght (m)	Diameter (mm)	main-ring	Sign	C=c*l [s2/m5]	Q[m3/s]	2abs(CQ)	C*Q*abs(Q)	Σ2absCQ	ΣCQ2	Dq
1-2	96	200	branch	1	818	0.0200	32.6984	0.3268			
2-3	235	150		1	8438	0.0091	153.3809	0.6970	153.3809	0.6970	
3-4	140	100		1	38173	0.0033	251.0154	0.4126	404.3963	1.1097	
4-5	105	100		1	28630	0.0024	137.3889	0.1648	541.7852	1.2745	
5-6	99	100	one-loop	1	26994	-0.0050	270.0339	-0.6753	811.8191	0.5992	
6-7	106	100		1	28903	-0.0057	332.0996	-0.9540	1143.9186	-0.3548	
7-8	162	100		1	44172	-0.0067	593.4061	-1.9929	1737.3248	-2.3478	
8-9	162	150		1	5817	-0.0079	91.8129	-0.3623	1829.1377	-2.7101	
9-2	168	150		1	6032	-0.0091	109.6510	-0.4983	1938.7887	-3.2083	0.0017
									failure	correction water quantity	

Table 10: *The pressure loss in the loop (according to the initial flow)*

2. iteration

Q (m3/s)	Q[m3/s]	2abs(CQ)	C*Q*abs(Q)	Σ2absCQ	ΣCQ2	Dq
0.0200	0.0200					
0.0107	0.0107	181.3079	0.9739	181.3079	0.9739	
0.0049	0.0049	377.3554	0.9326	558.6633	1.9065	
0.0041	0.0041	232.1438	0.4706	790.8071	2.3771	
-0.0033	-0.0033	180.6935	-0.3024	971.5006	2.0747	
-0.0041	-0.0041	236.4422	-0.4836	1207.9428	1.5911	
-0.0051	-0.0051	447.2127	-1.1319	1655.1555	0.4592	
-0.0062	-0.0062	72.5611	-0.2263	1727.7166	0.2329	
-0.0074	-0.0074	89.6862	-0.3334	1817.4028	-0.1004	0.0001

Table 11: Second iteration

3. iteration

Q (m3/s)	Q[m3/s]	2abs(CQ)	C*Q*abs(Q)	Σ2absCQ	ΣCQ2	Dq
0.0200	0.0200					
0.0108	0.0108	182.2405	0.9840	182.2405	0.9840	
0.0050	0.0050	381.5746	0.9535	563.8151	1.9375	
0.0041	0.0041	235.3082	0.4835	799.1233	2.4210	
-0.0033	-0.0033	177.7099	-0.2925	976.8333	2.1285	
-0.0040	-0.0040	233.2476	-0.4706	1210.0809	1.6579	
-0.0050	-0.0050	442.3305	-1.1074	1652.4114	0.5506	
-0.0062	-0.0062	71.9182	-0.2223	1724.3296	0.3283	
-0.0074	-0.0074	89.0195	-0.3284	1813.3490	-0.0001	0.0000

Table 12: third iteration

4. iteration

Q (m3/s)	Q[m3/s]	2abs(CQ)	C*Q*abs(Q)	Σ2absCQ	ΣCQ2	Dq
0.0200	0.0200					
0.0108	0.0108	182.2416	0.9840	182.2416	0.9840	
0.0050	0.0050	381.5793	0.9536	563.8209	1.9375	
0.0041	0.0041	235.3118	0.4835	799.1326	2.4211	
-0.0033	-0.0033	177.7066	-0.2925	976.8392	2.1286	
-0.0040	-0.0040	233.2441	-0.4706	1210.0833	1.6580	
-0.0050	-0.0050	442.3250	-1.1073	1652.4083	0.5507	
-0.0062	-0.0062	71.9175	-0.2223	1724.3258	0.3284	
-0.0074	-0.0074	89.0187	-0.3284	1813.3445	0.0000	0.0000

Table 13: Fourth iteration

Branches	Lenght (m)	Diameter (mm)	Q (m3/s)	Q (l/s)	water velocity (m/s)	local loss C*Q*abs(Q) (m)	specific pressure flow loss %o
1-2	96	200	0.0200	19.99	0.64	0.3268	3.4
2-3	235	150	0.0108	10.80	0.61	0.9840	4.2
3-4	140	100	0.0050	5.00	0.64	0.9536	6.8
4-5	105	100	0.0041	4.11	0.52	0.4835	4.6
5-6	99	100	-0.0033	-3.29	-0.42	-0.2925	-3.0
6-7	106	100	-0.0040	-4.03	-0.51	-0.4706	-4.4
7-8	162	100	-0.0050	-5.01	-0.64	-1.1073	-6.8
8-9	162	150	-0.0062	-6.18	-0.35	-0.2223	-1.4
9-2	168	150	-0.0074	-7.38	-0.42	-0.3284	-2.0

Table 14: The result of calculation

Than we check the ring, is it adequate for the hydraulically criteria:

- The pressure must be about 45-60 meter (4,5-6,0 bar) above the surface.
- The water velocity must be about 0,5-1,50 m/s.
- the pressure loss must be less than 10‰ (10m/km)

We can get the pressure at the nodes, if we start from the connection node where the pressure is known, what is about 45-60 meter (4,5-6,0 bar) above the surface and we add the signed flow pressure loss on the path (table 15). The pressure decrease in that direction whither the water flows. The pressures must be higher than 20 meter or buildings height + 10 meter at the node above the surface. The smallest allowed conduit is 100 mm (80 mm) in the water supply system. If the water system is not adequate for the pressure demand or water velocity, we have to change the pipe diameter.

Node 1		Branches		Node 2		Surface	Pressure above the surface	Pressure demand	Pressure
	H (mBf)		C*Q*abs(Q)		H (mBf)	(mBf)	h (m)	H(m)	
1	190	1-2	0.3268	2	189.67	149.8	39.87	25	OK
2	189.67	2-3	0.9840	3	188.69	150.1	38.59	25	OK
3	188.69	3-4	0.9536	4	187.74	151.2	36.54	25	OK
4	187.74	4-5	0.4835	5	187.25	155.7	31.55	25	OK
5	187.25	5-6	-0.2925	6	187.54	156.5	31.04	25	OK
6	187.54	6-7	-0.4706	7	188.02	154.7	33.32	25	OK
7	188.02	7-8	-1.1073	8	189.12	155.5	33.62	25	OK
8	189.12	8-9	-0.2223	9	189.34	151.2	38.14	25	OK
9	189.34	9-2	-0.3284	2	189.67	151.3	38.37	25	OK

Table 15: *The result of pressure calculation*

If we determined the system parameters to the maximum hourly consumption than we have to check the behaviour of the system in the minimum consumption and average consumption + firefighting too. (We don't modify the diameters of the pipes).

The results are shown in pressure longitudinal profile and layout map.
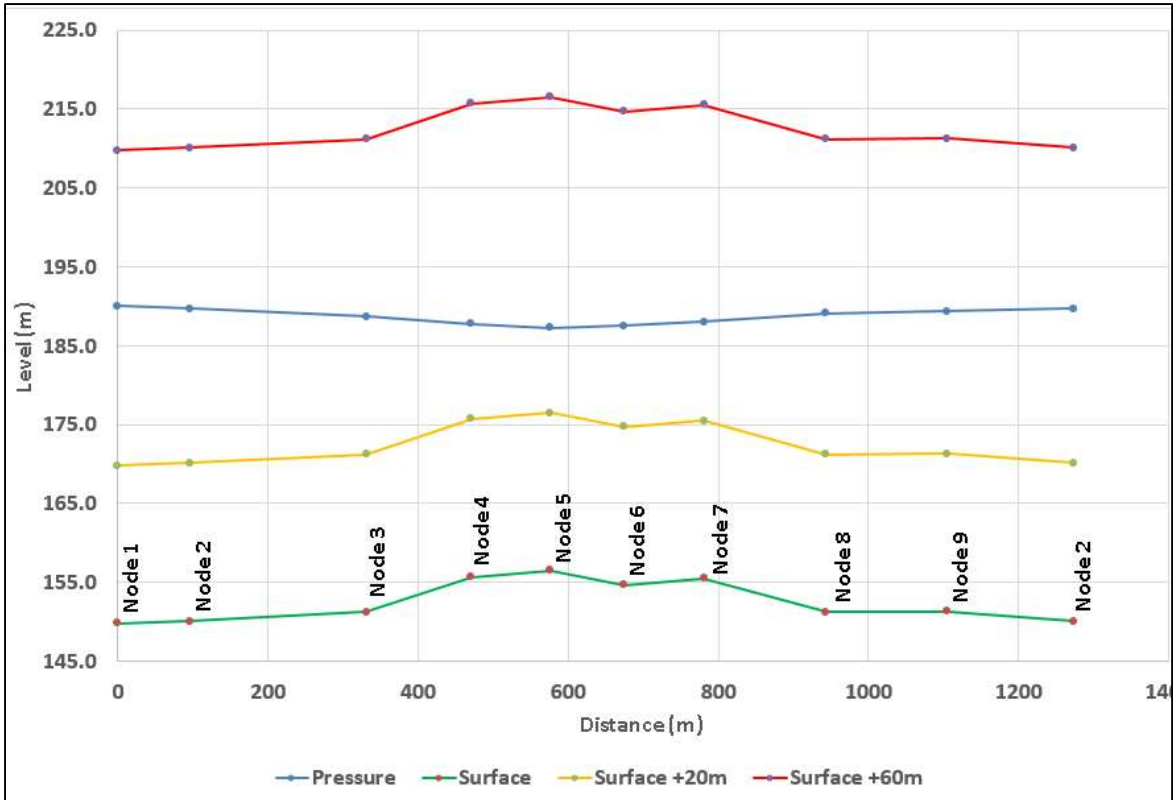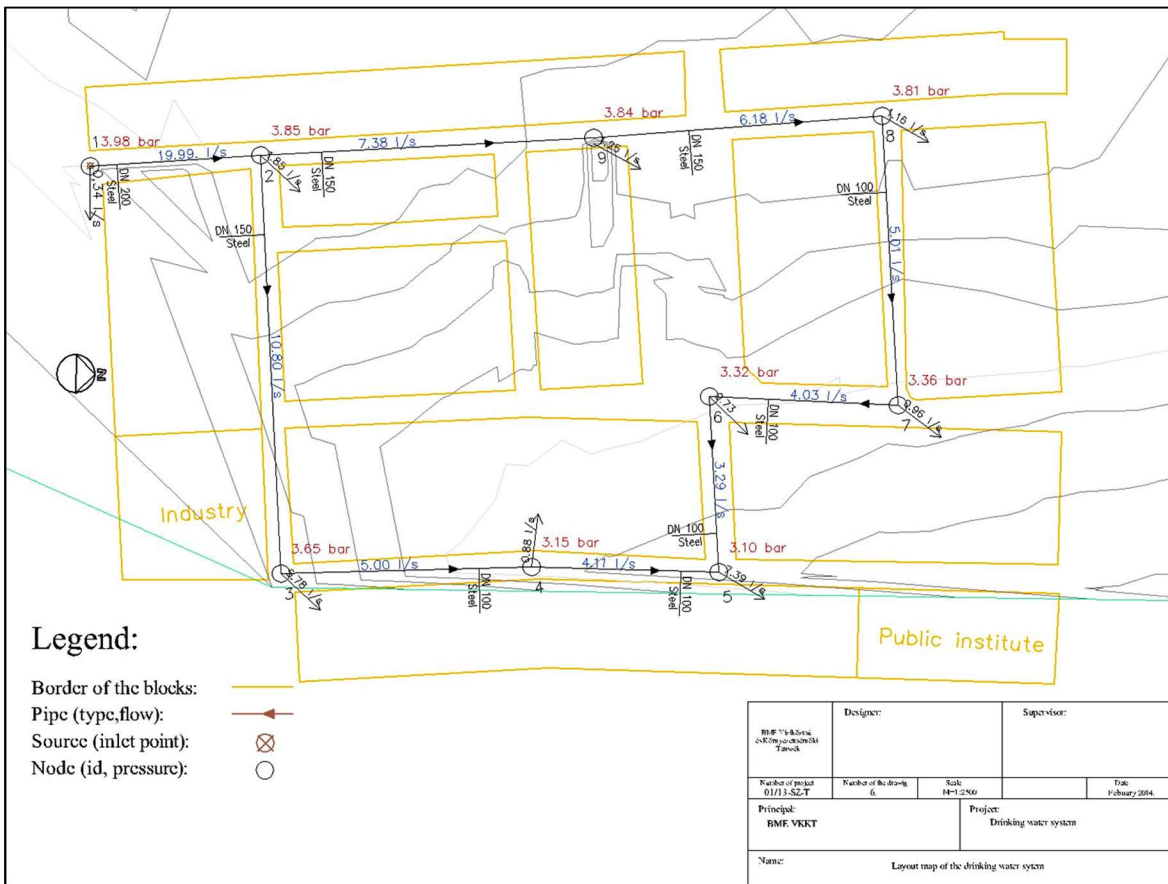
Figure 4. Pressure longitudinal profile

Figure 5. Final layout map

7. DOCUMENTATTION OF THE RESULTS(5TH PRACTICAL LESSON)

We have to present our results in our system. We have to make some thematic layout maps:

The final designed water supply system:

- The path of the pipes
- The types of the pipes (material, diameter)
- The nodes id number (according to the calculation)
- The absolute level of the nodes
- The consumption zones (industrial area, public institution, private area) with specific demand data

State of the water supply system (minimum, maximum, average + firefighting):

- The water demands at the nodes
- The right flow direction in the pipes
- The water velocity or quantity of water flow in the pipes
- The relative pressure at nodes
- The diameter of conduits

Longitudinal profile for the pressure

- Vertical axis is the height
- Horizontal axis the distance (with the nodes ID)
- Line of the surface
- Pressure lines in different state

8. WASTEWATER LOADS (6TH PRACTICAL LESSON)

Sewerage systems can be classified into combined sewerage and separate sewerage. Combined sewerage carries both stormwater and wastewater, while separate sewerage carries stormwater or wastewater separately. We plan a gravity separated sewerage system. First we have to determine the load of the sewer system on the area. The load derives from three different sources as the water supply system. The waste water was derived from the drinking water consumption. The waste water pipe is dimensioned for maximum waste water load. The maximum waste water load is calculated with following value:

- the peak water demand without loss
- waste-water fraction
- hourly-peak ratio

Waste water types	maximum water consumption (m3/day)	Waste water fraction / sewage ratio	Maximum waste water quantity (m3/day)	hourly-peak ratio	hourly maximum waste water quantity (m3/hour)
dweller	378	0.9	340.2	0.056	18.9
public institution	89.1	0.8	71.28	0.056	3.96
technological water	1100	0.6	660	0.042	27.5
social water	79.2	0.95	75.24	0.100	7.524

Table 16: *The wastewater loads*

The peak water demand: We use this value and not the designing water, because the loss of water won't waste water.

Waste-water fraction/sewage ratio: This value shows us how much proportion of water will waste water. It depends on the applications mode of water. This value is lower for example in a village, where the water is used for watering than in a downtown.

Hourly peak ratio/Peak flow coefficient show us how many proportion of daily waste water flows in the peak hour in the drainage. This value is varied by habits of the people. For example this value in a small village is 1/10 in a big city is 1/22.

9. PATH OF THE WASTEWATER SYSTEM (6TH PRACTICAL LESSON)

The sewer system was built by collector, main collector, and connection pipe. Topology of the system must be branch. We design only collector and main collector gravity pipe on our area. The principle of using gravity as the driving force for conveying wastewater in a sewerage system should be applied wherever possible, because this will minimise the cost of pumping. Catchment area is an area draining to a drain, sewer or watercourse (Standard EN 752:2008). Crossing a catchment boundary may mean that the water has to be unnecessarily pumped, requiring an energy source. In first step we need to draw the catchments and subcatchments according to the surface. The catchment is bounded by the watershed. We can draw the borderline of the catchment if we analyze the surface counter lines. First we have to draw our main collector conduits line. We need draw 2 path variant (Fig 4.).
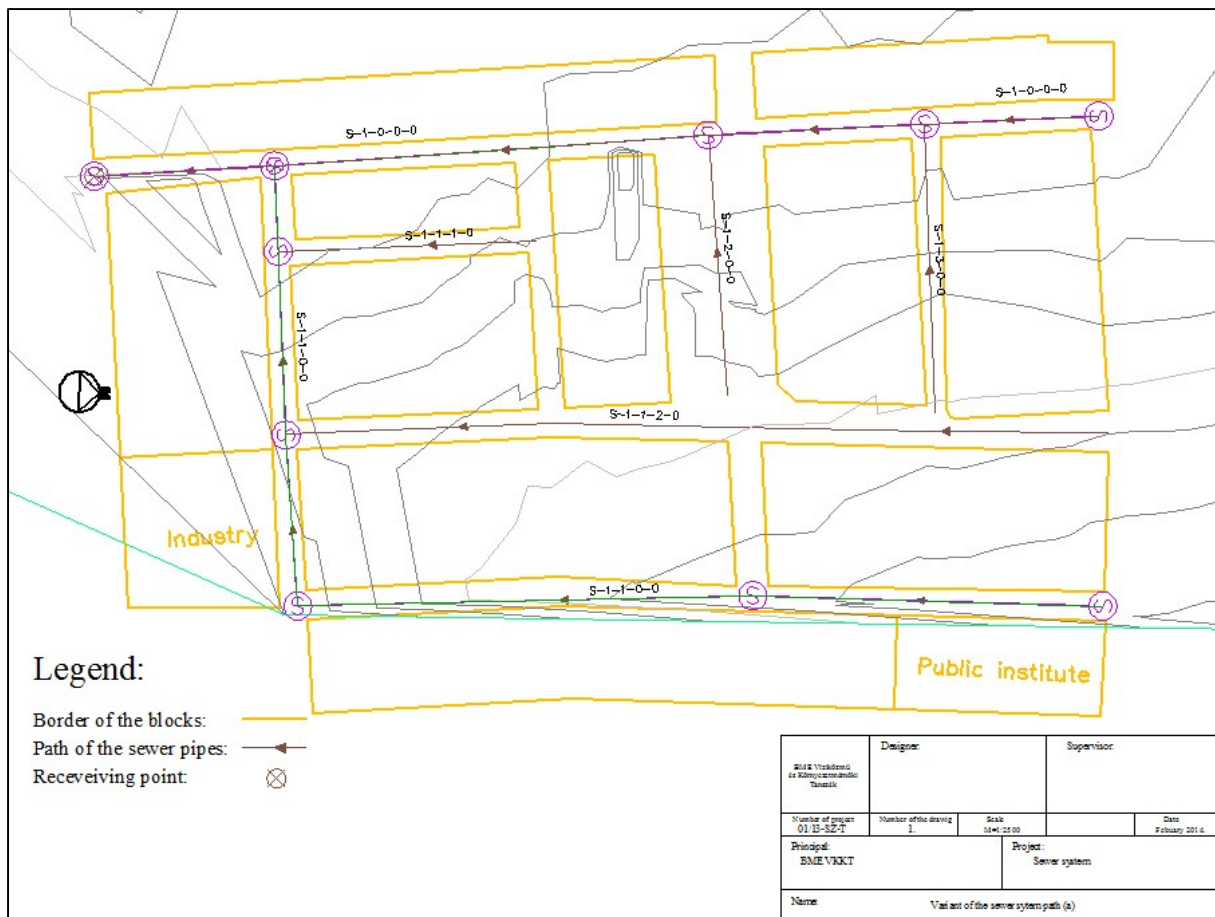
Figure 4. Path variant of the sewer sytem

The path of this pipe is in lowest level of the area. The collectors connect to the main pipe. The conduits must be laid down on the area of common use. We don't lay it on the private area, or below the buildings. The pipes must touch the consumption centers and the concentrated big consumer as the public institution and industrial area. It must connect to the given receiving point, which will be a output point in the hydraulic model.

If we drew the line of the pipes, we need to check the slope of the pipes because it is a gravity system. So we need to design some test longitudinal profile (Fig. 5). Then we assume a slope of the conduit. (we check the longitudinal section, is it proper for the surface?). The value of the slope must be between about 3 ‰ and 50 ‰. (D/100 ≤ I ≤ D/1000, D (mm))
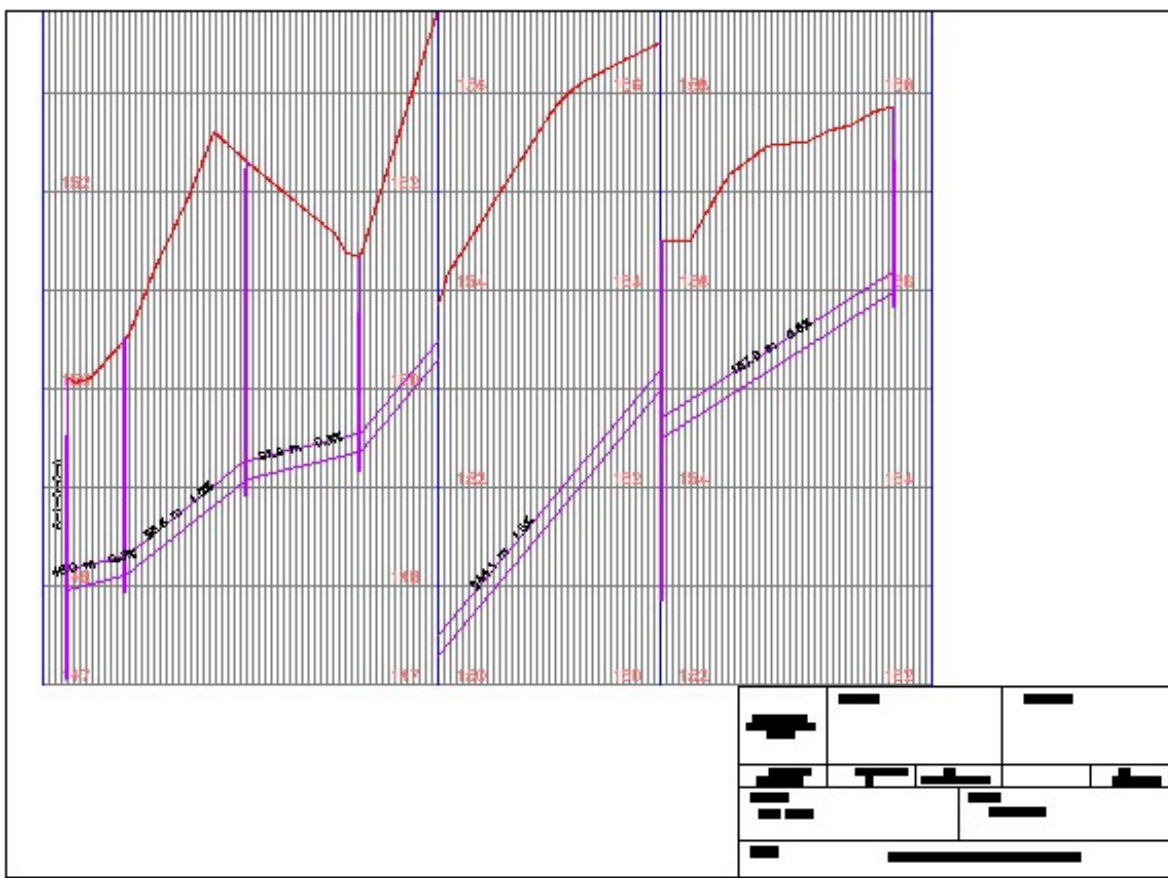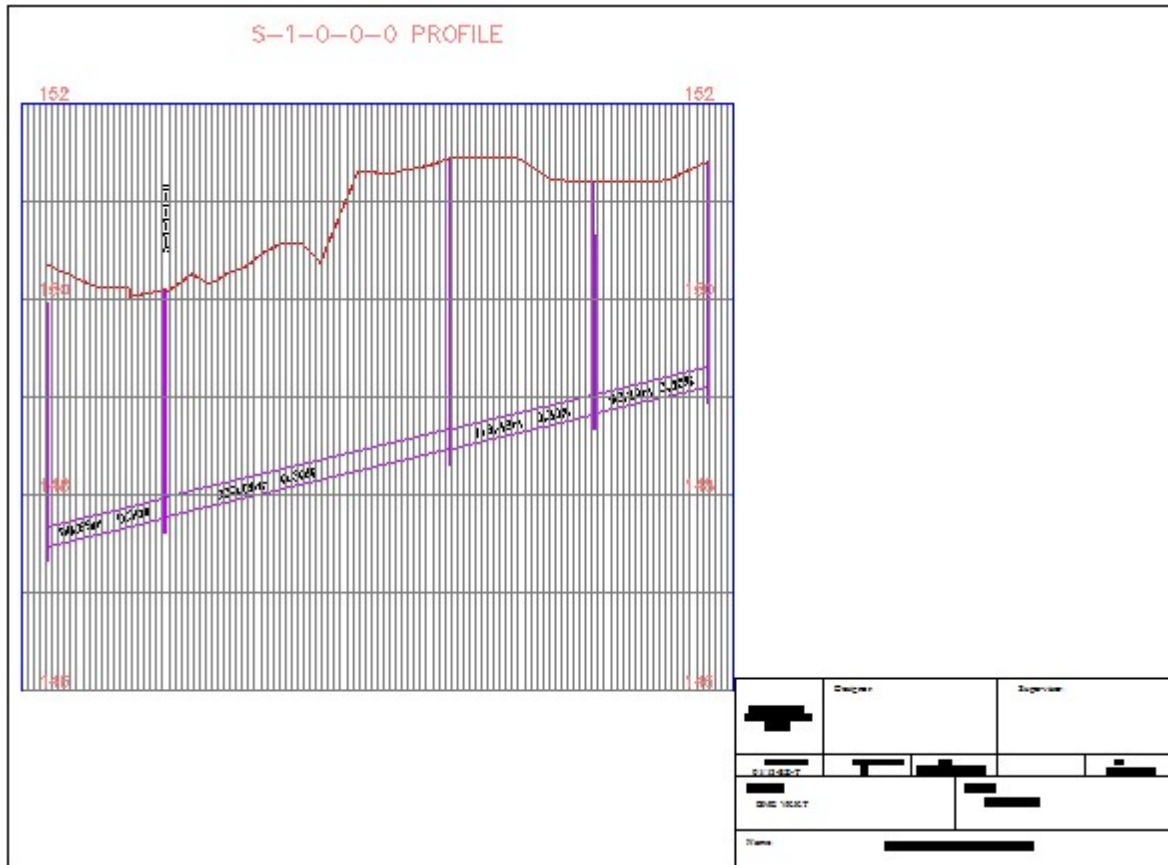
Figure 5. Longitudinal profile

If we have got the design waste water, and a slope we have to assume a pipe diameter. The pipe diameter must be larger than 200 mm. We give this minimum value by the public sewer, because this diameter can be cleaned with canal cleaner. If we have got a slope a diameter, we have to check is these values adequate for the waste water load. First we have to define the quantity of design waste water at the designing point. The designing point are at the:

- Receiving point
- Connection points
- After the big loads (public institution, industry)
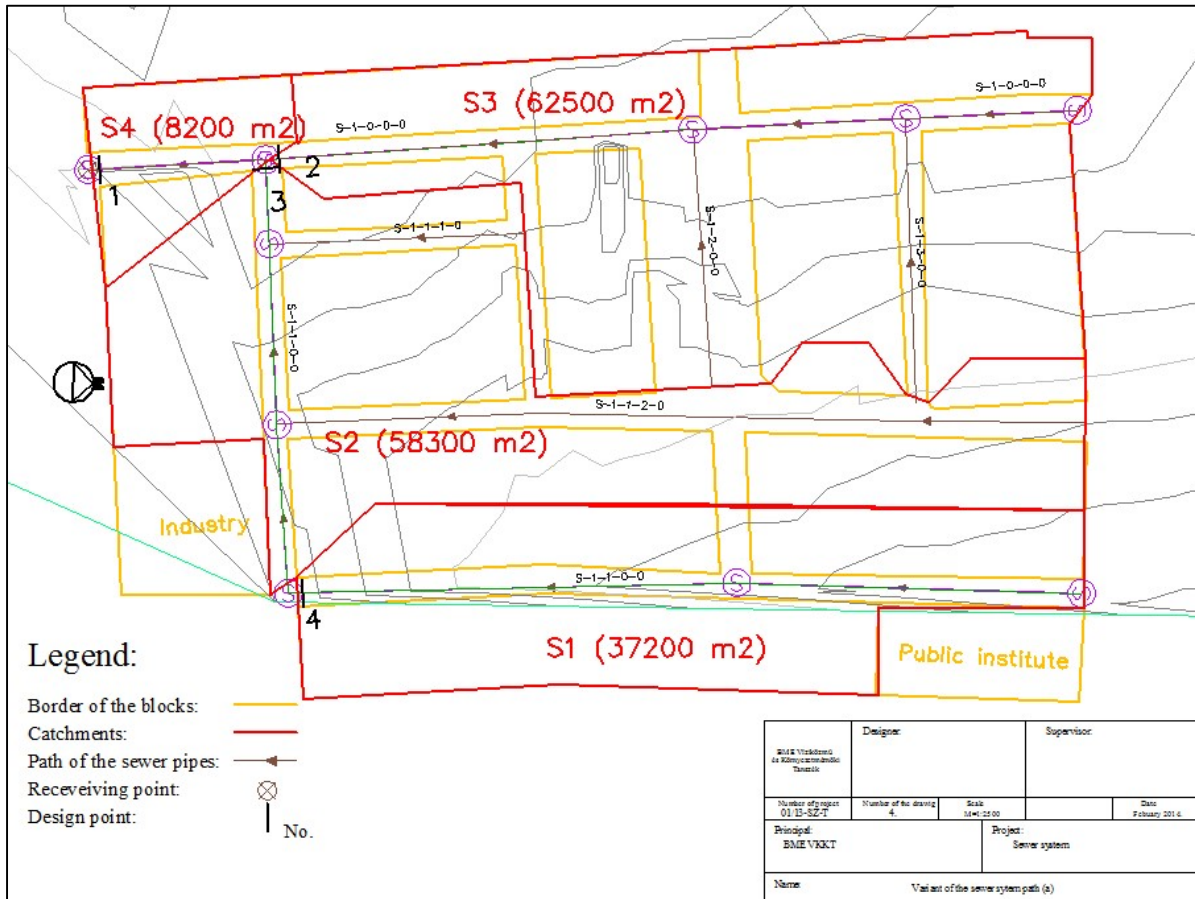
Figure 6. Catchments of the design points

The uniformly distributed loads as the dweller load is concentrated to the designing point according the catchment area. First we have to measure the catchment area of the designing points. We need 5-6 design point on our area.

Design point	Sign of the catchments	Catchment area (m2)	Wastewater load of the Inhabitants (m3/h)	wastewater of the public institution (m3/h)	wastewater of the industry (m3/h)	The design load (m3/h)
1	S1, S2, S3, S4	166 200	9.56	3.96	35.02	**48.54**
2	S3	62 500	3.60			**3.60**
3	S1, S2	95 500	5.49	3.96	35.02	**44.47**
4	S1	37 200	2.14		35.02	**37.16**
Sum:		**361 400**	**20.79**			

Table 17. Calculation of the wastewater load at the design points

We use for the checking the Prandtl-Kármán-Colebrook formula, what gives us the velocity of the waste-water in the full-section pipe. This formula can be applied to the gravity sewers designing. The load results from water velocity ($Q = v \cdot A$), A is the cross-sectional area of the pipe).

$$v_{tot} = \left[-2 \lg \left(\frac{2{,}51\upsilon}{d\sqrt{2gId}} + \frac{k}{3{,}71d} \right) \right] \bullet \sqrt{2gId} \qquad \qquad \text{1.1 formula}$$

$v -$ the water velocity (m/s)

$\upsilon -$ the kinematic viscosity of the waste-water $1{,}31*10^{-6}$ m/s^2

$g -$ acceleration of gravity (9,81 m/s^2)

$I -$ the slope of the conduit (m/m)

$d -$ diameter of the pipe (m)

$k -$ roughness of pipe wall (0,0025 m)

If we get the waste-water velocity, we have to check the velocity, and the minimal water depth in the sewer 3-4 cm, because the blockage or siltation. The real velocity of the waste-water must be 0,4-2,0 m/s. We can get the real water depth and the real velocity with next *graph 1.1.*.
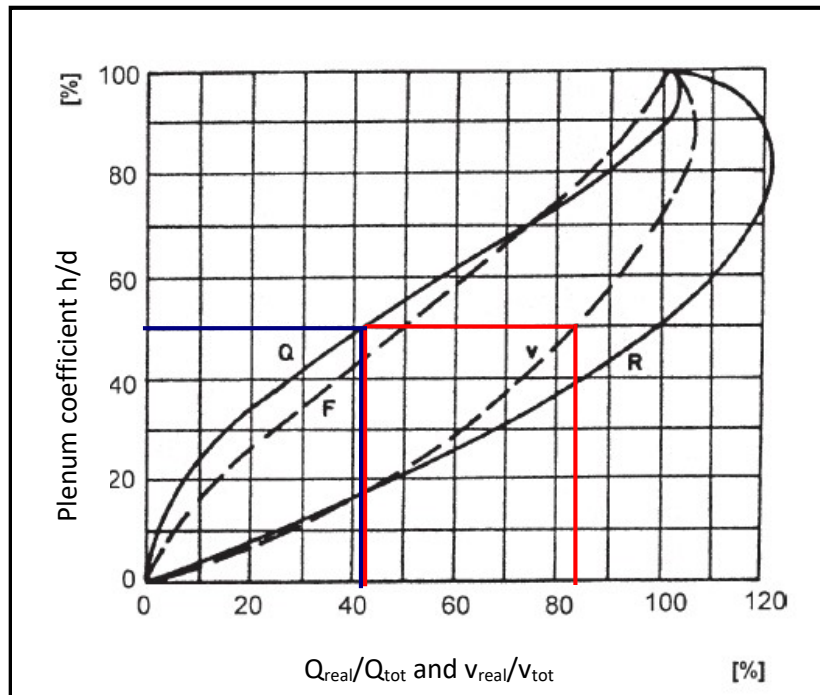
1.1. graph: *Plenum curve*

If we want get the real water velocity, and real water depth we have to calculate the total transported quantity, what the pipe can transport.

$$Q_{tot} = v_{tot} \cdot A \qquad\qquad\qquad \text{1.2. formula}$$

Q_{tot} − water flow with full section (m³/s)

v_{tot} − water velocity (m/s) derives from 1.1. formula

A − cross sectional area of pipe (m²)

Then we have to compute the next ratio:

$$\frac{Q_{real}}{Q_{tot}} \qquad\qquad\qquad \text{1.3. formula}$$

Q_{real} − the planning flow volume at the designing point (m³/s)

Q_{tot} – water flow with full section (m³/s)

If we have the ratio of Q_{real}/Q_{tot} we have to search this value on the horizontal shaft of the plenum curve. We have to project this value to the curve „Q", then we have to project the intersection we got, to the curve „v" horizontal. Then we get another section, and we have to project this section to the horizontal shaft. The value we got on the horizontal shaft is the v_{real}/v_{tot}. This method can be seen on the **graph 1.1.** *(red line)*. We can calculate the real velocity from this ratio and from v_{tot} (velocity of the waste-water in the full-section pipe).

$$v_{real} = v_{tot} \cdot \frac{v_{real}}{v_{tot}}$$

1.4. formula

Finally we have to calculate the water depth, the method can be seen on the **graph 1.1.** (blue line) We have to read the value h/d from the vertical shaft. We get the real water depth as it follows:

$$h = d\frac{h}{d}$$

d – pipe diameter

Finally we check the real water velocity, the real water depth, is it adequate for the rules, if the values aren't adequate we have to modify the slope or the pipe diameter until the result is satisfying.

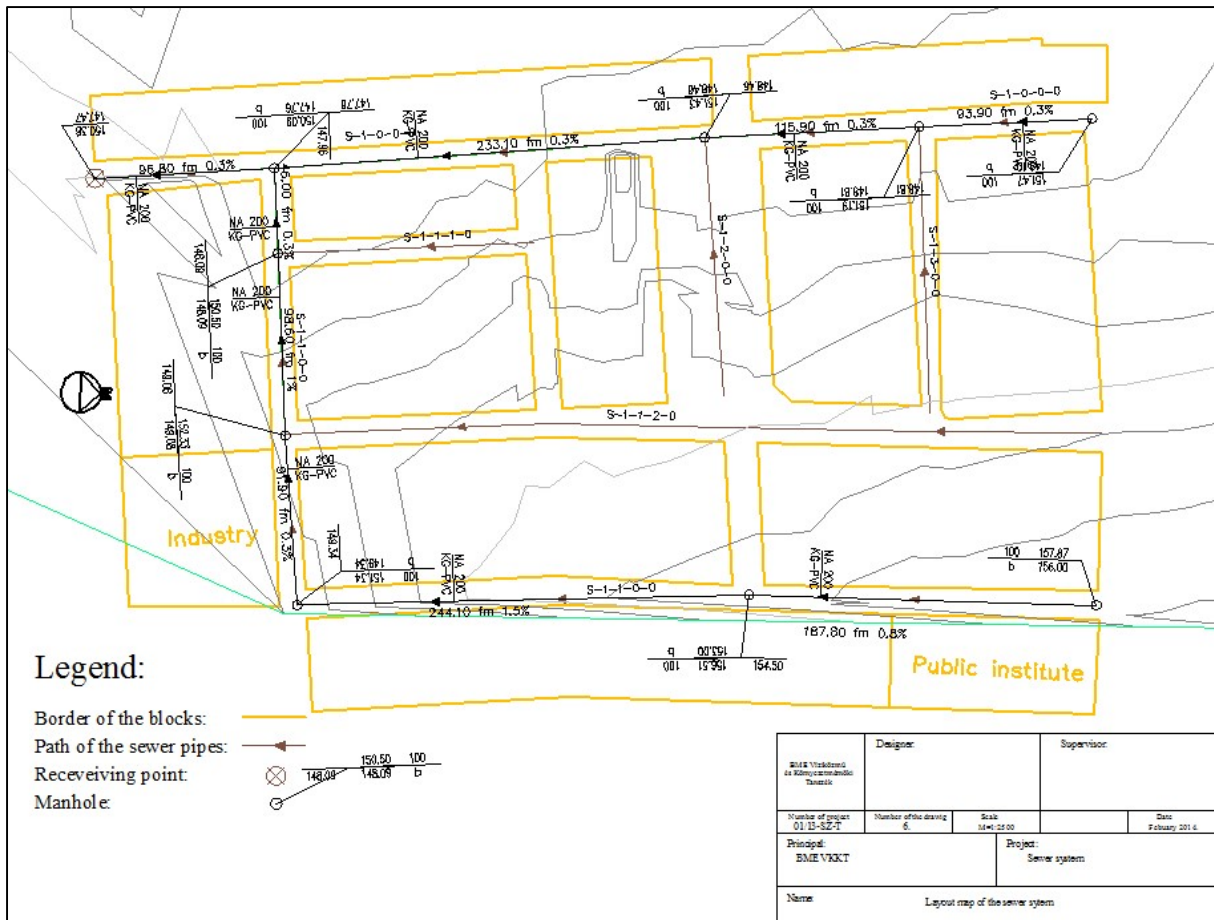| Design point	Slope (‰)	Diameter (mm)	Roughness of the pipe (mm)	vtot (m/s)	Qtot (l/s)	Q design (l/s)	Qdesign/ Qtot	v/vtot	vreal (m/s)	h/d	h (cm)
1	3	200	0.4	0.69	21.55	13.48	0.63	1.15	0.72	0.65	13.00
2	3	200	0.4	0.69	21.55	1.00	0.05	0.42	0.02	0.18	3.60
3	3	200	0.4	0.69	21.55	12.35	0.57	1.07	0.61	0.63	12.60
4	15	200	0.4	1.56	49.04	10.32	0.21	0.84	0.18	0.33	6.60

Table 17. Calculation of the pipe diameter

Figure 7. Final layout map of the sewer systems

Dimension of storm water pipe

The dimension of the storm water pipe is similar to the waste-water pipe. First we have to define the load of the storm water. We have to define the catchment, what belongs to the designing point. First we have to border the catchment. We mark the area, wherefrom the storm water flows to the designing point. Then we draw our sewer on the area. We can lay our sewer just on area of common use. We have to measure the sewers length. We use the rational method to the design. The theory of rational method is the next. The time of concentration is equal to the period of the design storm. The time of concentration (T) has got two parts (*figure 2.1*.):

  − the runoff on surface T_1
  − the runoff in the sewer T_2
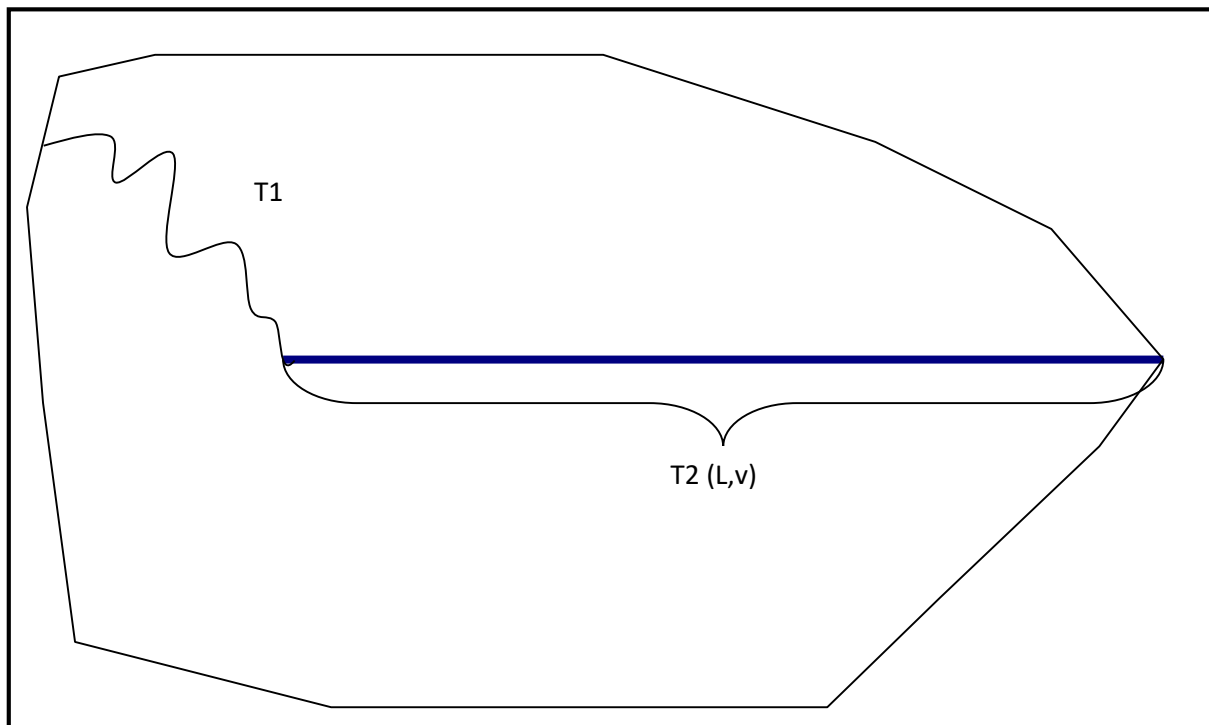
2.1. figure

T= T_1+T_2 (min)

We calculate the time of concentration from the farthest point. The run off on surface is given on the worksheet. This value depends on the slop of the surface the length on the surface the quality of surface. We have to calculate the runoff in the sewer if we assume the water velocity in the pipe 1 m/s (v) and we know the length of sewer (L).

$$T_2 = \frac{L}{v}$$

The next step we calculate precipitation intensity whit this formula:

$$i_p = a \cdot (\frac{T}{10})^{-m} \text{ (l/s ha)}$$

Where:

$T -$ the time of concentration (min)

$a, m -$ this value belong to rate of occurrence (see **next table 2.1.**)

Rate of occurrence	A	M
1 year	133	0,69
2 years	203	0,71
4 years	270	0,72

2.1. table

We calculate the load of storm water at the design point with next formula:

$$Q = i_p \cdot A \cdot \alpha \text{ (l/s)}$$

Where:

$i_p -$ precipitation intensity

$A -$ area of catchment

$\alpha -$ run-off coefficient

The run-off coefficient depends on the surface for example run-off coefficient of the covered surface is higher value than uncovered. This value can be seen on the worksheet.

If we have the load of the sewer at the design point. We can calculate the diameter of the pipe with Prandtl-Kármán-Colbrook formula. Then we have to check the real water velocity, and water depth. See by the waste water design method. If the difference is between the calculated real water velocity and assumed water velocity more than 20 per cent we have to change the slope or diameter of pipe until the difference lower than 20 per cent. We have got other possibility to correct the failure, for example we can change the assume water velocity.