# DESIGN OF A DRAINAGE SYSTEM TO IMPROVE THE WATER REGIMEN IN LANZHOT (SOUTH MORAVIA)


ČESKÁ ZEMĚDĚLSKÁ UNIVERZITA V PRAZE

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ESCUELA POLITÉCNICA SUPERIOR

UNIVERSIDAD DE ZARAGOZA

Universidad Zaragoza 1542

**STUDENT:** JUAN SURRA

**TUTOR:** DOC. ING. JAKUB ŠTIBINGER, CSC.

**TUTOR of UNIZAR:** CÉSAR GONZÁLEZ CEBOLLADA


JUNE 2016

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REPORT

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# 1. INTRODUCTION

On the border between Czech Republic and Slovakia, alongside to the Morava river is located Lanžhot (Region of South Moravia), that belongs to the municipality of Břeclav, Czech Republic.

Lanžhot is well known due to the meadows surrounding the region, therefore exists many problems with the area, that it´s annually flooded. The soil surface in the zone is low permeable and doesn´t allow the water to percolate the surface. For that reason is necessary a solution to improve the water regimen in the specified area.

The aim of this work is to design a drainage system (ditches, pipe drains, mobile gates, layouts etc.) to manage the water regimen and use the amount of water for our welfare, constructing special reservoirs and using them for agricultural purposes.

The role is to correct the soil surface´s shape and to construct the correct drainage system to avoid the surface runoff, to remove the water and conduct it into reservoirs, which are especially important in dry seasons. Besides, it´s necessary to arrange the problems with the subsurface water flow and also snow melting complications in winter seasons, when the water level increases and therefore waterlogging appears on the surface of the soil.

So the objective is design the properly drainage system and to carry the water into the reservoirs.

# 2. LAND DESCRIPTION

## 2.1 SITUATION

The land field is situated at Lanžhot, southwest of Slovakia, on the border between Czech Republic and Slovakia, alongside to the Morava River. It belongs to the region of South Moravia, district of Břeclav.

The town is located near to important cities like Brno (80 km) to the north, Bratislava (70 km) or Vienna (95km) to the south as we can see in the figure 1.1.


Fig 1.1 Lanžhot localization

In the Plane 1 we can see the localization of Lanžhot with more details.

The summary of the site are the next:

- City: Lanžhot.
- Country: Czech Republic.
- District: Břeclav.
- Population: 37030 habitant (2006 est.).
- Altitude over sea level: 164 m.
- Latitude: 48°43′N.
- Longitude: 16°58´E.
- Nearby Cities: Brno (65 km), Bratislava (70 km), Vienna (95 km).

Plane 2 shows the location of the study field, located few kilometres to the west of the town.

Due to the software viewers it´s possible to measure one essential parameter for the study, the area of the field.

Figure 1.2 shows the measures of the study field (Natura 2000 viewer).

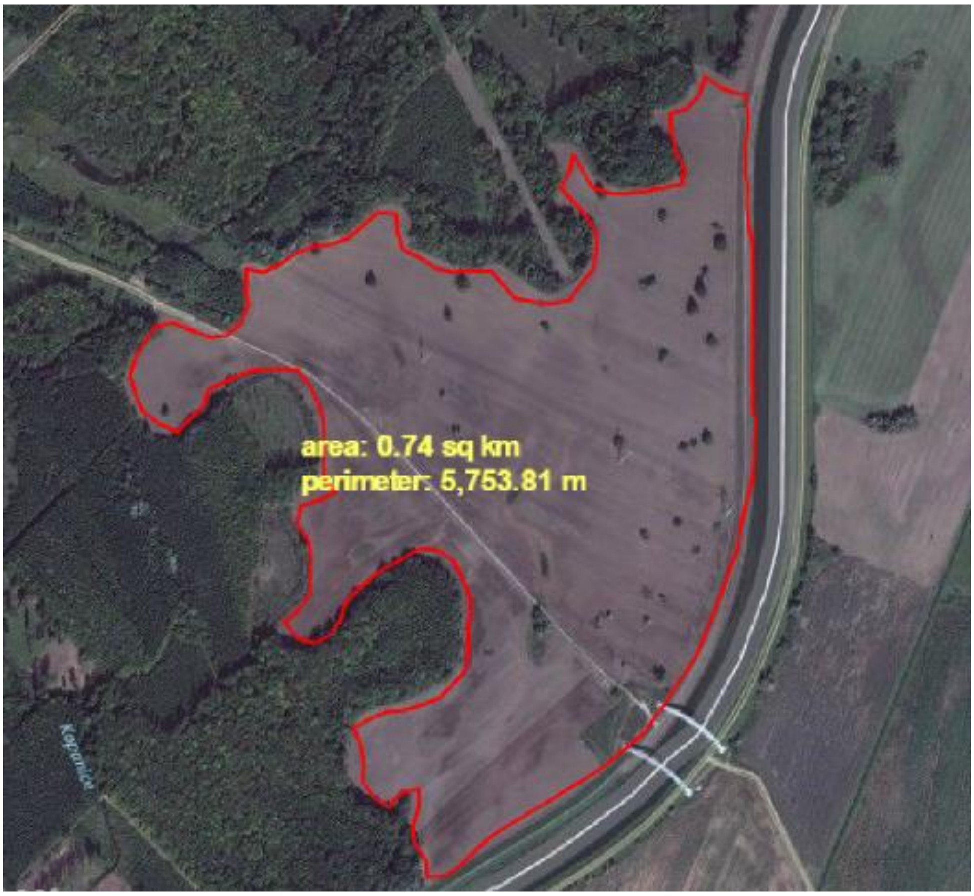


Fig 1.2 Area of the study field

Accordingly to the software, the area of the field is above 75 km$^2$:

$$A = 0.75\ km^2 = 750000\ m^2$$

## 2.2 GEOMORPHOLOGIC DESCRIPTION ON THE AREA

The main geomorphologic characteristics of Lanžhot are shown in figure 1.3, more detail in plane 3.

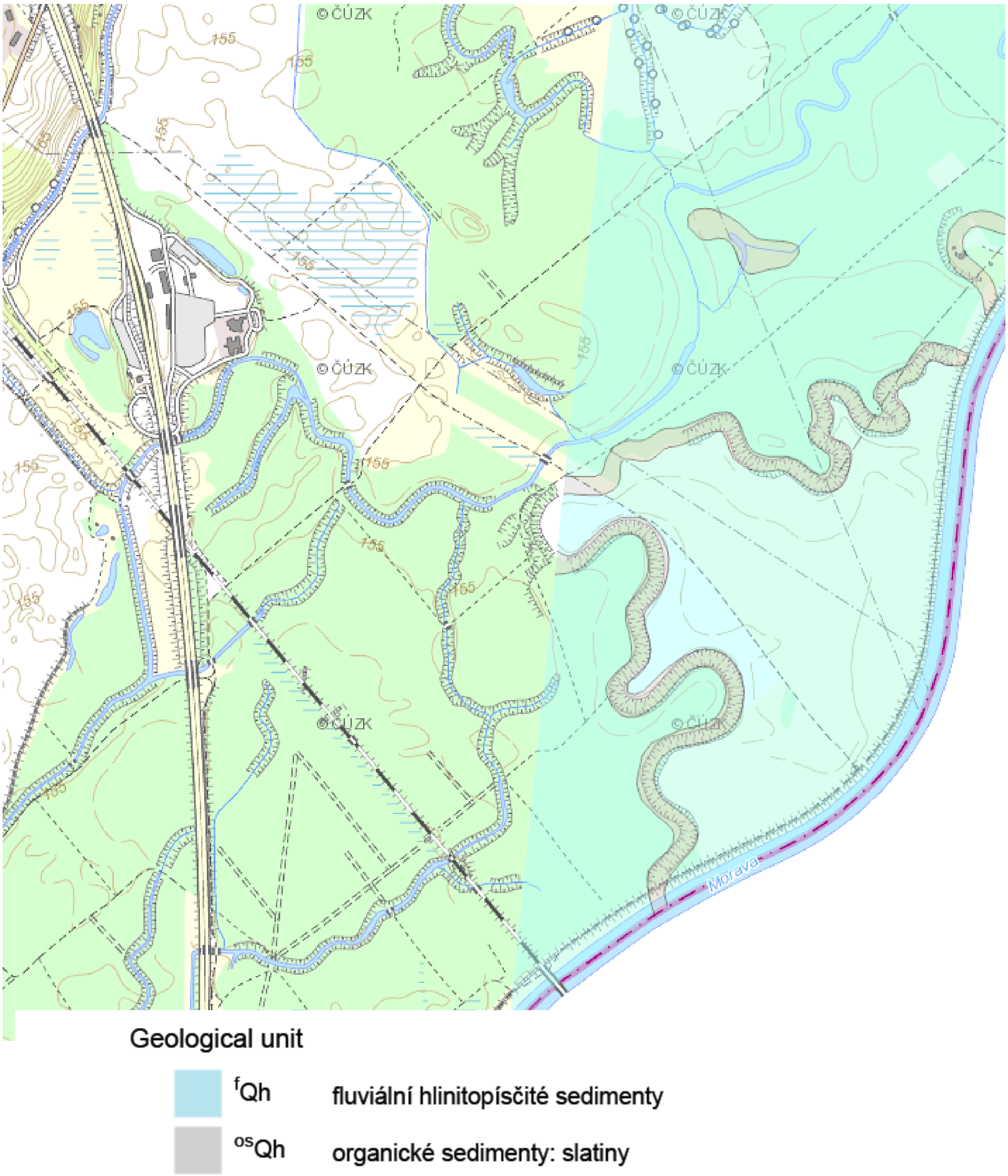


Fig 1.3 Geological map of Lanžhot region

As we can see in the plane 3, the characteristics of the surface soil can be described as loamy sediments. These kinds of soils are most susceptible to low infiltration and ponding due to their structures and characteristics.

Therefore, even with low-intensity rainfall, the infiltration in the soil could be lower than the intensity of the precipitation and the water is accumulated into ponds form on the ground surface.

## 2.3 CLIMATIC DESCRIPTION OF THE AREA

The weather in Lanžhot quite variable along the year. The precipitation in Lanžhot remains during the whole year, even in summer.  The closer meteorology station to Lanžhot is in Breslav and the can resume the characteristics on the figure 1.4 (World Weather Online).

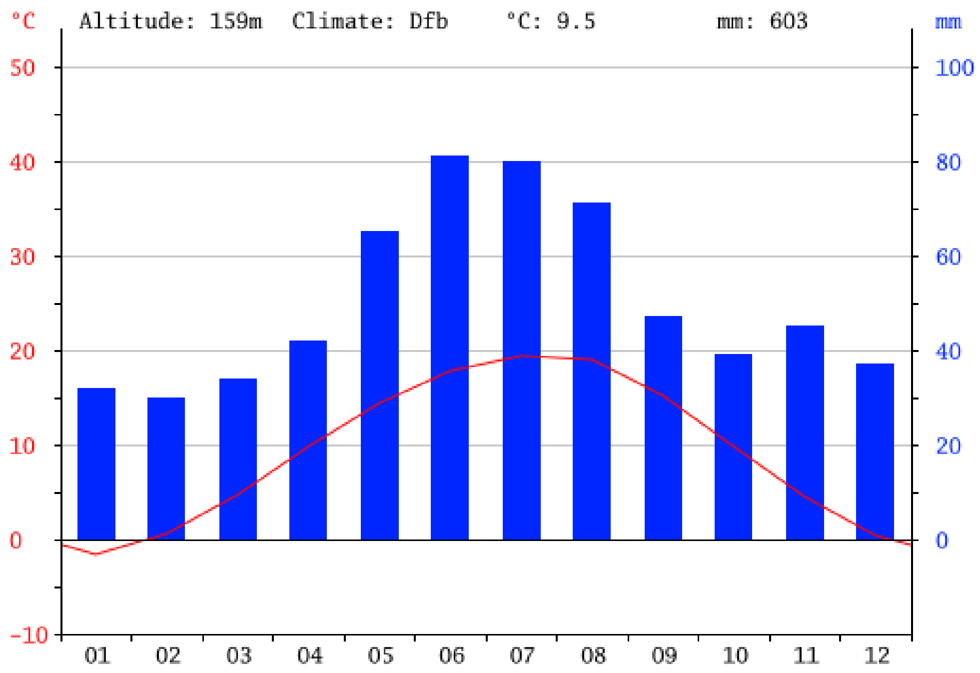


Fig 1.4 Average temperatures and precipitation in Lanžhot

The average temperature in Břeclav is 9.5 ºC. The hottest month is July with an average of 19.4 ºC. On the other hand, the coldest month is January with -1.6.

The long term annual precipitation is above 600 mm/year. February represents the drier month with an average of 25 mm and in June the average is over 80 mm.

For this reason, it´ll be evaluated the system drainage considering  precipitation conditions among 0.5 – 4 mm/day.

## 2.4 PHOTOGRAPHIC DESCRIPTION OF THE FIELD

To have an idea about the problems, the following pictures shows the difficulties on the field, related to the low permeability of the soil.

# 3. LAND FORMATION

## 3.1 INTRODUCTION

On flat agricultural lands, where the infiltration into the soils is less than the amount of water accumulated is common the pond forms. Besides fine-textured soils and soils that dorm crusts easily are most susceptible to low infiltration and ponding. The cause is usually near to the ground surface, in the form of natural pans or human-induced compacted layers such as plough soles.

Rainfrall intensity and infiltration are both rate funtions of time, and their combination leads to a  critical period when conditions are worst, therefore the surface drainage is requiered.

Surface drainage is applied on flat lands where slow infiltration, low permeabiliy or restricting layers in the profile avoid to absorbe the high intensity rainfall. According to this, the objetive of the drainage system is to eliminate ponding by accelerating the flow to an outlet system.

The negative effects of ponds and poor drainage on the agricultural can be summarised as (Sevenhuijsen, 2006):

- Inundation of crops, resulting in deficient crops growth.
- Lack of oxygen in the rootzone,hamperring germination and the uptake of nutrients.
- Insufficient accessibility of the land for mechanized farming operations.
- Low soil temperatures in spring time

As surface drainage aim is to remove the exccess of water form the land surface,it has an effect on the enviroment of the area due to its nature. The drains are often placed in a pattern determined by the characteristics of the area.

There are many drainage systems  methods determinated by the characteristics of the area. The desing of a surface drainage system has two components:

- Shaping of the surface by land forming (Paragraph 3.2).
- Constrution of open drains to the main outlet (Paragraph 3.3).

## 3.2 SYSTEM-BEDDING

In order to apply the drainage effectively, the land surfafe should be reasonably smooth by eliminating minor differences in elevation. It should preferably have some slope toward collection points from where the water is discharged through specials outlets. Land smoothing is the cheapest surface drainage practise and it can be performed on an annual basis after completion of tillage operations.

The bedding system is one of the oldest surface drainage systems. The soil is formed into beds and seperated by parallel furrows oriented in the direction of the greatest land slope.

The water drains from the beds into the dead furrows, which discharge into a field drain constructed at the lower end of the field and perpendicular to the dead furrows (figure 3.1).

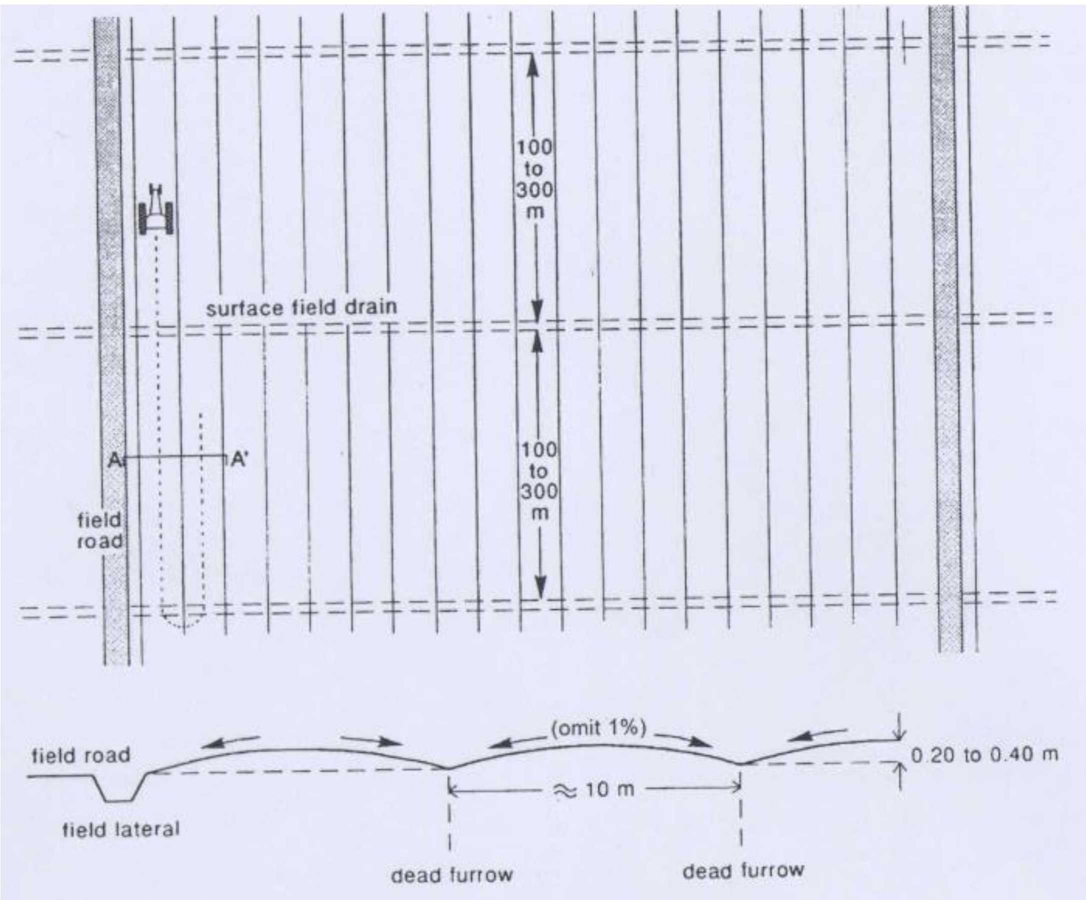


**Figure 3.1. Bedding system**

Because of the construction and land preparation, the top soil of the bed has better hydraulic propierties that the subjacent soil, which is much more impermeable.

In modern farming is not considered an acceptable drainage for row crops because it doesn´t allow drainage satisfactorily (rows are to adjacent to the dead furrows). It is acceptable for grasslands, althought there will be some crop loss due to the adjacent to the dead furrows.

## 3.3 DESIGN AND CONSTRUCTION

During the first ploughing, the beds has to had uniform width throughout the field and the direction of the dead furrows must be on the greatest slope.

The obstructions or low points in the dead furrows should be eliminated because they will cause standing water and loss of crops.

The bedding system principle is ilustrated in Figure 3.2.

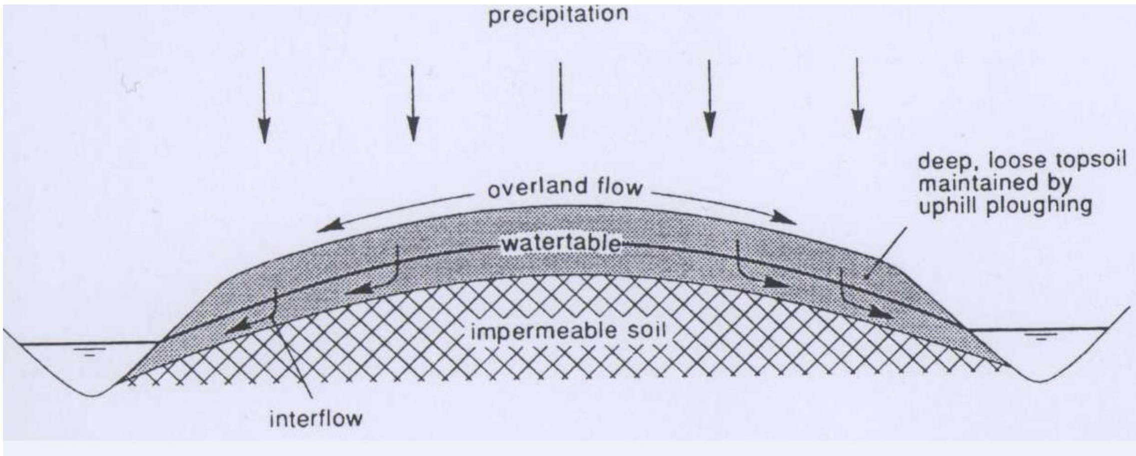


**Figure 3.2. Bedding system principle**

Although it is a drainage system applied to grow vegetables, tree crops, stable crops like maize or cassava it presents some disadventages like (FAO, 2008):

- Reduction of the yield at the sides due to the movement of the soil from the bed to the middle.
- Restriction for mechanised farmings.
- Ponded areas if the slope of the dead furrows if not sufficient.
- Requiered regular maintenance to prevent weed growth.

To sum up, our objetive is to apply the bedding system besides open ditches to increase the efficiency of the system, as it shows in figure 3.3.

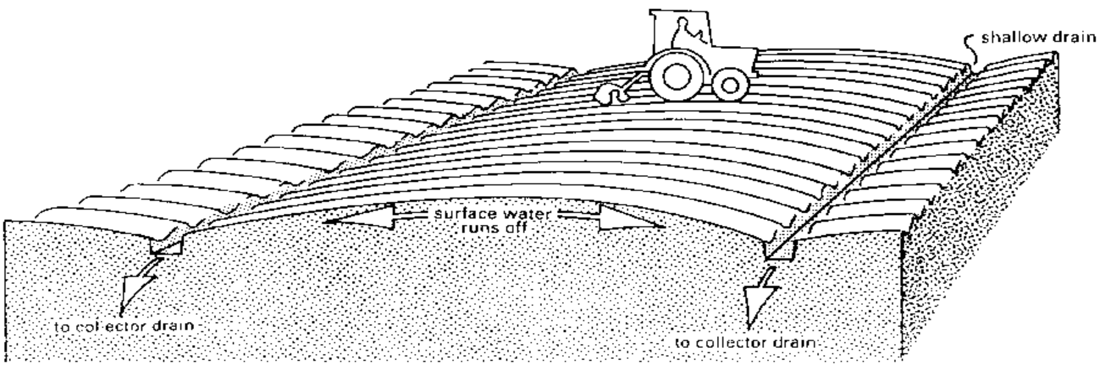


**Figure 3.3. Bedding system with open ditches**

## 3.3 POND

Ponds are very important infrastructures in irrigation systems, due to their function of regulation and storage of water. The aim is to storage the water in the seasons when it´s not needed to use it when the crops required it in dry sessions.  Therefore, is used to equilibrate the supply and demand of water along the year either irrigation or supply.

For the irrigation, the water may be released into networks of canals for use in farmlands or secondary water systems. Irrigation may also be supported by reservoirs which maintain river flows, allowing water to be abstracted for irrigation lower down the river.

Besides, it serves as a flood control system, collecting the water at times of very high rainfall, and releases it slowly during the following weeks or months.

For the present project, a special pond has been designed according to our conditions. It is shown in the figure 3.4 and also in Annex 3 with more details.

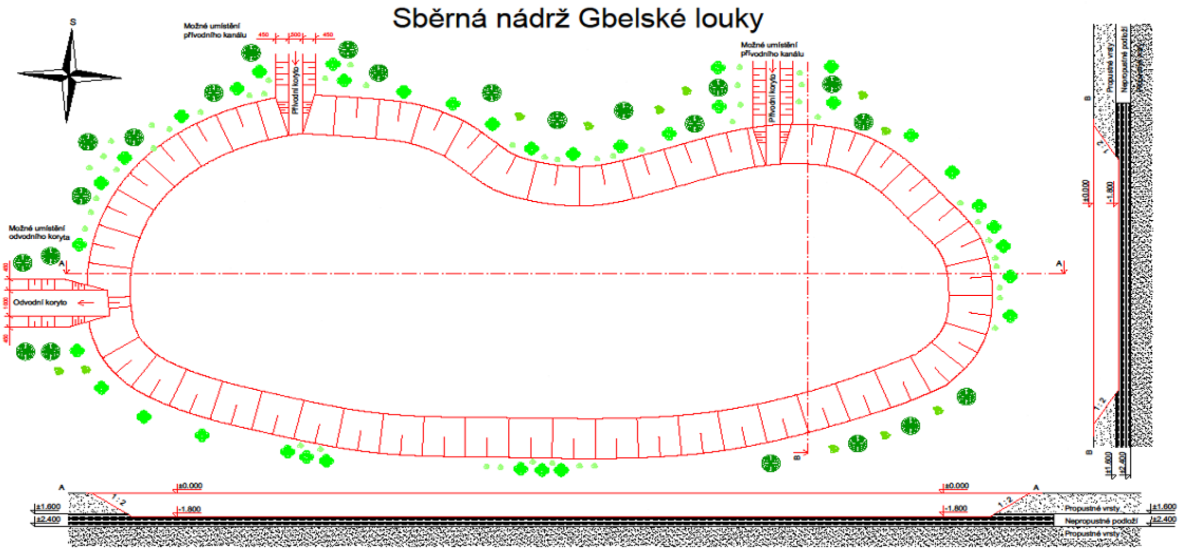


Figure 3.4. Designed pond for the project

## 3.4 ALTERNATIVES

Two alternatives of open ditches network are proposed in order to evaluate appropriately the drainage system. Both of them are designed consequently according to the conditions of the project.

The possibilities are shown in annex 1 and 2. Therefore, for subsequent calculations, the main parameters are shown in the figure 3.5

**ALTERNATIVE 1** **ALTERNATIVE 2**

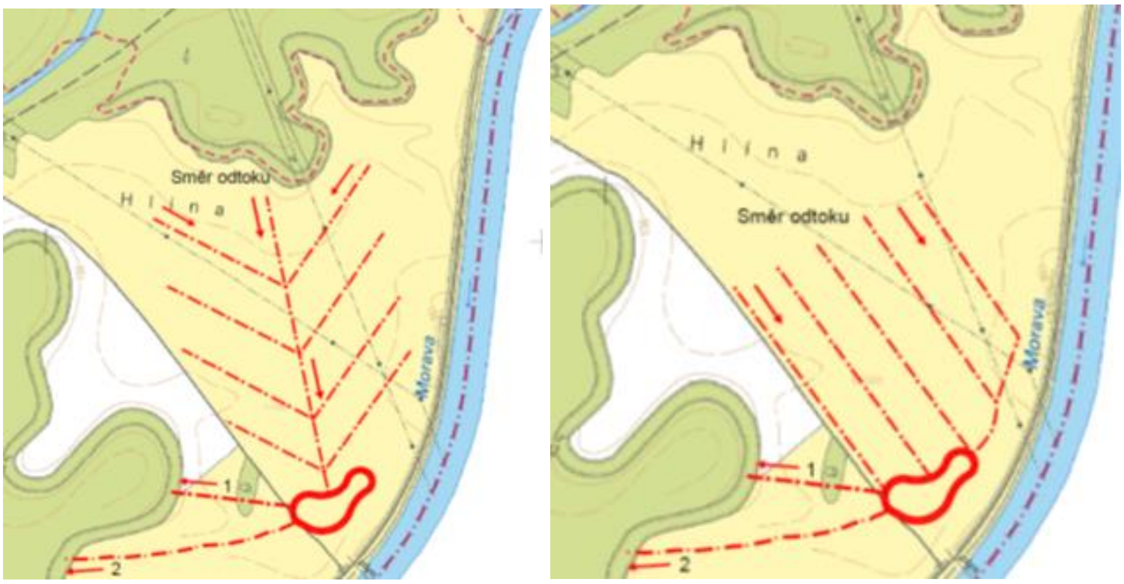


Figure 3.5. Ditches network alternatives

**ALTERNATIVE 1 (Herringbone Netwok)**

$$L_{DITCHES} = 300\ m$$

For the runoff calculation, it´s important to know the longitude of the main ditch, as:

$$L_{MAIN\ DITCH} = 630\ m$$

$$A_{DRAINAGE} = 184900\ m^2$$

**ALTERNATIVE 2 (Parallel Network)**

$$L_{DITCHES} = 350\ m$$

$$L_{MAIN\ DITCH} = 150\ m$$

$$A_{DRAINAGE} = 122500\ m^2$$

# 4. DRAINAGE AND IRRIGATION SYSTEM

The objective of surface drainage is to improve crop growth conditions by providing timely removal of excess water near the ground surface before the crops is damaged.

Drainage is also needed to guarantee soil workability and trafficability, preventing delays in soil preparation operations and harvesting, respectively.

Though complexing nature, two types of problems are usually distinguished, though they may occur in combination:

- Surface drainage problems, where there is an excess of water on the land surface. Surface drainage then aims at an orderly removal of excess water from the surface of the land.

- Subsurface drainage problems, where there is an excess of water within the soil at close proximity to the land surface. Subsurface drainage then aims at removing the excess water from the soil and maintaining the watertable at an adequate level.

## 4.1 PLANNING CONSIDERATIONS

Soils, climate, topography, and other conditions vary from place to place and consequently each condition could change the design of the drainage system.

**Climate**

Climate has the major impact on the environment and is often responsible for variations in soils, water, and the appearance of plants. It is a decisive factor in determining the type of drainage system to be applied.

In humid climates, drainage is largely required to evacuate excess rainfall, however, in arid and semi-arid climates; drainage is needed mainly to remove excess irrigation water.

The main climatic data needed are the average monthly temperature and the average annual precipitation.

An assessment of the magnitude of the land-drainage problem also requires information on rainfall intensity. Data on 24-hour rainfall should be examined, and return periods of high-intensity rainfall should be determined.

For the project, it will be studied the relation between the distance of the ditches and the discharge. The discharge will variable between 0.5-4 mm/day, according to the climate conditions mentioned.

Finally, supposing the worst situation, in order to evaluate a high intensity precipitation, it will be assumed as 300 mm/day for the runoff possibility.

**Soils**

Since geological conditions can cause drainage problems, a geological map of the region may be most helpful in delineating problem areas. Therefore, certain knowledge of the geology of the project area and its surroundings it´s attached in plane 3.

The geological map should be supplemented by a number of cross sections showing the lithological sequence, the depth and thickness of water-transmitting layers, and the depth of the impervious layer. A classification of the soils according to a standard taxonomy may provide useful information on the extent of the drainage problem.

Nevertheless, for the present project it´ll be used basic parameters of the soil as:

- Hydraulic Conductivity (Annex 4).
- Manning Coefficient (Annex 5).
- Runoff Coefficient (Annex 6).
- Maximum and minimum recommended values for the water in the channel (Annex 7).

**Economics**

To keep the costs of excavation to a minimum, excess surface water from an agricultural area should be removed along the shortest possible routes (the direction and alignment of the main canals depend largely on the topography and the slope of the land).

Most wet soils can be drained successfully, but for some soils, the installation cost may be so great that the benefits derived do not justify the expense. The long-range economic benefits must exceed the cost of the system. To make this analysis, the cost of the drainage system and the potential crop yields must be estimated over time.

Consulting with a farm management or using appropriate interest tables would determinate if the investment has economic viability.

**System patterns**

In a singular drainage system, each drain discharges into an open collector drain. In a composite system, the field drains discharge into a collector, which in turn discharges into an open main drain. The collector system itself may be composite with sub-collectors and a main collector.

Four commonly used subsurface system patterns are illustrated in figure 4.1. The system designed will selected the best pattern that fits on the topography of the land and makes the most efficient use of the investment in the subsurface drainage system.

1. **Parallel (a).** Consists of parallel lateral drains located perpendicular to the main drain. The laterals in the pattern may be spaced at any interval consistent with site conditions. This pattern is used on flat, regularly shaped fields and on uniform soil. Variations of this pattern are often combined with others.

2. **Herringbone pattern (b).** Consists of parallel laterals that enter the main at an angle, usually from both sides. The main is located on the major slope of the land, and the laterals are angled upstream on a grade. This pattern is often combined with others to drain small or irregular areas. The herringbone pattern can provide the extra drainage needed for the less permeable soils that are found in narrow depressions.

3. **Double main (c).** It´s a modification of the parallel and herringbone patterns. It is applicable where a depression, frequently a watercourse, divides the field in which drains are to be installed. This pattern also is sometimes chosen where the depressional area is wet because of seepage coming from higher ground.

4. **Random (d).** The random pattern is suitable for undulating or rolling land that contains isolated wet areas. The laterals in this pattern are arranged according to the size of the isolated wet areas. Thus, the laterals may be arranged in a parallel or herringbone pattern or may be a single drain connected to a submain or the main drain.

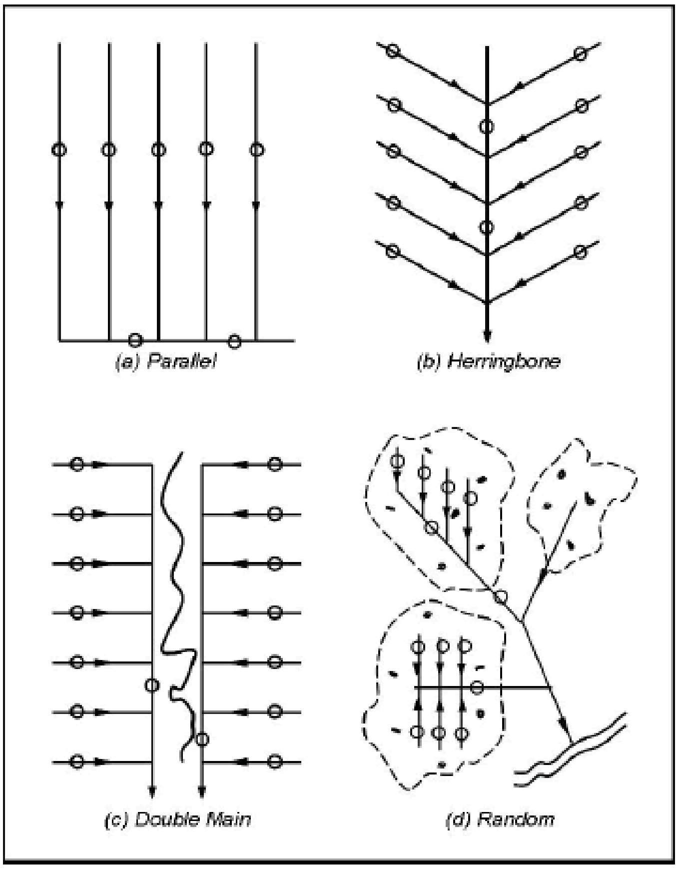


Fig 4.1. Basic patterns for surface drainage system

**Materials**

Drainage systems requires to collect all excess water from the problem area and evacuate this water beyond the project boundary without affecting neighboring agricultural land.

The results of common experiences in the region show that natural drainage ways can be used to remove the water instead of artificial drainage system.

When an open channel is used as an outlet, it must be large enough to remove the drainage runoff from the watershed quickly enough to prevent the crop damage. The open channel outlet must also be deep enough.

## 4.2 DRAINAGE SYSTEM

Drainage ditches are open trenches uses to improve drainage in relatively flat areas with wet or low permeability soils. Ditches are interesting due to the fact that you can see what is happening inside.

The ditches take out the excess of water from the root zone flows of a soil affected by high water table into the open drains, decreasing the water table level, as it shows in figure 4.2.

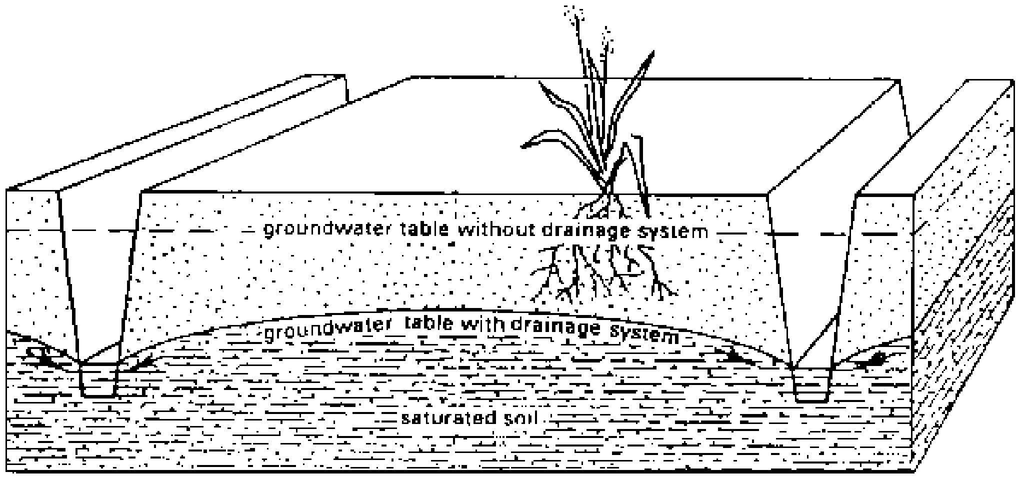


**Figure 4.2. Water table level with/without open ditch**

They don´t need special machinery and maintenance is not technical. But in addition ditches fill some space, need to be cleaned, a great terrain is necessary and also people can fall into them.

The most common advantages of the ditches can be summarized as:

- Improve the soil environment for vegetative growth by regulating the water table level and ground water flow.
- Intercept and prevent water movements into a wet area.
- Serving as an outlet for other subsurface drains.
- Regulate and control ground water for sub-irrigated areas or waste disposal areas.
- Remove surface runoff and ponded water around buildings, roads, airports, recreational fields and physical improvements.

## 4.3 IRRIGATION SYSTEM

Ditch systems are not only useful for being an appropriate drainage system, it´s also suitable for irrigation system. Although  the ditch irrigation is a traditional method, it´s effective for small and medium areas where ditches are dug out and seedlings are planted in rows. With this system, lawns and plants are maintained with the minimum amount of water required. The plantings are watered by placing canals or furrows in between the rows of plants.

Sustainable channel design can result in ditches that are largely self-maintaining due to natural geomorphological equilibrium. Slowed net siltation and erosion result in net reduction in sediment transport.

The main problem of this method is that it´s necessary to provide and maintain certain water flow on the furrows. For this purpose, it´s necessary to find one water source near to the field.

Fortunately, close to the field it´s located the water power station of Hódonin, which provides water with appropriately properties for the surrounding fields that needs water for their irrigation requirements.

This power station, located about 20 km from the field, provides with the corresponding pipelines system a water flow between 2.5 and 3 m/s.

$$Q_{water} = 2.5 - 3\ (m/s)$$

Consequently, it’s necessary to calculate if this source of water will be enough for the field, taking count all the possible loses in the field, mainly the evapotranspiration.

# 5. THEORETICAL

## 5.1 SATURATED HYDRAULIC CONDUCTIVITY

Hydraulic conductivity is one of the principal and most important soil hydrology and hydraulic parameter and it´s an important factor on the water transport in the soil and it´s used in all equations for groundwater and subsurface water flow.

The definition of the hydraulic conductivity follows from the Darcy's Law. In the saturated flow conditions the velocity of water flow $(v)$ in the soils or in the other porous media, is directly proportional to the hydraulic gradient $(I)$, as it can be generally expressed by the following equation:

$$v = K \cdot I$$

In the saturated flow conditions and according to the Darcy's Law, the flow velocity $(v)$ can be expressed:

$$v = K \frac{\partial h}{\partial x}$$

Where

$x$ is the distance in the direction of groundwater flow

$h$ is hydraulic head.

Hydraulic conductivity $(k)$ can characterize the hydraulic properties of soils, earths and also the hydraulic properties of the other porous materials and media, from the point of view of the velocity of water flow in their porous fully saturated flow conditions.

Soil hydraulic conductivity is defined as a constant of proportionality in Darcy's Law. If the hydraulic gradient, which is the difference of h over a small difference of x, converges to 1 the last equations can be rewritten as:

$$v = K$$

What means, that the hydraulic conductivity can be regarded as the groundwater velocity, when the hydraulic gradient equals unity (when the hydraulic gradient is 1).

Very often the hydraulic conductivity is less than 10$(m/day)$, so the velocity of groundwater flow is almost always less than 1$(m/day)$. Of course, the K-value of saturated soils introduces the average hydraulic conductivity, which is dependent mainly on the shape, size and distribution of the pores and also depends on viscosity and density of water and on the soil temperature.

In some structure-less soils (sandy soils) the K-value is the same in all directions, but usually the K-values varies with flow direction. Soil layers vertical permeability is very often different from horizontal permeability because of vertical differences in the structure, texture and porosity.

In practise, it´s common to use generalized table with the ranges of K-values for certain soil texture is presented in table 5.1 (Ritzema 2006).

| Texture | Hydraulic conductivity K ($m.s^{-1}$) |
|---|---|
| **Gravel** | $10^{-3}$ - $10^{-3}$ |
| **Medium sand** | $10^{-3}$ - $10^{-4}$ |
| **Sandy loam, fine sand** | $10^{-4}$ |
| **Loam, clay loam, clay (well structured)** | $10^{-5}$ - $10^{-6}$ |
| **Very fine sandy loam** | about $10^{-6}$ |
| **Clay loam, clay (poorly structured)** | $10^{-7}$ - $10^{-8}$ |
| **Dense clay (no cracks, pores)** | $10^{-8}$ - $10^{-10}$ |

Table 5.1 K-value range by soil texture (Ritzema 2006)

Table 5.1 should be used with care. Soils with identical texture may have quite different K-values, due to differences in structure and also some heavy clay soils have well-developed structures and much higher K-values than those indicated at the Table 5.1.

K-values in vertical direction is marked as $K_v$, hydraulic conductivity in horizontal direction represents symbol $K_h$ and K-value in intermediate direction is $K_r$. The value of $K_r$ can by approximated by formula:

$$K_v = \sqrt{K_r \cdot K_h}$$

or

$$\ln K_v = \frac{In\, K_v + InK_h}{2}$$

Hydraulic conductivity in the soil profile can be highly variable from place to place and also can be variable with a different depth, what means spatial variability. K-values can be variable not only in connection with different soil layers, but also within a one soil layer.

Hydraulic conductivity plays a key role in an environmental and water regime protection. In water engineering practice is necessary to know how to apply corresponding fundamental methods to determine K to describe, explain and analyse subsurface and surface water flow not only in landscape, but also in rural and urban areas.

There are many various methods, the laboratory methods and the field's methods, to determine or to approximate the value of hydraulic conductivity.

On the present project,it´s used data from a field experiment (annex 4) that shows that the hydraulic conductivity of the soil could be assumed as:

$$K = 0.24\, {}^{m}/_{day} = 2.77 \cdot 10^{-6}\, m/s$$

As we can see, the field experiments shows results according with the values given by the table 5.1 according  to the type of soil shows in the geomorphological description (clammy soils).

## 5.2 CHEZY EQUATION

Chezy and Manning equations are the foundation of our present science of open channel hydraulics.

**Law of Continuity**

The Law of Continuity, published in 1628 by Benedetto Castelli, who was a pupil of Galileo Galilei, shows the fluid law in open channels:

$$Q = A \cdot V$$

where:

- $Q$ is volumetric flow rate in $(m^3/s)$;

- $A$ is flow cross-sectional area in $(m^2)$

- $V$ is average velocity over a cross-sectional area perpendicular to the flow in $(m/s)$

**Chezy Equation**

In the 1760s, French engineer Antoine Chezy compared flow behavior between two streams having similar characteristics and formulated his observations as:

$$V = C\sqrt{RI}$$

here:

- C is Chezy's coefficient$(-)$;

- $R$ is hydraulic radius $(m)$ as can be defined as:

$$Hidraulic\ Radius\ (R) = \frac{Flow\ area}{Wetted\ perimeter}$$

- $I$ is the slope of the channel bottom, in the project:

$$I = 1\ \%$$

Very many studies have been made of the evaluation of C for different natural and manmade channels. These have resulted in today most practicing engineers use some form of this relationship to give C as:

$$C = \frac{R^{\frac{1}{6}}}{n}$$

This is known as Manning´s formula, and the $n$ as Manning´s $n$.

It has always been known that coefficients, C and n, are numerically variable. They appear to vary with roughness of the channel boundaries and for very shallow and very steep slopes. However, in practice, is common to use average values as it shown in the Annex 5.

## 5.3. STEADY-STATE EQUATION (HOOGHOUDT EQUATION).

Steady-State drainage equations are bases on the assumption that the rate of the recharge to the groundwater is uniform and steady. Thus, the water table remains in the same position as long as the recharge continues.

This is the typical situation on areas with prolonged periods of fairly uniform and medium-intensity rainfall. To describe the flow of groundwater to the drains, we have to make the following assumptions:

- Two-dimensional flow. This means that the flow is considered to be identical in any cross section perpendicular to the drains; this is only true for infinitely long drains.
- Uniform distribution of the recharge.
- Homogeneous and isotropic soils. We thus ignore any spatial variation of the hydraulic conductivity within a soil layer, although we can treat soil profiles consisting of two or more layers.

Consider a steady-state flow to vertically-walled open drains reaching an impervious layer (Figure 5.2). Darcy's Equation can be applied to describe the flow of groundwater ($q_x$) through a vertical plane ($y$) at a distance ($x$) from the ditch.

$$q_x = K\,y\frac{dy}{dx}$$

Where

$q_x$ = unit flow rate in the x-direction ($m^2/d$)

$K$ = hydraulic conductivity of the soil ($m/d$)

$y$ = height of the watertable at $x$ $(m)$

$-dx$ = hydraulic gradient at $x$ (-)

The continuity principle requires that all the water entering the soil in the surface area midway between the drains and the vertical plane (y) at distance (x) must pass through this plane on its way to the drain (Figure 5.2)

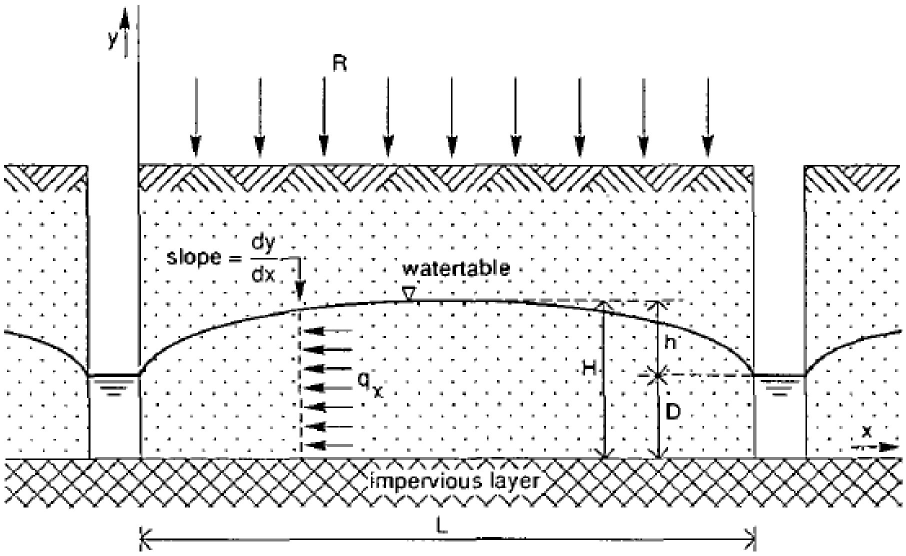


Fig. 5.2 Flow to vertically-walled drains reaching the impervious layer

If $R$ is the rate of recharge per unit area, then the flow per unit time through the plane $(y)$ is:

$$q_x = R\left(\frac{1}{2}L - x\right)$$

where

$R$ = rate of recharge per unit surface area $(m/d)$

$L$ = drain spacing $(m)$

Since the flow in the two cases must be equal, we can equal e the right side of the last equations

$$Ky\frac{dy}{dx} = R\left(\frac{1}{2}L - x\right)$$

This can also be written as

$$Kydy = R\left(\frac{1}{2}L - x\right)dx$$

The limits of integration of this differential equation are

for $x = 0 \ \ y = D$

and

for $x = {}^{1}/_{2}L\ ;\ y = H$

where

$D$ = elevation of the water level in the drain $(m)$

$H$ = elevation of the watertable midway between the drains $(m)$

Integrating the differential equation and substituting the limits yields

$$L^2 = \frac{4K(H^2 - D^2)}{R}$$

or

$$q(R) = \frac{4K(H^2 - D^2)}{L^2}$$

where

$q$ = drain discharge $(m/d)$

This equation, which was derived by Hooghoudt in 1936, is also known as the Donnan Equation

This equation can be rewritten as:

$$q = \frac{4K(H^2 - D^2)}{L^2}$$

From last figure, that $H - D = h$ and thus $H + D = 2D + h$, where $h$ is the height of the watertable above the water level in the drain. Subsequently, the equation changes to:

$$q = \frac{8KDh + 4Kh^2}{L^2}$$

These considerations lead to the conclusion that, if the soil profile consists of two layers with different hydraulic conductivities, and if the drain level is at the interface between the soil layers, the equation can be written as:

$$q = \frac{8K_bDh + 4K_th^2}{L^2}$$

where

$K_t$ = hydraulic conductivity of the layer above drain level $(m/d)$

$K_r$ =hydraulic conductivity of the layer below drain level $(m/d)$

This situation is quite common, the soil above drain level often being more permeable than below drain level because the soil structure above drain level has been improved by:

- The periodic wetting and drying of the soil, resulting in the formation of cracks.

- The presence of roots, micro-organisms, micro-fauna, etc.

If the pipe or open drains do not reach the impervious layer, the flow lines will converge towards the drain and will thus no longer be horizontal (Figure 5.3 A). Consequently, the flow lines are longer and extra head loss is required to have the same volume of water flowing into the drains. This extra head loss results in a higher watertable.

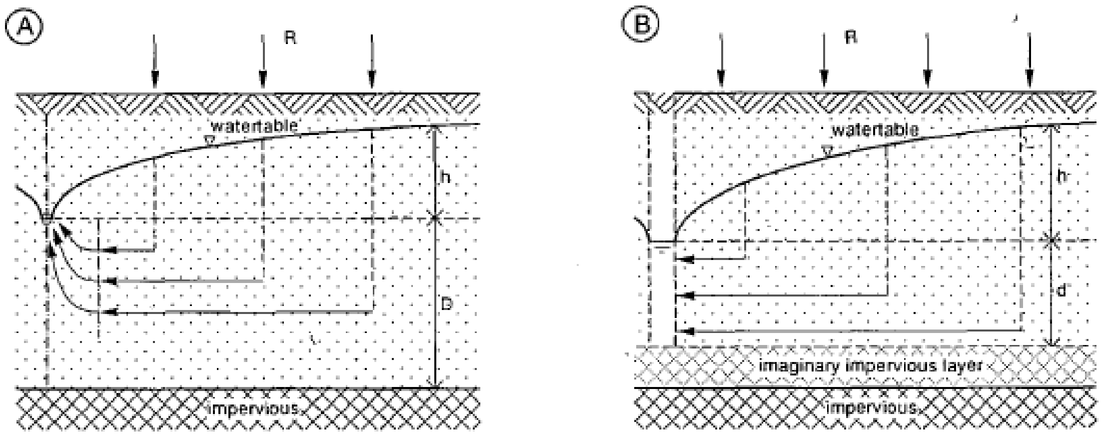


**Fig 5.3 The concept of the equivalent depth, d, to transform a combination of horizontal and radial flow (A) into an equivalent horizontal flow (B)**

To be able to use the concept of horizontal flow, Hooghoudt introduced two simplifications (Figure 5.3 B):

- He assumed an imaginary impervious layer above the real one, which decreases the thickness of the layer through which the water flows towards the drains.

- He replaced the drains by imaginary ditches with their bottoms on the imaginary impervious layer.

Under these assumptions, it´s possible to express the flow towards the drains, simply by replacing the actual depth to the impervious layer $(D)$ with a smaller equivalent depth $(d)$. This equivalent depth $(d)$ represents the imaginary thinner soil layer through which the same amount of water will flow per unit time as in the actual situation. This higher flow per unit area introduces an extra head loss, which accounts for the head loss caused by the converging flow lines. Hence the equation can be rewritten as:

$$q = \frac{8KDh + 4Kh^2}{L^2}$$

The only problem that remains is to find a value for the equivalent depth. Since the drain spacing $L$ depends on the equivalent depth $d$, which is a function of $L$, the equation can be solved by iteration. For this issue we will use the following formula:

$$d = \frac{D}{\frac{8}{\pi}\frac{D}{L} ln \frac{D}{2\pi \cdot r_0} + 1}$$

Where

$r_0$ = Equivalent hydraulic radio $(m)$

## 5.4 RUNOFF CALCULATION

It´s necessary to calculate also the risk of possible runoff on the field´s soil. To avoid this possibility, the potential flow available in the ditch should be higher than the flow of a possible runoff.

To resume, we have to check if:

$$Q_D > Q_R$$

The rational formula for the calculation for the runoff in every ditch can be described as:

$$Q_R = A \cdot \tau \cdot r$$

*where:*

$Q_R$ is the runoff rate $(m^3/s)$

$r$ *rainfall intensity* $(m/s)$

$A$ *drainage area* $(m^2)$

$\tau$ *runoff coefficient*

To estimate the runoff coefficient it´ll be used Annex 6 that shows the runoff coefficient for every kind of soil.

To calculate $Q_D$ it will be used the law of continuity, explained in paragraph 5.2.

It´s necessary to check the velocity of the water in the ditch, because it has never to overcome certain maximum recommended parameters, as it´s is shown in Annex 7.

## 5.5 EVAPOTRANSPIRATION DUE TO THE DITCHES

The equation that related the evapotranspiration losses with and the distance between ditches can be describes as:

$$L^2 = \frac{4 \cdot K}{E_t}(H_i^2 - z^2)$$

Where

$E_t$ is the potential evapotranspiration $(m/day)$

$H_i$ is the aquiculture depth or the distance between the surface and the impermeable soil $(m)$

$z$ is the distance between the impermeable soil and the allowed water table level $(m)$

Once calculated the potential evapotranspiration, it´s possible to estimate the flow lost by evapotranspiration as:

$$Q_{ET} = A_{drainagetotal} \cdot E_T$$

Where

$$A_{drainagetotal} = A_{drainage} \cdot n_{ditches}$$

# 6. ENGINEERING SOLUTION

## 6.1 DISTANCE BETWEEN DITCHES

As it was mentioned in paragraph 5.3 the Hooghoudt formula is used in order to relationated the distance between ditches with the discharge.

$$q(R) = \frac{8Kdh + 4Kh^2}{L^2}$$

Where $d$ represents the Hooghoudt equivalent depth

$$d = \frac{D}{\frac{8}{\pi}\frac{D}{L} ln \frac{D}{2\pi \cdot r_0} + 1}$$

Considering the open ditches of 0.9 m of height and 0.5 m of width in the bottom, with an inclination ratio of 1:1 (Figure 6.1):

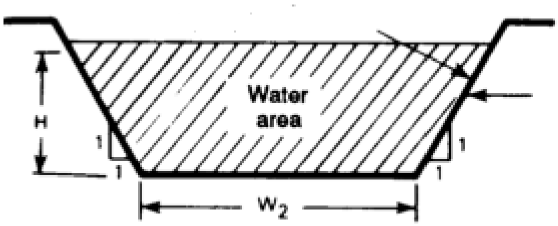


Figure 6.1 Transversal area of the ditch

It´s necessary to calculate the equivalent radius of the ditch, as we can define as the relation between the water area between the wetted perimeter.

The figure 6.2 shows the ditches system, according to Hooghoudt formula:

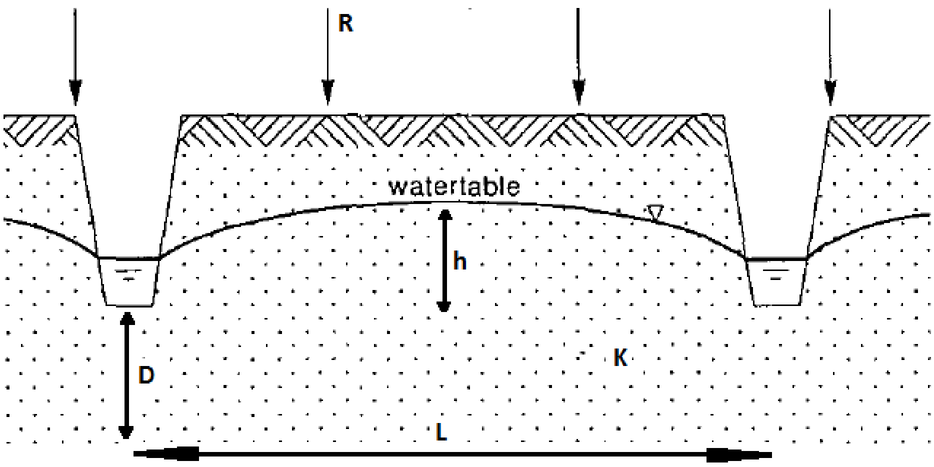


Figure 6.2 Ditches systems

To facility the possible alternatives of our design, it will be used the software Engineering Equation Solver (EES)

Ditch parameters

Width

$b = 0{,}5 \quad [m]$

Height ditch

$h_{ditch} = 0{,}9 \quad [m]$

Height water

$y = 0{,}25 \quad [m]$

angle ditch

$\alpha = 45 \quad [°]$

Wetted perimeter

$$w = b + 2 \cdot \frac{y}{\sin(\alpha)}$$

Water area

$$Area_{water} = b \cdot y + y^2$$

Equivalent radious

$$r_0 = \frac{Area_{water}}{w}$$

Field experiments shows that in our region we can consider the aquiculture depth (distance between soil surface and the impermeable layer) as 1.5 m. Therefore, considering our ditches as 0.9 m of depth:

$$D = 1.5 - 0.9 = 0.6\ m$$

**Water table depth**

This is the minimum depth below the surface at which the water table should be controlled and is determined by farming needs especially crop tolerance to water. Typically, it varies from 0.5 to 1.5 m. Drain depth, however, is constrained by soil and machinery limitations.

***Typical Drain Depths(D)***

| ***Soil Type*** | ***Drain Depth (m)*** |
|---|---|
| *Sandy loam* | *0.8 - 1.0* |
| *Clay loam* | *0.6 - 0.8* |
| *Sand* | *0.6* |
| *Silt loam* | *0.8 - 1.8* |
| *Peat* | *1.2 - 1.5* |

For the present project, it will be decreased the water table level 0.6 m bellows the surface, consequently:

$$h = 0.9 - 0.6\ m = 0.3\ m$$

In the same experiment, it was evaluated the conductivity of the soil, as results as (Annex 4):

$$K = 0.24\ m/day$$

According to the Hooghoudt equation, we must define $d$ (Hooghoudt equivalent layer), as it can be defined as:

$$d = \frac{D}{\frac{8}{\pi}\frac{D}{L}ln\frac{D}{2\pi \cdot r_0} + 1}$$

The objective is to evaluate the rainfall value relationated with the discharge and design the drainage system with the appropriate distance between ditches.

For this purpose we suppose the discharge between 0.5-4 mm/day.

As it can be shown in EES:

distance impermeable soil to ditch

$D = 0{,}6 \quad [m]$

distance between ditch and water table level

$h_{wl} = 0{,}3 \quad [m]$

Conductivity

$K = 0{,}24 \quad [m/day]$

Discharge calculation

$$q \cdot L^2 = 8 \cdot K \cdot d_1 \cdot h_{wl} + 4 \cdot K \cdot h_{wl}^2$$

$$q = \frac{q_1}{1000}$$

Hooghoudt´s equivalent layer

$$d_1 = \frac{D}{\frac{8}{\pi} \cdot \frac{D}{L} \cdot \ln\left[\frac{D}{2 \cdot \pi \cdot r_0}\right] + 1}$$

Finally, once obtained the results about the $L$ depending on the discharge $q$ , we can calculate the number of ditches, as it is well know the total distance of the main ditch in alternative 1 ($L_{total} = 630\ m$) or in alternative 2 ($L_{total} = 165\ m$)

number of ditches

$$n_{ditches} = \frac{L_{total}}{L}$$

## 6.2 RUNOFF CALCULATION

In this chapter it will be described the possible runoff that will be supported by the ditch $(Q_R)$ and the capacity of the ditches to be able to receive all the possible water$(Q_D)$. As it is explained in paragraph 5.4.

The rational formula for the calculation for the runoff in every ditch can be described as:

$$Q_R = A \cdot \tau \cdot r$$

*where:*

$Q_R$ is the runoff rate $(m^3/s)$

$r$ *rainfall intensity* $(m/s)$

For the conditions of South Moravia and if it´s supposed the worst situation, the rainfall intensity is assumed as:

$$r = 300(mm/day)$$

$A$ *drainage area* $(m^2)$*,* it depends on the alternative chosen.

$\tau$ *runoff coefficient*

According to Annex 6 that shows the runoff coefficient for heavy soils lawns, we assumed the runoff coefficient as:

$$\tau = 0.15$$

The drainage area depends on the alternative chosen as well.

$L$ distance between ditches

$L_{ditch}$ length of the ditch

$$L_{ditch} = 300\ m$$

Finally, the flow of the main ditch can be described as:

$$Q_{RTOTAL} = Q_R \cdot n_{ditches}$$

According to the previously mentioned:

RUNOFF

$Q_{runoff}$

$$Q_R = Area_{drainage} \cdot \tau \cdot r$$

$$Q_{RTOTAL} = Q_R \cdot 1000$$

$$r = \frac{r_1}{1000 \cdot 3600 \cdot 24 \cdot 1} \quad [m/s]$$

$$r_1 = 300 \quad [mm/day]$$

$$Area_{drainage} = 184900$$

$$\tau = 0{,}15$$

Consequently, it´s necessary to calculate the potential flow available in the ditch ($Q_D$), according to the law of continuity (Paragraph 5.2).

The slope of the main ditch is assumed as 1%.

Q ditch

$$Q_D = Area_{water} \cdot V_{ditch}$$

$$V_{ditch} = C \cdot \sqrt{r_0 \cdot I}$$

$$I = 0{,}01$$

$$C = \frac{1}{n} \cdot r_0^{(1/6)}$$

$$n = 0{,}04$$

For security reasons, it´s necessary to check the velocity of the water in the ditch, because it has never to overcome certain maximum recommended parameters, shows in annex 7.

## 5.3 EVAPOTRANSPIRATION

The equation that related the evapotranspiration losses with and the distance between ditches can be describes as:

$$L^2 = \frac{4 \cdot K}{E_t}(H_i^2 - z^2)$$

Where

$E_t$ is the potential evapotranspiration $(m/day)$

$H_i$ is the aquiculture depth or the distance between the surface and the impermeable soil

$$H_i = 1.5\ m$$

$z$ is the distance between the impermeable soil and the allowed water table level (Fig 6.2)

$$z = D + h_{wl}$$

Once calculated the potential evapotranspiration, it´s possible to estimate the flow lost by evapotranspiration as:

$$Q_{ET} = A_{drainagetotal} \cdot E_T$$

Possible Evapotranspiration

$$L^2 = \frac{4 \cdot K}{E_t} \cdot (H_i^2 - z^2)$$

$$E_{T2} = E_t \cdot 1000$$

$$z = D + h_{wl}$$

$$H_i = 1{,}5$$

$$Q_{ET} = Area_{drainage} \cdot E_t$$

$$Q_{ET2} = \frac{Q_{ET}}{24 \cdot 3600}$$

# 7. RESULTS

The relation between the discharge $(q)$ and the distance between ditches $(L)$ doesn't depend on the type of network. Besides, the water velocity on the channel $(V_{ditch})$ doesn't depend neither on the network, only in the dimensions of the channel. As it was explained in paragraph 5.1 :

$$V_{min} > V_{ditch} > V_{max}$$

Where:

$$V_{min} = 0.5\ m/s$$

$$V_{max} = 1\ m/s$$

The results are shown the following:

| 1..10 | $q_1$ [mm/day] | L [m] | $V_{ditch}$ [m/s] |
|---|---|---|---|
| Run 1 | 0,5 | 29,69 | 0,7224 |
| Run 2 | 1 | 21,09 | 0,7224 |
| Run 3 | 1,5 | 17,27 | 0,7224 |
| Run 4 | 2 | 15 | 0,7224 |
| Run 5 | 2,5 | 13,45 | 0,7224 |
| Run 6 | 3 | 12,3 | 0,7224 |
| Run 7 | 3,5 | 11,42 | 0,7224 |
| Run 8 | 4 | 10,7 | 0,7224 |

As we can see:

$$V_{min} > V_{ditch} > V_{max}$$

The results on the parameters $Q_R,, E_t$ and $Q_{ET}$ will depend on the alternatives already mentioned:

**ALTERNATIVE 1**

| 1..10 | $q_1$ [mm/day] | L [m] | $Q_{DTOTAL}$ [l/s] | $Q_{RTOTAL}$ [l/s] | $E_t$ [m/day] | $Q_{ET2}$ [m3/s] |
|---|---|---|---|---|---|---|
| **Run 1** | 0,5 | 29,69 | 135,4 | 96,3 | 0,001568 | 0,003355 |
| Run 2 | 1 | 21,09 | 135,4 | 96,3 | 0,003109 | 0,006654 |
| Run 3 | 1,5 | 17,27 | 135,4 | 96,3 | 0,004633 | 0,009916 |
| Run 4 | 2 | 15 | 135,4 | 96,3 | 0,006144 | 0,01315 |
| Run 5 | 2,5 | 13,45 | 135,4 | 96,3 | 0,007642 | 0,01635 |
| Run 6 | 3 | 12,3 | 135,4 | 96,3 | 0,00913 | 0,01954 |
| Run 7 | 3,5 | 11,42 | 135,4 | 96,3 | 0,01061 | 0,0227 |
| Run 8 | 4 | 10,7 | 135,4 | 96,3 | 0,01208 | 0,02585 |

**ALTERNATIVE 2**

| 1..10 | $q_1$ [mm/day] | L [m] | $Q_{DTOTAL}$ [l/s] | $Q_{RTOTAL}$ [l/s] | $E_t$ [m/day] | $Q_{ET2}$ [m3/s] |
|---|---|---|---|---|---|---|
| Run 1 | 0,5 | 29,69 | 135,4 | 63,8 | 0,001568 | 0,002223 |
| Run 2 | 1 | 21,09 | 135,4 | 63,8 | 0,003109 | 0,004408 |
| Run 3 | 1,5 | 17,27 | 135,4 | 63,8 | 0,004633 | 0,006569 |
| **Run 4** | 2 | 15 | 135,4 | 63,8 | 0,006144 | 0,008711 |
| Run 5 | 2,5 | 13,45 | 135,4 | 63,8 | 0,007642 | 0,01084 |
| Run 6 | 3 | 12,3 | 135,4 | 63,8 | 0,00913 | 0,01295 |
| Run 7 | 3,5 | 11,42 | 135,4 | 63,8 | 0,01061 | 0,01504 |
| Run 8 | 4 | 10,7 | 135,4 | 63,8 | 0,01208 | 0,01713 |

As we can see in both cases:

$$Q_D > Q_R$$

# 8. CONCLUSION

Once the results are obtained, it´s necessary to choose the correct drainage network besides the right parameters according to the suitability of the project.

Both possibilities satisfy the constrictions previously mentioned, the potential capacity of the main ditch is higher than the possible runoff calculated$(Q_D > Q_R)$ and the network is more homogeneous as well.

With the facts previously mentioned, the alternative chosen is alternative 1 due to it has a bigger drainage area, alternative 1 has exactly almost 185000 $m^2$ instead of 122000 $m^2$ of alternative 2.

For the other hand, this alternative would cause more economics costs, but in this project the priority is the effectiveness of the drainage system.

According to the distribution of the ditches, the possibility chosen it´s:

$$q = 2\ (mm/day\ );\ L = 15\ (m)$$

This distribution supposed a homogeneous network which can be measured easily on the field. Furthermore, the evapotranspiration assumed with this distribution $(E_T)$ is 6.2 $(mm/day)$ which is a normal value in the South Moravia region according to field studies. Values over this assumption would represent typical values for other environments, like wetland forests for example.

To conclude, the possible lost by evapotranspiration $(Q_{ET})$ doesn´t reach $0.02\ m^3/s$ which is a low value if it´s supposed the water source given by Hódonin power water station, above $2.5 - 3\ (\ m^3/s)$. Consequently, even with a lot of splits on the water distribution, loses by evapotranspiration $(Q_{ET})$ doesn´t represent a threat regarding water requirements.

To sum up, with the distribution selected, all the results with the possibility chosen are:

| | | |
|---|---|---|
| $\alpha = 45$ [º] | $Area_{drainage} = 184900$ [m$^2$] | $Area_{water} = 0,1875$ [m$^2$] |
| $b = 0,5$ [m] | $C = 18,33$ [-] | $D = 0,6$ [m] |
| $d_1 = 0,6313$ [m] | $E_t = 0,006144$ [m/day] | $E_{T2} = 6,144$ [mm/day] |
| $h_{ditch} = 0,9$ [m] | $H_i = 1,5$ [m] | $h_{wl} = 0,3$ [m] |
| $I = 0,01$ [-] | $K = 0,24$ [m/day] | $L = 15$ [m] |
| $L_{total} = 630$ [m] | $n = 0,04$ [m] | $n_{ditches} = 42$ [-] |
| $q = 0,002$ [m/day] | $q_1 = 2$ [mm/day] | $Q_D = 0,1354$ [m$^3$/s] |
| $Q_{DTOTAL} = 135,4$ [l/s] | $Q_{ET} = 1136$ [m$^3$/day] | $Q_{ET2} = 0,01315$ [m$^3$/s] |
| $Q_R = 0,0963$ [m$^3$/s] | $Q_{RTOTAL} = 96,3$ [l/s] | $r = 0,000003472$ [m/s] |
| $r_0 = 0,1553$ [m] | $r_1 = 300$ [mm/day] | $\tau = 0,15$ [-] |
| $V_{ditch} = 0,7224$ [m/s] | $w = 1,207$ [m] | $y = 0,25$ [m] |
| $z = 0,9$ [m] | | |

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## OTHER RESOURCES


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http://www.utdallas.edu/~brikowi/Teaching/Applied_Modeling/SurfaceWater/LectureNotes/Rational_Method/Runoff_Coefficient.html

# DESIGN OF A DRAINAGE SYSTEM TO IMPROVE THE WATER REGIMEN IN LANZHOT (SOUTH MORAVIA)

## ANNEX 1: ALTERNATIVE 1

JUNE 2016

1:10 000

# ALTERNATIVE 1

# DESIGN OF A DRAINAGE SYSTEM TO IMPROVE THE WATER REGIMEN IN LANZHOT (SOUTH MORAVIA)

## ANNEX 2: ALTERNATIVE 2

JUNE 2016

1:10 000

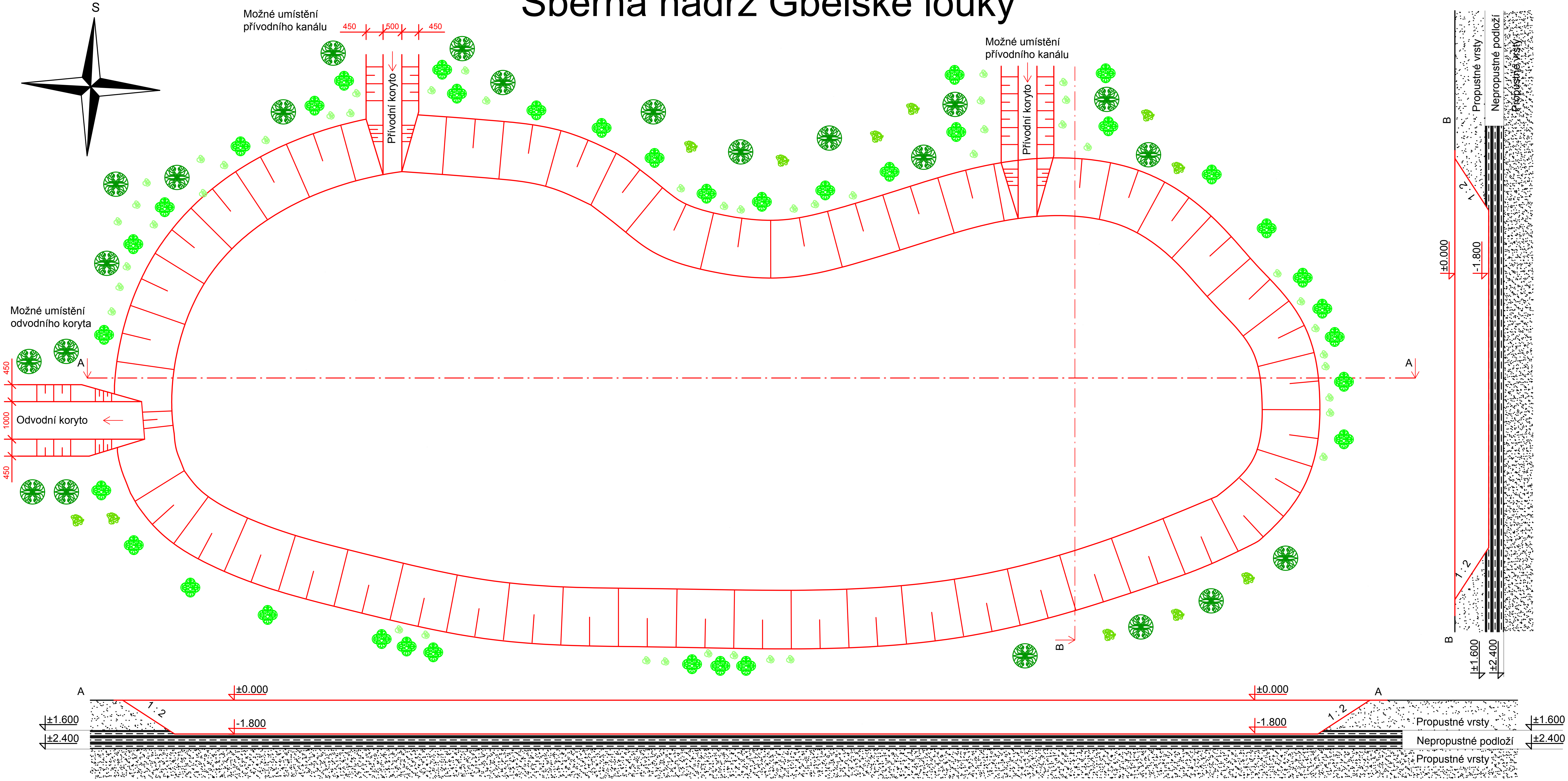
# ALTERNATIVE 2

# DESIGN OF A DRAINAGE SYSTEM TO IMPROVE THE WATER REGIMEN IN LANZHOT (SOUTH MORAVIA)

## ANNEX 3: POND

JUNE 2016

# Sběrná nádrž Gbelské louky




Souřadnicový systém: JTSK

LEGENDA:

- Dub zimní
- Olše lepkavá
- Vrba bílá
- Vrba křehká
- Navrhovaný stav

Pozn.: veškeré umístění odvodňovacích prvků podléhá místním podmínkám.

| | | | |
|---|---|---|---|
| ZODP. a VED. PROJEKTANT<br>prof. Dr. Ing. Miroslav Šlezingr | VYPRACOVAL<br>Ing. Jaroslav Blahuta | ZAKÁZ.ČÍSLO<br>----- | MENDELOVA UNIVERZITA<br>V BRNĚ<br>LESNICKÁ A DŘEVAŘSKÁ FAKULTA<br>ZEMĚDĚLSKÁ 3, 613 00 BRNO |
| KATASTR: Mikulčice | KRAJ: JIHOMORAVSKÝ | | |
| POŘIZOVATEL: | | | STUPEŇ: DUR |
| AKCE | MELIORAČNÍ ZÁSAHY - GBELSKÉ LOUKY | | DATUM: 8/2015 |
| ČÁST | C. SITUAČNÍ VÝKRESY | | ČÍSLO PARÉ |
| PŘÍLOHA | C.2 SITUACE | | MĚŘÍTKO 1 : 200 |

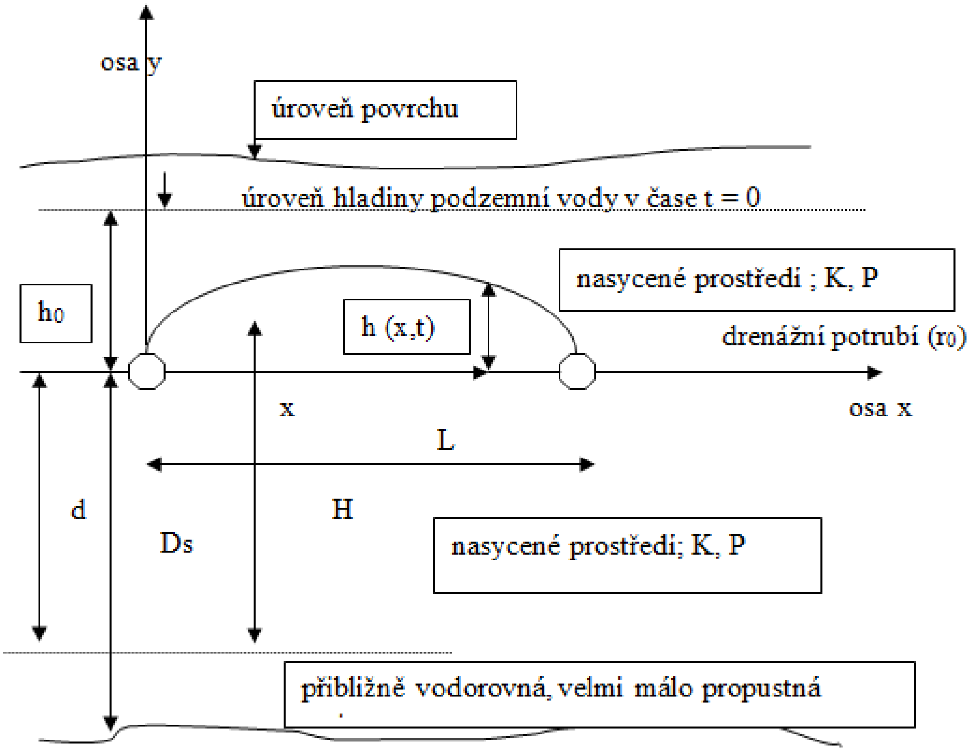
# DESIGN OF A DRAINAGE SYSTEM TO IMPROVE THE WATER REGIMEN IN LANZHOT (SOUTH MORAVIA)

## ANNEX 4: K-VALUE

JUNE 2016

*Sporadický (ojedinělý) příkop*

Po odvedení povrchových vod z úrovně terénu bude hladina vody ve sporadickém (ojedinělém) příkopu na určité, relativně vysoké úrovni $H_0$ (M) ze které bude regulací snížena na úroveň $H_1$ (M). Je možné předpokládat, že v průběhu povrchového odtoku a bezprostředně po odvedení povrchových vod do retenční nádrže bude počáteční úroveň volné hladiny v příkopu $H_0$ = 1,5 m. Řízenou regulací pomocí pohyblivého stavítka byla nastavena (snížena) úroveň volné hladiny vody v příkopu asi 5 cm nad dno příkopu,




Obr.GM1a. Výška vody nad drény h (x,t) ve vzdálenosti x > 0 od drénu v čase t > 0 v podmínkách nasyceného neustáleného drenážního proudění.

*Základní vstupní data a předpoklady řešení*

Parametry hodnoceného příkopu (lichoběžníkového tvaru):

Šířka ve dně: 1,0 m
Hloubka dna: 0,9 m pod úrovní terénu
Sklon stran: 1:1

Mocnost zvodnělé vrstvy: 1,50 m, hydraulická vodivost: 0,24 m/den, efektivní drenážní pórovitost: 0,048 (-).

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# DESIGN OF A DRAINAGE SYSTEM TO IMPROVE THE WATER REGIMEN IN LANZHOT (SOUTH MORAVIA)

## ANNEX 5: MANNING COEFFICIENT

JUNE 2016

| Channel Description | Average Value of $n$ |
|---|---|
| Grassland | |
| Short grass | 0.030 |
| Tall grass | 0.035 |
| Cultivated ground | |
| Bare ground | 0.030 |
| Mature row crops | 0.035 |
| Mature field crops | 0.040 |
| Brushy areas | |
| Dense weeds and sparse brush | 0.050 |
| Brush-covered with some trees (winter) | 0.050 |
| Brush-covered with some trees (summer) | 0.060 |
| Dense brush (winter) | 0.070 |
| Dense brush (summer) | 0.100 |
| Forested | |
| Densely covered with willows (summer) | 0.150 |
| Cleared land with stumps; no new growth | 0.040 |
| Cleared land with stumps; dense new growth | 0.060 |
| Dense stands of large trees; flood stage below branches | 0.100 |
| Dense stands of large tress; flood stage reaching branches | 0.120 |

As It shows in the table, according to the characteristics of the project :

$$n = 0.04$$

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# DESIGN OF A DRAINAGE SYSTEM TO IMPROVE THE WATER REGIMEN IN LANZHOT (SOUTH MORAVIA)

## ANNEX 6: RUNOFF COEFFICIENT

JUNE 2016

**TABLE 7–10** Runoff Coefficients for the Rational Method

| Description of Area | Range of Runoff Coefficients | Recommended Value* |
|---|---|---|
| Business | | |
| Downtown | 0.70–0.95 | 0.85 |
| Neighborhood | 0.50–0.70 | 0.60 |
| Residential | | |
| Single-family | 0.30–0.50 | 0.40 |
| Multiunits, detached | 0.40–0.60 | 0.50 |
| Multiunits, attached | 0.60–0.75 | 0.70 |
| Residential (suburban) | 0.25–0.40 | 0.35 |
| Apartment | 0.50–0.70 | 0.60 |
| Industrial | | |
| Light | 0.50–0.80 | 0.65 |
| Heavy | 0.60–0.90 | 0.75 |
| Parks, cemeteries | 0.10–0.25 | 0.20 |
| Playgrounds | 0.20–0.35 | 0.30 |
| Railroad yard | 0.20–0.35 | 0.30 |
| Unimproved | 0.10–0.30 | 0.20 |

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surface in the drainage area. This procedure often is applied to typical "sample" block as a guide to selection of reasonable values of the coefficient for an entire area. Coefficients with respect to surface type currently in use are listed below.

| Character of Surface | Range of Runoff Coefficients | Recommended Value* |
|---|---|---|
| Pavement | | |
| Asphaltic and Concrete | 0.70–0.95 | 0.85 |
| Brick | 0.75–0.85 | 0.80 |
| Roofs | 0.75–0.95 | 0.85 |
| Lawns, sandy soil | | |
| Flat, 2% | 0.05–0.10 | 0.08 |
| Average, 2 to 7% | 0.10–0.15 | 0.13 |
| Steep, 7% | 0.15–0.20 | 0.18 |
| Lawns, heavy soil | | |
| Flat, 2% | 0.13–0.17 | 0.15 |
| Average, 2 to 7% | 0.18–0.22 | 0.20 |
| Steep, 7% | 0.25–0.35 | 0.30 |

The coefficients in these two tabulations are applicable for storms of 5- to 10-year frequencies. Less frequent, higher intensity storms will require the use of higher coefficients because infiltration and other losses have a proportionally smaller effect on runoff. The coefficients are based on the assumption that the design storm does not occur when the ground surface is frozen.

*Recommended value not included in original source.

*Source: Design and Construction of Sanitary and Storm Sewers*, American Society of Civil Engineers, New York, p. 332, 1969.

According to the characteristics of the project (Soil lawn, heavy soil):

$$\tau = 0.15$$

# DESIGN OF A DRAINAGE SYSTEM TO IMPROVE THE WATER REGIMEN IN LANZHOT (SOUTH MORAVIA)

## ANNEX 7: V.MIN AND V.MAX RECOMMENDED

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| canal material | maximum velocity, m/s |
|---|---|
| fine sand | 0.45 |
| sandy loam | 0.55 |
| silt, silty loam | 0.6 |
| loam | 0.75 |
| fine gravel | 0.75 |
| clay | 1 |
| clay loam | 1.1 |
| coarse gravel | 1.25 |
| densely compacted clay loam | 2 |
| low density polyethylene, unprotected | 2 |
| grass | 2 |
| bricks, dry laid | 3 |
| stone masonry, mortared | 3.5 |
| cement/sand plaster | 4 |
| concrete | >5 |

*table 10.3 Permissible non-scouring mean velocities*

| sediment type | minimum velocity, m/s |
|---|---|
| fine silt | 0.3 |
| fine sand | 0.4 |
| sandy loam | 0.5 |

*table 10.4 Non-silting mean velocities*

According to the characteristics of the project:

$$V_{min} = 0.5\ m/s$$

$$V_{max} = 1\ m/s$$

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## MAP 1: LANŽHOT LOCALIZATION

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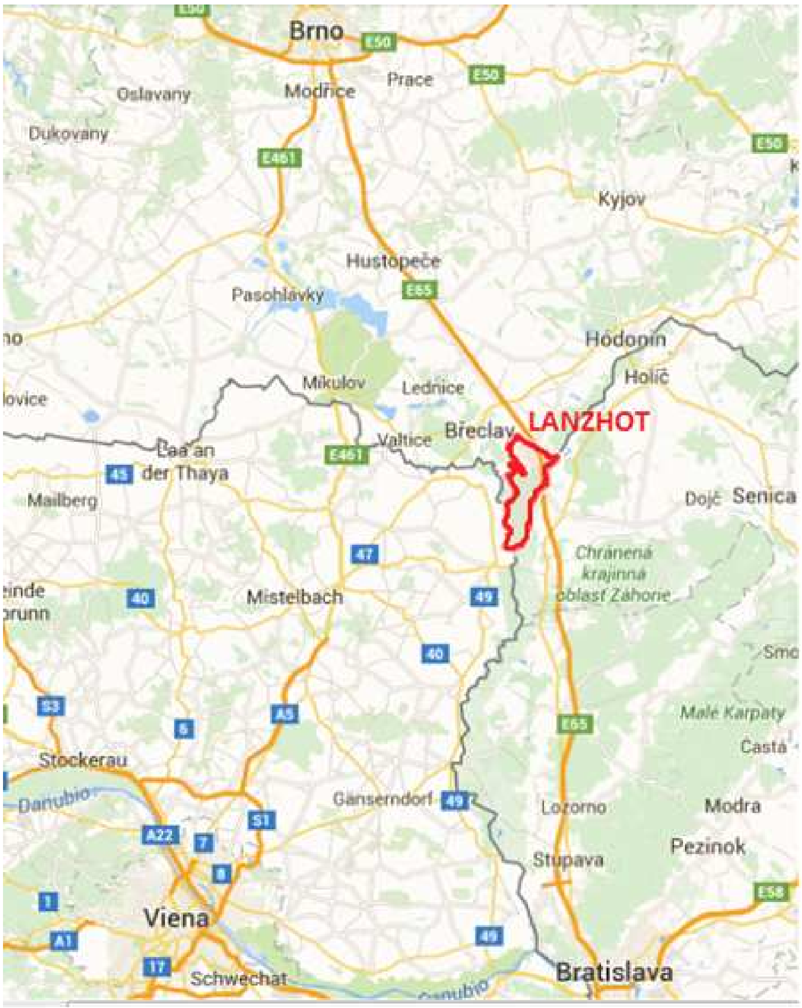



| | | | |
|---|---|---|---|
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## MAP 2:FIELD LOCALIZATION

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## MAP 3:GEOLOGIC MAP

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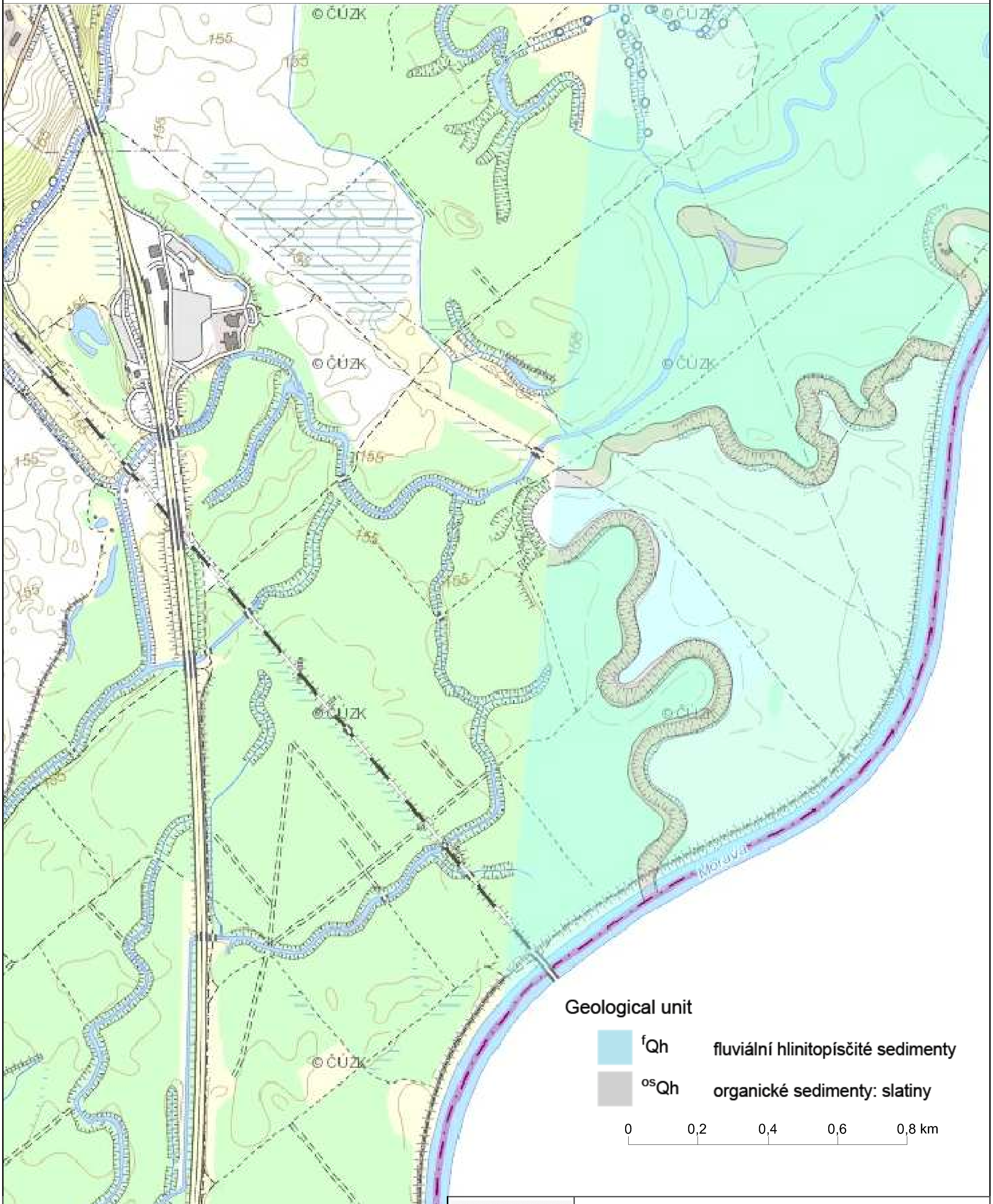
# Geological map 1 : 25,000

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| | |
|---|---|
| TITLE: GEOLOGICAL MAP | PLAN Nº: 3 |
| STUDENT: JUAN SURRA | TUTOR: JAKUB STIBINGUER |
| DATE: MAY 2016 | ESCALA: 1:25000 |