CZECH TECHNICAL UNIVERSITY IN PRAGUE

Faculty of Civil Engineering

Department of Hydrotechnic Structures

MASTER'S THESIS

Fuse Plugs and Gates as Dam Safety Improvement Measures

Využití Odplavitelného Hrazení pro Zvýšení Bezpečnosti Vodních Děl

Study program:	Civil Engineering
Branch of study:	Water Engineering and Management
Supervisors:	Ing. Miroslav Brouček, Ph.D.
	Prof. Dr.-Ing. habil. Reinhard Pohl

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ZADÁNÍ DIPLOMOVÉ PRÁCE

I. OSOBNÍ A STUDIJNÍ ÚDAJE

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Studijní program: Stavební inženýrství

Studijní obor: Vodní hospodářství a vodní stavby

II. ÚDAJE K DIPLOMOVÉ PRÁCI

Název diplomové práce: Využití odplavitelného hrazení pro zvýšení bezpečnosti vodních děl

Název diplomové práce anglicky: Fuse plugs and gates as dam safety improvement measures

Pokyny pro vypracování:
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Proveďte variantní návrh nouzového přelivu hrazeného odplavitelným hrazením pro zabezpečení vodního díla Ostrov u obce Ostrov nad Oslavou. Požadovaný rozsah výkresové dokumentace variant je na úrovni studie. U návrhu vyjděte z informací poskytnutých o vodním díle zaměstnanci státního podniku Povodí Moravy.
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Seznam doporučené literatury:
Novak, P., Moffat, A.I.B., Nalluri, C. and Narayanan, R. 2007: Hydraulic Structures 4th ed.; Taylor & Francis
Water Control Gates; Guidelines for Inspection and Evaluation; ASCE; 2012; 978-0-7844-1220-6
Přehrady, Broža, V., Kratochvíl, J., Peter, P., Votruba, L., 04-728-87, SNTL 1987
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Jméno vedoucího diplomové práce: Ing. Miroslav Brouček, Ph.D.

Datum zadání diplomové práce: 5.10.2018 Termín odevzdání diplomové práce: 6.1.2019
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In Prague / V Praze dne .... Jiří Wildt ....

Annotation

This diploma thesis is focused on fuse plug and fuse gate systems placed most often in auxiliary or emergency spillways, and their utilization in dam safety improvement measures. The thesis describes these systems, their different types, their design, their hydraulics and their utilization in practice. Then the dam Ostrov nad Oslavou is described and analyzed, and study project with the design of safety improvement measure of this waterworks by fuse plug and fuse gate systems. At last, the process of the theoretical 1000-year flood through the reconstructed dam is assessed.

Keywords

Auxiliary spillway, emergency spillway, dam safety improvement measures, fuse plug, fuse gate, flood, reservoir capacity, spillway capacity, overflow, discharge, dam/waterworks Ostrov nad Oslavou, 1000-year flood.

Anotace

Tato diplomová práce je zaměřena na systémy odplavitelných hrázek a odplavitelného hrazení, nejčastěji umístěných na pomocných nebo nouzových přelivech, a na jejich využití při zvýšení bezpečnosti na vodních dílech. Práce popisuje oba systémy, jejich různé typy, jejich návrh, jejich hydraulické chování a jejich využití v praxi. Dále je zde popsáno vodní dílo Ostrov na Oslavou a zpracován projekt studie s návrhem na zlepšení bezpečnosti tohoto vodního díla s využitím systémů odplavitelných hrázek a odplavitelného hrazení. Nakonec je posouzen průchod teoretické 1000leté povodňové vlny zrekonstruovaným vodním dílem.

Klíčová slova

Pomocný přeliv, nouzový přeliv, zvýšení bezpečnosti na vodních dílech, odplavitelné hrázky, odplavitelné hrazení, povodeň, kapacita zásobního prostoru, kapacita přelivu, průtok přes přeliv, odtok, přehrada/vodní dílo Ostrov nad Oslavou, 1000letá povodňová vlna.

Acknowledgements

Here, I would like to express my gratitude to Ing. Miroslav Brouček, Ph.D. who was the supervisor of my thesis from the Czech Technical University in Prague. He provided me many sources which were necessary to make the thesis and he found the supervisor for me at the foreign university where I was during the work on this project. Also, I would like to thank my supervisor from the Dresden Technical University, Prof. Dr.-Ing. habil. Reinhard Pohl, whose ideas and comments were very beneficial and useful for me.

Table of contents

1. Introduction

The purposes of this thesis are the theoretical part and the practical part. The theoretical part is focused on fuse plug and fuse gate systems placed most often in auxiliary or emergency spillways, and their utilization in dam safety improvement measures. The theory of the thesis includes collection of information about fuse plugs and fuse gates which are taken mostly from foreign sources. Gated spillways what fuse plug and fuse gate systems are, are discussed in the Czech Republic and also in the world more and more. It is due to draught periods which are nowadays longer than before, and due to extreme precipitation events which appear nowadays more often than before. The main advantage of the gated spillway is that it can simultaneously increase the water level in the reservoir (or capacity of the reservoir) and improve the safety of the dam.

The practical part is to find satisfying solution for the Ostrov nad Oslavou dam. Its safety has to be improved by an auxiliary spillway. After the dam reconstruction, the dam should withstand 1000-year flood. Currently, the dam is able to withstand only 100-year flood and it does not meet with the requirements in the Czech Republic. The design should include the solution of the auxiliary spillway with a fuse plug or fuse gate system.

For this study project, technical drawings which are attached in the last chapter, were created by CAD programmes such as AutoCAD and AutoCAD Civil 3D, and calculations and graphs were performed by the Excel programme.

The theoretical part of this thesis begins by the introduction of the fuse plug system. This system has never been used in the Czech Republic and it is not very common solution in the world, too. Fuse plugs are more utilized for levees and mine tailing dams than for auxiliary spillways. The fuse gate system is described next. It has very similar function as the fuse plug has, but it works in a mechanical way. This system is used for auxiliary spillways more than the fuse plug system but in the Czech Republic, there is also none.

Theoretical and practical parts are divided by the description of the dam Ostrov nad Oslavou. The dam is located in the Czech Republic in the Jihlava region and it needs a solution which comprises its safety improvement to withstand 1000-year flood. The main sources for this part of the thesis were the operation manual of the Ostrov nad Oslavou dam [1] and the study project for the safety improvement of the Ostrov nad Oslavou dam [2] which does not include the solution of the fuse plug or the fuse gate.

The design of the auxiliary spillway with the fuse plug system follows as the practical part of the thesis. At first, the capacity of the existing spillway has to be detected. Then, the auxiliary spillway crest and the spillway channel are designed. Finally, the design of the fuse plug is performed. All parts of the fuse plug are designed there according to the theoretical part of the thesis.

At the end of this thesis, there are the lists of used and removed materials, the assessment of the 1000-year flood after the reconstruction of the dam and the conclusion where the most important things about this thesis are summarized.

2. Fuse plugs and fuse gates

Many dam failures are due to inadequate spillway capacity. It is approximately one-third of them and because of it in some countries the design flood requirements are tightened. Then the dams which do not meet the requirements are needed to raise the value of their design flood. It can be provided in many ways, for example, increasing the depth of overflow, widening the existing spillway, or the design of an auxiliary spillway (*Figure 1*). However, the additional capacity is often needed only for the low-probability floods and therefore these solutions can be impractical or may involve high investment costs. For these instances, a fuse plug or a fuse gate appear to be the best alternative. These options can be safe and economical when their constructions are designed properly and their best advantages include that the spillway is uncontrolled with the maximized storage capacity of the reservoir. [3], [4], [5]

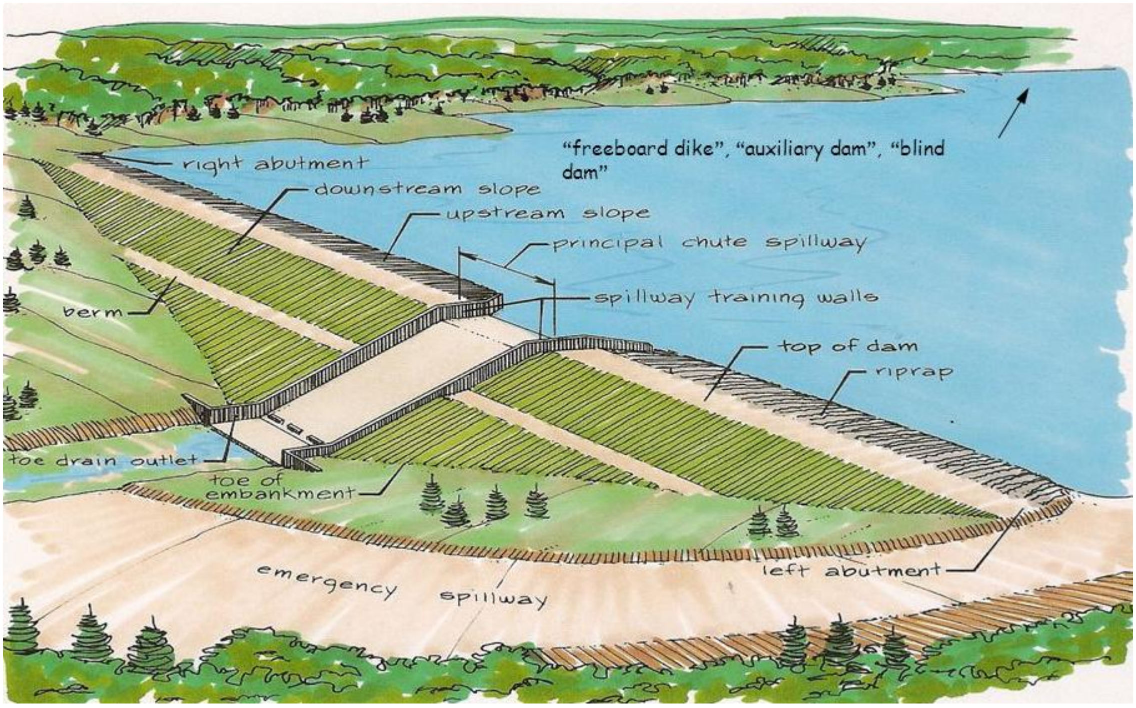


Figure 1 - Parts of a typical dam - mainly an emergency spillway [6]

2.1. Fuse plug

Fuse plug or in other words breaching section. It is a separate part of an earth dam which is predetermined for erosion. When the inflow to the tank exceeds, the spillway capacity and the specified level in the reservoir is reached, the fuse plug washes out. After the fuse plug is overtopped it starts to collapse. It collapses over a reasonable time frame gradually. As the fuse plug collapses, the surplus flood is released, the main dam is without danger and the reservoir level is not lower than before. This automatic solution reduces the possibility of mechanical or human error. An uncontrolled auxiliary spillway with a higher crest elevation for the same design flood would be very large because of a smaller depth of overflow. On the other hand, a fuse plug can be much shorter because the fuse plug embankment gets washed away there is a much deeper channel where the flood can be released. [3], [4], [5], [7], [8], [9], [10], [11], [12]

2.1.1. Criteria for using fuse plug

There are many factors which decide if the fuse plug embankment can be provided in an existing or a proposed dam or not.

Topography – A suitable saddle is required for a fuse plug. The saddle would be in a reasonable distance from the main dam because the discharging excess flood must not threaten the main dam. There should be a natural or an artificial tail channel at the saddle. The channel is connected with the same river or with a neighbourhood valley. [4], [5], [13]

Geology – Foundation of the fuse plug should be based on a rock with good quality so it could withstand the force of the water flow when the fuse plug is washed out. Usually, it is necessary to design concrete cut-off walls beneath the fuse plug embankment. The walls should prevent undermining of the foundation when deep overburden exists in the saddle. [5]

Downstream condition – The tail channel in which the water flows from the fuse plug should not be prone to clogging by the fuse plug eroded material and it should not threaten some other structures or buildings under the dam. [3], [4], [5]

Fuse plug foundation level – Foundation of the fuse plug influences a reservoir level. When the fuse plug is washed away and its non-erodible foundation is lower than Full reservoir level (FRL) the current water level will not be higher than non-erodible foundation until the fuse plug will be restored. This must be considered during the design. [4], [5]

Maintenance and operation costs – The cost of fuse-plug rehabilitation measures, repairs of downstream damages and a loss by a lower reservoir level during fuse plug restoration should be involved into an economic analysis account. [4], [5]

2.1.2. Fuse plug design

Fuse plug embankments are most used for levees and mine tailing dams. However, there are only few examples of actually operating fuse plug spillways. The designers do not have confidence for this type of spillways for several reasons. One of them is that the fuse plug tends to be stabilized and compacted because of armouring over a long period, traffic and vegetal growth. There is a question about the rate at which the fuse plug embankment would erode and pass the excess flood in the right time. If it would not, the main dam can be overtopped. [3], [5]

The upper limits of existing fuse plugs are:

- Unit discharge up to 83 cumecs/m
- Height up to 10 m
- Maximum head [from base elevation up to maximum water level (MWL)] of 13.5 m
- Breaching length up to 1200 m [5]

The principal features of a fuse plug spillway are:

- Pilot channel
- Impervious core
- Filters
- The composition of sand and gravel
- Slope protection [5]

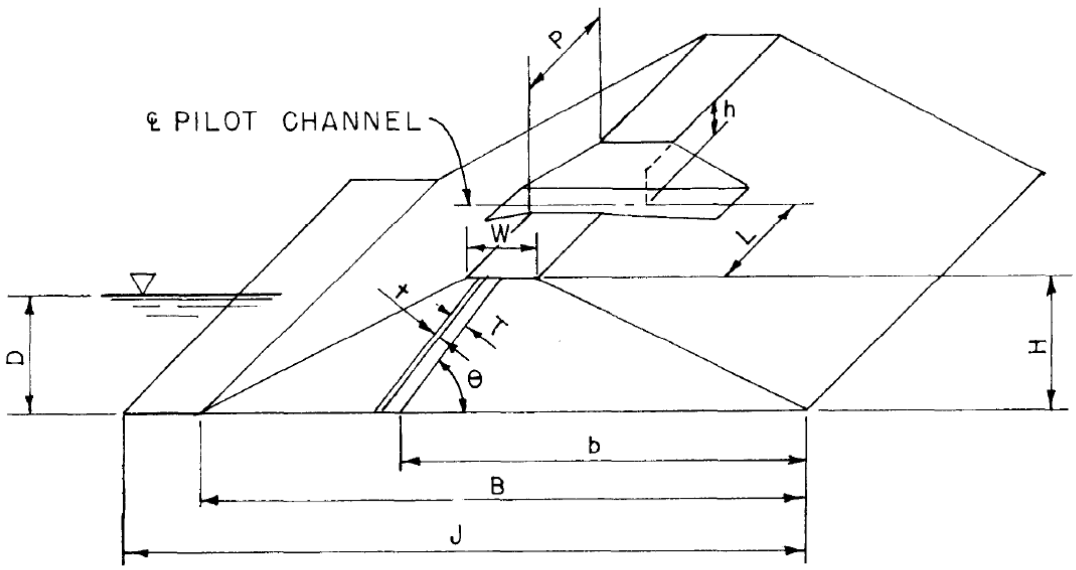


Figure 2 - Perspective view of the fuse plug embankment with the pilot channel [3]

The pilot channel is a short part of the embankment crest. When the water level is slightly lower than the crest, then the pilot channel is overtopped. Effective washout of the fuse plug is ensured by highly erodible material below the pilot channel. Instead of a pilot channel a piping device can be used. When the water level reaches the crest, the piping device is saturated and the material is eroded (*Figure 3*). Easily erodible material is used there. [5], [9], [11], [12]

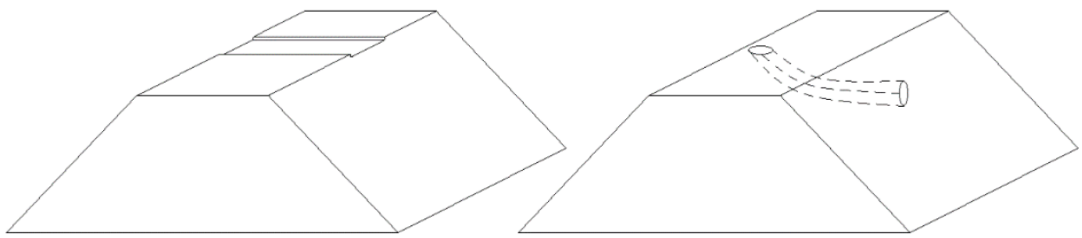


Figure 3 - Pilot channel on the left and piping device on the right [9]

One of the most important parts of the fuse plug is the impervious core. When a discharge is smaller than the design flood, the core prevents to its washout, but when the fuse plug is overtopped, the core collapses under its own weight. The core is a thin layer which is inclined in the downstream direction. [3], [5], [9], [11], [14]

There is a problem with the impervious core. Because the water level is usually not at this elevation, the core dries and cracks. Therefore, there are suitable filters. These filters should be provided as a cover of the core to prevent piping and undesirable washout of the fuse plug. The filters have also a significant effect on erosion rate. [3], [5], [9], [14]

The biggest part of the fuse plug is made up from sand and gravel. The rate of fuse plug washout is influenced by the size and gradation of this material. [3], [5], [9]

There is another issue with erosion by wind, waves, rainfall and snowmelt. This issue is solved by slope protection which is consisted of riprap and coarse gravel. It is provided on both sides of the fuse plug embankment on the upstream and downstream. [5], [14]

A fuse plug may not be in a function for many years so the materials from which the fuse plug is constructed must be very durable. [4]

Fuse plugs are designed by guidelines which are based on empirical relationships. These relationships were derived from hydraulic model studies of Oxbow project and hydraulic model studies for the performance of fuse plug embankment under breaching condition. [3], [5]

When a fuse plug is overtopped, the washout occurs in a predictable manner. For this reason, the fuse plug should be consisted of earth and rock-fill and designed as these types of dams. At the beginning of overtopping, the fuse plug should not be overtopped in its full length, but there is a pre-selected location which is a little bit lower than the entire fuse plug. This location, predetermined elevation, is called a pilot channel and highly erodible materials are placed there. The materials can be washed out easily and rapidly and the fuse plug embankment will washout laterally at a constant rate without being overtopped, then. It is the preferred method how to initiate a breaching of the fuse plug. Until the full capacity of the auxiliary spillway is not exceeded, the entire fuse plug embankment should not be washed out. [3], [5], [9], [11]

It was discovered that the erosion rate with the pilot channel near the centre of the embankment is similar to the erosion rate with the pilot channel close to one end of the embankment. The pilot channel width should be about 1/2 of the fuse plug height for passing breaching flow through the channel. [3]

If the fuse plug embankment is too long, we can use rigid walls which divide the whole embankment into sections. These walls are called splitter walls. Different crest elevation and different pilot channel elevation can be in each section, thereby the washout of each section occurs with the different water level. In this way, the washout process can be matched with successively less frequently occurring floods. [3], [4], [5]

Design of the rate of lateral erosion is very important. There is dependence between the rate of lateral erosion and the flood discharge through the auxiliary spillway. The flood discharge is limited by the elevation of the non-erodible foundation of the fuse plug. The rate of lateral erosion depends on many factors. These are mainly gradations and the type of used material, the depth of flow above the non-erodible foundation of the fuse plug and geometry of the fuse plug embankment. [3], [5], [9] The designer of the fuse plug must keep in his mind the economic life of the project and safety requirements of the country in which the fuse plug is designed. Then he decides for which interval of flood discharge the fuse plug will be designed. In general, the fuse plug should be in action with minimal 100-year flood and nowadays rather more. The flow velocity in the tail channel should be such to avoid clogging it by the eroded material from the fuse plug. [3], [4], [5]

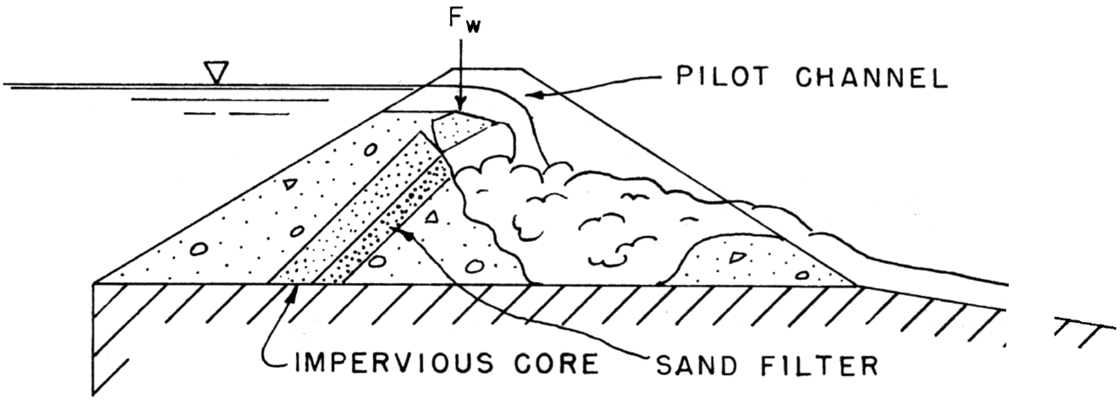


Figure 4 - Discharge through the pilot channel showing the failure of the impervious core [3]

Appropriate zoning of the embankment is important because in the other way the fuse plug will not work correctly by requirements. The impervious core is inclined in the downstream direction (*Figure 4*). During model studies which were under leading by Clifford Pugh, angle 30° and angle 45° above horizontal were used. It was discovered the 45° angle is better because with the 30° angle the material downstream was shielded more by the core. This design is required. When material behind the core is washed away, the impervious core degradates under the water load and under its own weight. The core normally consists of silt and clay. [3], [5], [9], [11], [14]

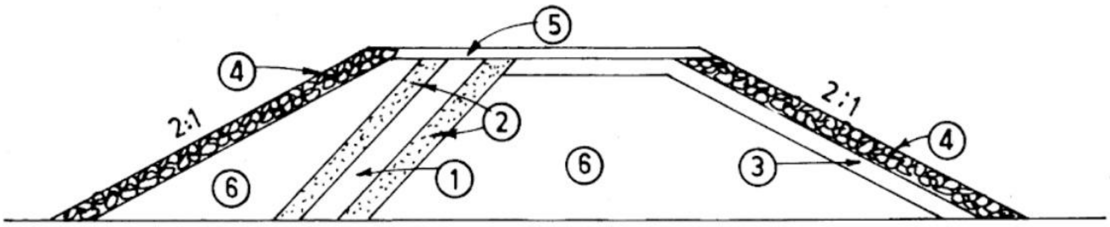


Figure 5 - Cross section of the pilot channel [3]
1 – Core; 2 – sand filter; 3 – sand and gravel; 4 – slope protection; 5 – gravel surfacing; 6 – compacted rock fill

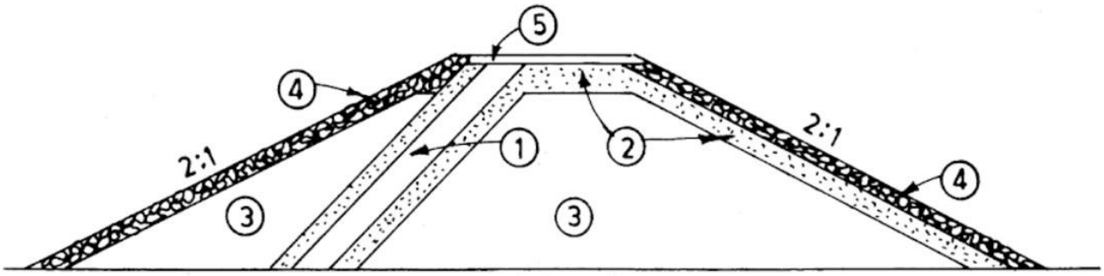


Figure 6 Cross section of the main embankment [3]
1 – Core; 2 – sand filter; 3 – sand and gravel; 4 – slope protection; 5 – gravel surfacing

On the upstream and downstream around the impervious core, there are filter zones. These zones prevent piping the core by water through some cracks which can be made due to drought. The washout is ensured by non-cohesive and easily erodible material. These materials are mainly in a part of the pilot channel, it is called rock-fill zone, and also it is the main section of the fuse plug embankment which is called sand and gravel zone (*Figure 7*). [3], [5], [14]

In the United States the model studies of the fuse plugs have been implemented. The model embankments simulated fuse plug prototypes from 3 to 9 metres high at scales of 1:10 and 1:25. In these studies, material gradation was mainly tested. The result is a series of gradation curves for embankment materials which were recommended for the pilot channel and for the main sections of the embankment (*Figure 7*). Some of the examples of typical cross-sections of designed fuse plugs are in the next figures (*Figure 8, Figure 9, Figure 10*). [3], [5]

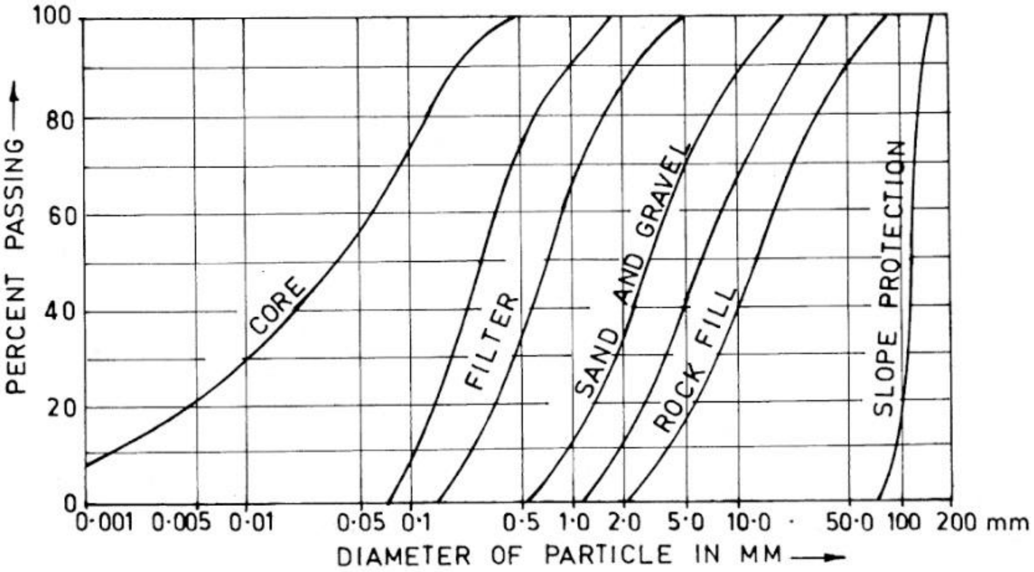


Figure 7 - Zoning of materials and their gradation curves [3]

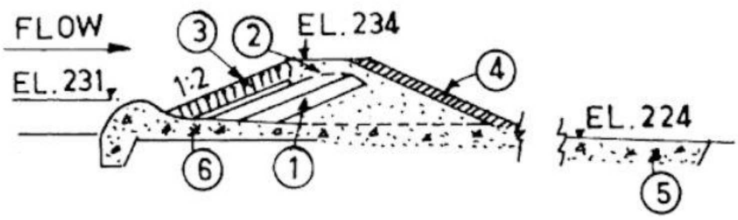


Figure 8 - Emergency spillway section of Mrica dam, Indonesia [7]
1 – Clay core; 2 – filter; 3 – rip rap; 4 – gravel fill; 5 – oversize rock; 6 – concrete weir

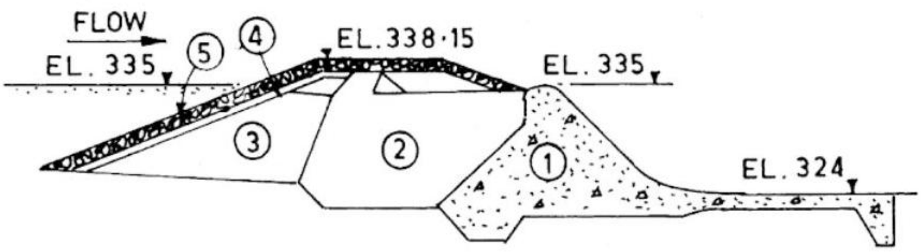


Figure 9 - Fuse plug spillway - typical cross-section [7]
1 – Concrete dam; 2 – clay core; 3 – sandy gravel; 4 – gravel filter; 5 – riprap

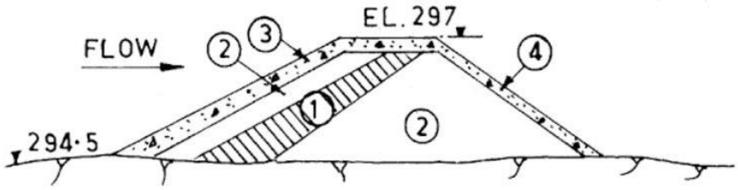


Figure 10 - Typical section of erodible bund: Mnjoli dam - Swaziland [7]
1 – Impervious core; 2 – filter sand; 3 – a blanket of concrete (500 mm); 4 – ditto (150 mm)

2.1.3. Providing a fuse plug in an existing dam

Sometimes an existing dam needs to raise the value of its design flood so there is a possibility to provide a fuse plug in it. Nevertheless, a suitable location for the fuse plug and for a tail channel has to be in the dam. The discharge through the tail channel must not endanger the main dam and other buildings or constructions in the neighbourhood. An important thing is segregating the part which must not be overtopped from the fuse plug part. In the construction of the fuse plug, a rigid non-erodible overflow structure is designed. The structure ensures the required safety of the spillway (*Figure 11*). [5], [13]

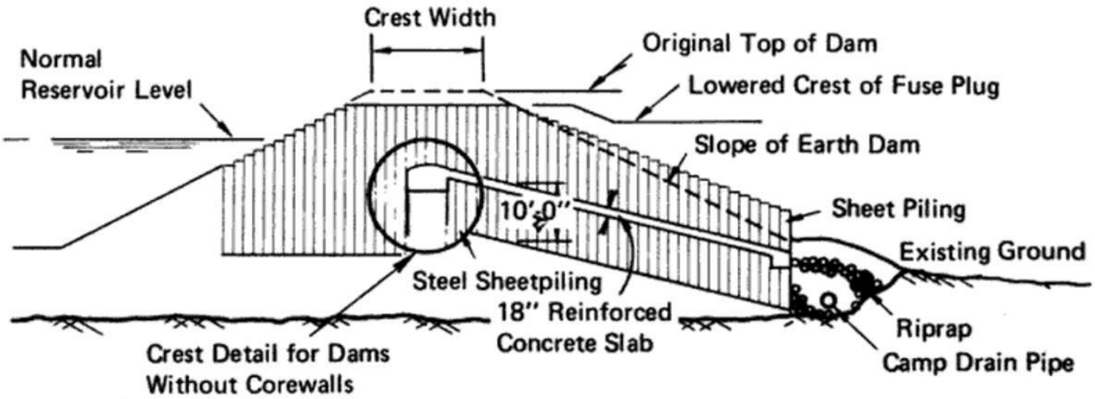


Figure 11 – A typical cross-section of fuse plug spillway in an existing dam embankment [13]

2.1.4. Hydraulic of fuse plugs

When the fuse plug is in function and the water is discharging through it, it is very similar to an earth dam breaching. However, the fuse plug embankment is breached in the section which is predetermined for breaching. If the fuse plug is full washed away the flow through the opening is the same as the flow over a broad crested weir. [3], [5]

Flow over the broad crested weir is dependent on the ratio between the depth of flow above the crest (H_0) and the length of the crest (J). If the ratio H_0/J is in the range between 0.08 and 0.5, the flow can be counted by *Equation 2.1*. If the ratio is lower than 0.08, the flow is controlled by the friction of the crest. [3]

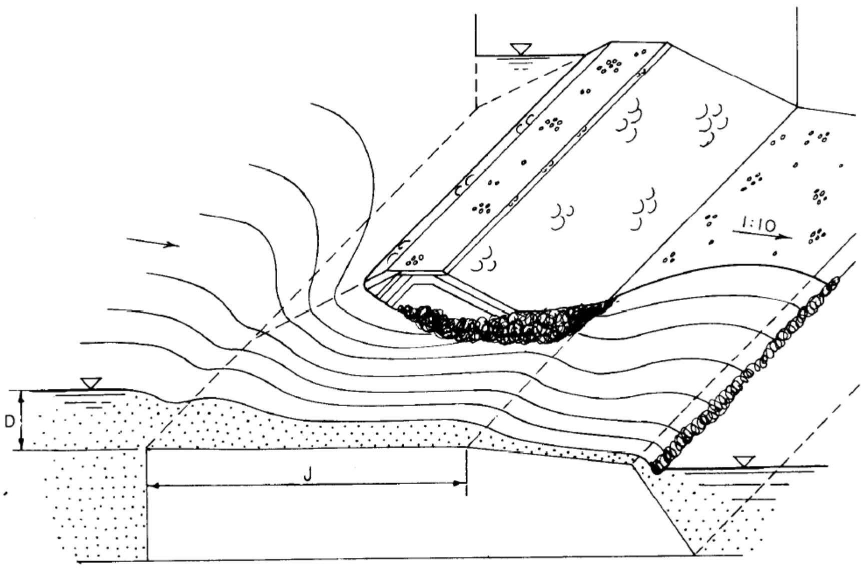


Figure 12 - Schematic of the lateral erosion process [3]

$$Q = C \cdot L \cdot H_0^{3/2} \tag{2.1}$$

Where Q – Flow over the broad crested weir [m^3/s]
C – Coefficient of discharge [$m^{1/2}/s$], its values are:
- During washout in one direction: $1.51\ m^{1/2}/s$
- During washout in both directions: $1.71\ m^{1/2}/s$
- After washout is complete: $1.44\ m^{1/2}/s$

L – Distance along fuse plug crest (breach length) [m]
H_0 – Water depth above the crest [m]

The rate of the lateral erosion can be estimated by *Equation 2.2*. The rate is after the initial breaching and for typical embankment design. This is an empirical equation which can be used for fuse plugs from 3 to 9 metres high. [3]

$$ER = 14.6\,H_f + 48 \tag{2.2}$$

Where ER – Lateral erosion rate [m/hour]
H_f – Height of the fuse plug [m]

Generally, there are two ratios which have significant effects on erosion rates. The first one is the depth of water to embankment height and the second one is the depth of water to weir width. [3]

2.1.5. Fuse plug model studies

Fuse plugs have been tested in many model studies in many countries around the whole world. Some model studies were implemented for one project. Some fuse plug guidelines are based on other studies.

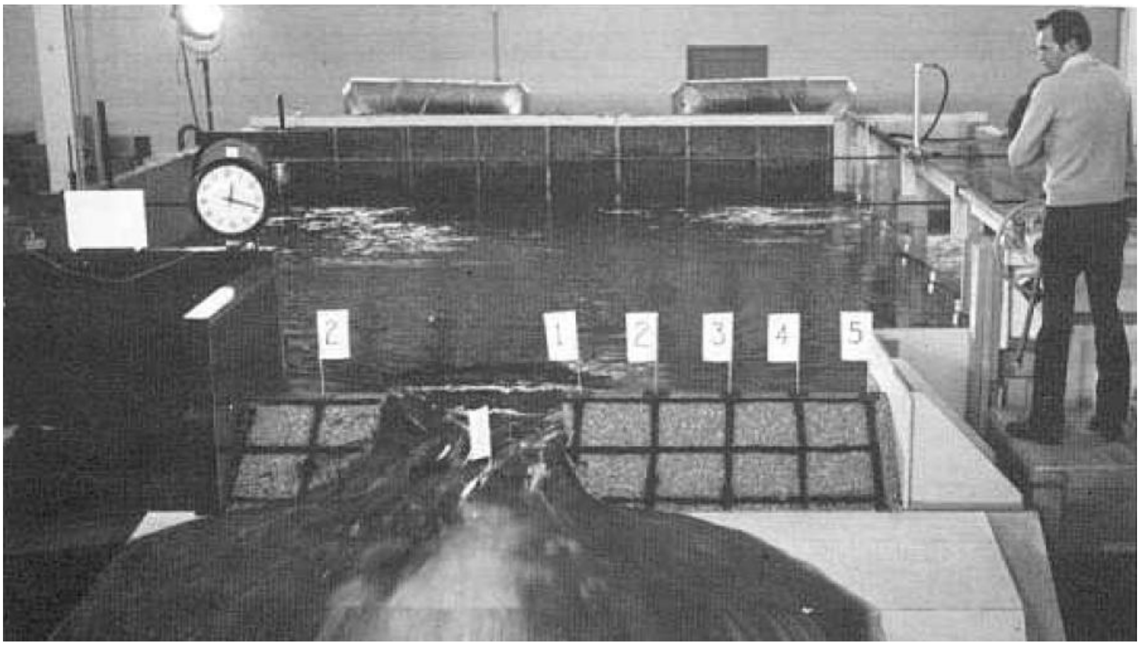


Figure 13 - Model of a fuse plug embankment (washout) [3]

The first easily findable model study is the Oxbow Fuse Plug Model Report which describes modelling of fuse plug spillway in the Oxbow dam from 1959. Many model studies have been implemented in U.S. Department of the interior bureau of reclamation institute, for example, Hydraulic model studies of fuse plug embankments from Clifford Pugh (1984), Hydraulic model study of Horseshoe dam fuse plug auxiliary spillway from Tony Wahl (1993) or Guidelines for using fuse plug embankments in auxiliary spillways from the committee of the USBR (1987).

One of the model study topic from Sweden is “Can a laboratory test replace a field test?”. This model study is made by SVC Vattenbyggnad Energiforsk in the year 2018. The model study compares results from models in small scales (1:3 and 1:6) with results from the model with full scale. The study shows that the models in smaller scales can replace the full-scale tests in the fuse plug modelling because their results are very similar sometimes identical. [9]

2.2. Fuse gates

Fuse gates work on a very similar principal as fuse plugs do but in a mechanical way. These are individual units which are placed side by side and they make a type of labyrinth weir. They need to be placed on a flat crested spillway. When a predetermined water level comes in a reservoir, the fuse gates automatically tilt and fall down from the crest. The fuse gates tilt in a sequential manner, one after the other in a predetermined mode. This is different from a fuse plug which is washed out full after its activation. The advantages of a fuse gated spillway are the storage capacity like with a gated spillway and flood control safety like with an ungated spillway. In developed countries, the storage of fresh water is enough for 1000 days, but in other countries which are not so developed, the storage is enough only for 150 days. So fuse gates are a very good solution for areas where the problem with the absence of fresh water is. This system of fuse gates was invented in 1989 and patented in the US, Europe and other countries by Hydroplus International Company from France in 1991. [5], [10], [15], [16], [17], [18], [19], [20], [21], [22], [23], [24] The storage capacity and the spillway capacity can be raised by fuse gate system. If there is only requirement for increasing the spillway capacity, the elevation of the fuse gate crest is close to the original spillway crest. When we want to raise also the storage capacity, the crest of the fuse gate must be set above the original crest elevation. Many units of fuse gates side by side are on an auxiliary spillway and they fill the whole width of the spillway. The existing spillways where this system has been used, have received the capacity increasing up to factor 1.71 and the spillway capacity increasing up to factor 8.5. The biggest fuse gates are 6.5 metres high. They are on the Terminus dam in the US and on the Shongweni dam in South Africa. [5], [10], [15], [17], [19], [21], [23]

2.2.1. Criteria for fuse gate selection

On an existing dam, the main criterium for selecting the fuse gate system is whether the existing dam can carry the additional loads imposed by fuse gates. [19]

Then the main criteria on all dams for selection fuse gates are:

- The required limits of control of the reservoir level. In this point the reservoir level after the fuse gates tilt to downstream during a flood is also included.
- The time and effort which is required after a flood to recover the fuse gate section in the initial condition. It is possible that replacement of damaged gates or repair of reusable gates would be needed.
- The frequency of flood events which cause tilting of the fuse gates. [19]

2.2.2. Fuse gates functioning

Figure 14 shows the main components from which each gate consists of. There are a bucket, a base and an intake well. The bucket can be made from steel, concrete or combination of both. The intake well is connected to a pressure chamber in the base. The intake well can be connected with the base by a conduit and can be located remotely. [5], [10], [15], [16], [20], [21], [23]

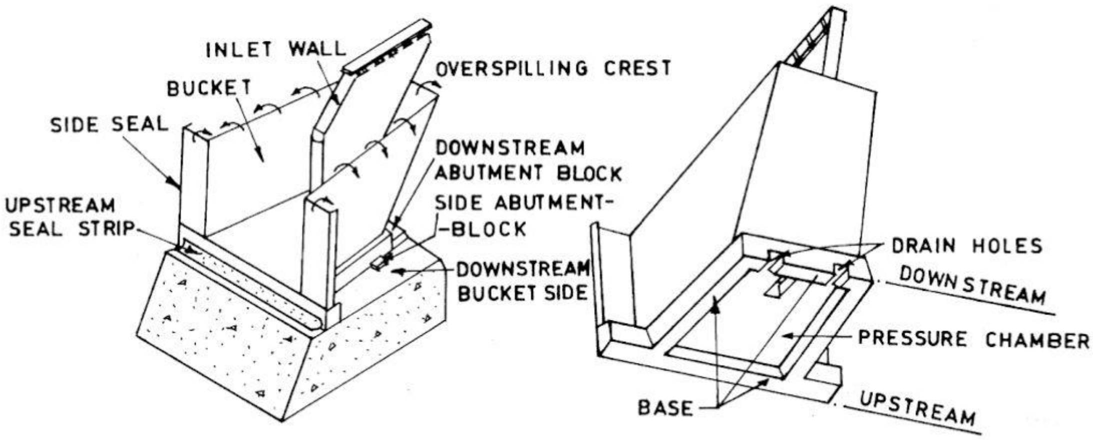


Figure 14 - Components of a standard fuse gate [25]

A flat rubber gasket seals the joint between the spillway crest and the fuse gates. A sealing joint system is also between fuse gates which make the system as a waterproof wall. Each fuse gate pressure chamber is equipped by drain holes to prevent accumulation of seepage water.[5], [15], [19], [21], [23]

Figure 15 explains how the Fuse gate works. When the discharge is up to the design flow, the fuse gate is like a labyrinth weir and water flows over it. Generally, the design flow is a minimal 100-year flood. The fuse gate crest is approximately 3 times longer than the fuse gate width. If the discharge increases over the design flow, water begins to flow into the pressure chamber through the well. The opening of the well is not horizontal but it follows the shape of the water surface on the sharp-crested weir. The water level in the well increases if the inflow through the well exceeds the outflow through the drain holes. Thereby, the pressure increases in the pressure chamber, there is an uplift force and the stability of the gate is decreased. A predetermined water level provides instability of the gate and then the gate tilts around its downstream edge. The tilting level of each gate is just the height of their inlet well, to ensure the gates tilt progressively in order to compensate for the effect of exceptional flood levels. The tail channel should be large enough to allow tilting of the gates and their elimination from it.[5], [10], [15], [17], [18], [20], [21]

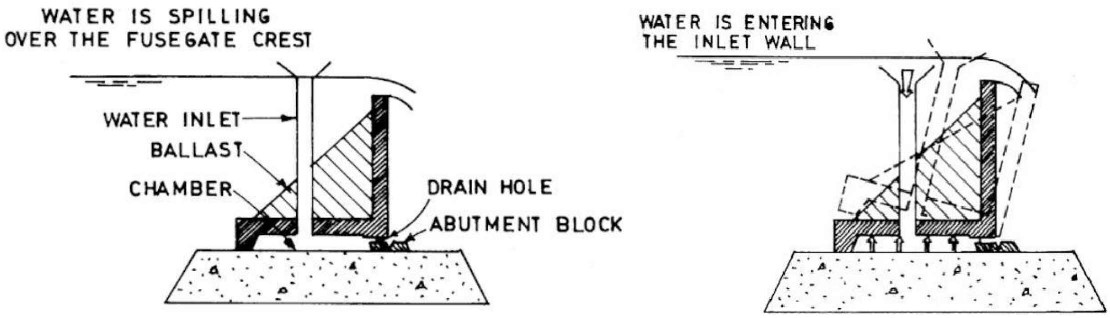


Figure 15 - Operating sequence of a fuse gate [25]

The fuse gates with labyrinth weir are available in some different variations. The types of fuse gates are divided by the width, W (wide) and N (narrow), and by the height of the opening of the well, LH (low head) or HH (high head) (*Figure 16*). The fuse gate range of heights is from 1.5 metres to 6.5 metres.[5], [21]

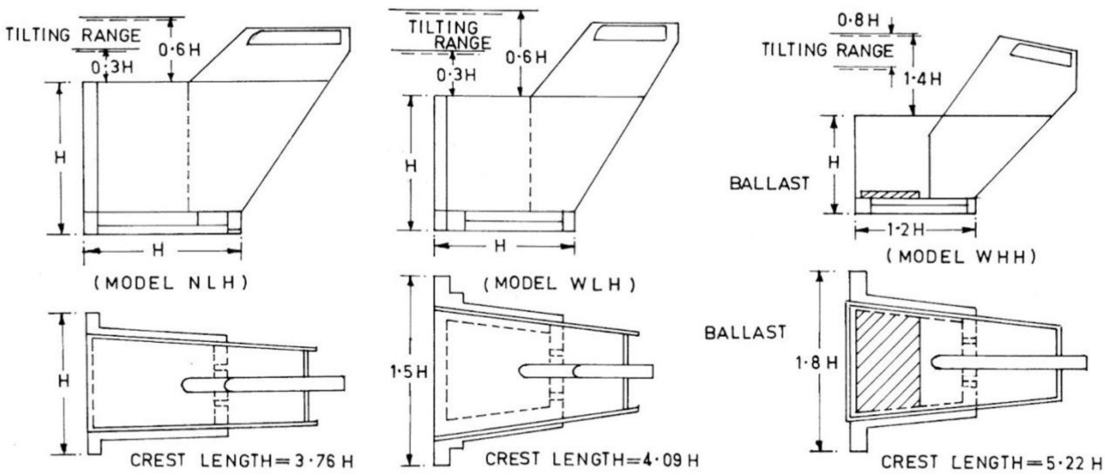


Figure 16 - Standard variations of fuse gates [5]

2.2.3. Fuse gate stability

Sliding stability of the fuse gate is ensured by downstream abutment blocks which are located in the downstream of the fuse gates. The blocks must withstand the horizontal load from the fuse gates and they are built into the spillway sill (Figure 14). [5], [15], [21]

For designing a fuse gate, the stability against overturning is more interesting. The point about which the fuse gate would be overturned is the downstream edge of it. Moments from every force acting on the fuse gate are counted to this point. The figure shows all forces which influence the fuse gate.[5], [19], [21]

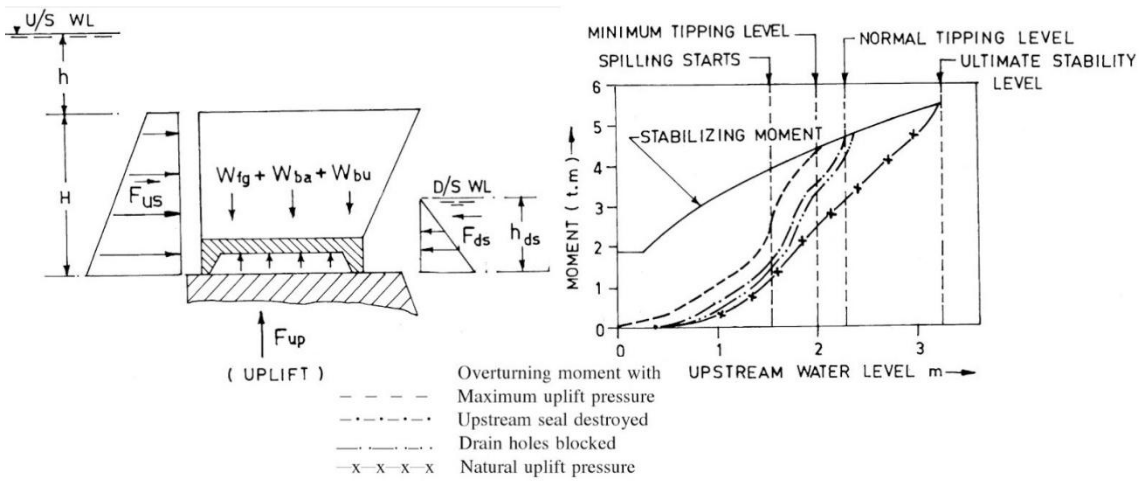


Figure 17 - Forces acting of fuse gate and analysis of stability [25]

Forces which make an overturning moment:

- F_{us} – Hydrostatic pressure from reservoir water on the upstream side of the fuse gate.
- F_{up} – Uplift pressure in the chamber and under the fuse gate base.

Forces which make a stabilizing moment:

- W_{fg} – The dead weight of the fuse gate.
- W_{ba} – Weight of the ballast (if any).
- W_{bu} – Weight of the water on the bucket floor.
- F_{ds} – Back pressure from the downstream water.

If the water level in the reservoir is below a minimum tipping level, the fuse gates cannot overturn. Even if the water enters the well by an accident, but the reservoir level remains below the minimum tipping level, the gate would return back to its position. This level corresponds to a head of the fuse gate crest which is 60-80 % of the normal overturning head. From the time when the water achieves the well to the time the fuse gate is overturned is approximately 2-3 % of the height of the fuse gate. [5], [15]

Some failures can happen, for example, blocked drain holes or damaged upstream sill, then uplift pressure and overturning moments can come. However, their influence is not so big to overturn the gate in normal conditions. Even if there is something wrong with the intake well, for example, blocked the intake by something, and water cannot enter the well, the overturning moment increases with raising the water level and there is an upstream water level by which the fuse gate tilts anyway. This water level is called the ultimate stability level H_u. [5]

2.2.4. Fuse gate design

There are several parameters by which the fuse gates are selected. These are spillway width, sill elevation, fuse gate required height, the quantity of fuse gates and the water levels in the reservoir at which the fuse gates will be overturned. [5], [21]

Generally, the fuse gates are installed on a flat surface of the spillway crest. If the spillway has an ogee crest, the top part of the crest has to be removed and provided with a flat surface (*Figure 18*). If the fuse gates are designed in the new dam a flat sill is usually provided. So standard ogee profile with better hydraulic parameters is replaced by a broad crested or a long-crested weir with different discharge characteristics. [5], [21], [23]

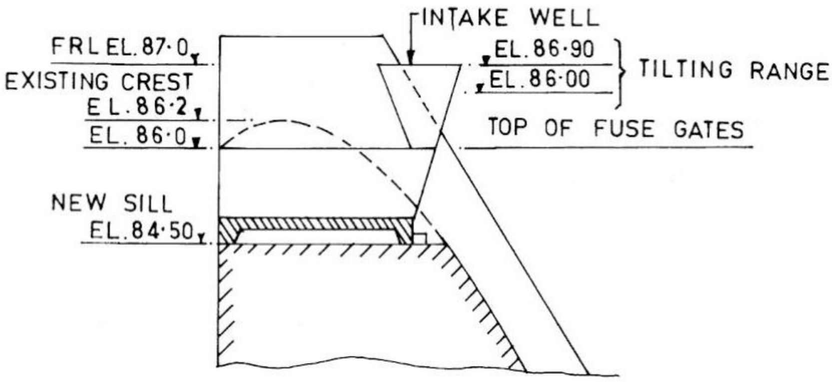


Figure 18 - Installation of fuse gate on an existing spillway [5]

2.2.5. Hydraulic of fuse gates

There are two types of discharge characteristics which are considered with fuse gate systems. One of these is discharge over the sharp crest of the fuse gate and the second one is the discharge over the flat horizontal sill on which at normal conditions fuse gates are placed. The fuse gate discharge can be counted by *Equation 2.3*. [5], [21]

$$Q = \frac{2}{3} \cdot \sqrt{2g} \cdot C_d \cdot L_f \cdot h^{3/2} \tag{2.3}$$

Where Q – Discharge over the sharp crest of the fuse gate [m^3/s]
g - Gravitational acceleration [m/s^2]
C_d – Coefficient of discharge [-]
L_f – Crest length of the fuse gate [m]
h – Depth of overflow [m]

For the coefficient of discharge, *Equation 2.4* fit as the best.[21]

$$C_d = C_1 \cdot \left[\left(\frac{h}{H}\right) - C_2\right]^{C_3} \tag{2.4}$$

Where C_1; C_2; C_3 – Experimentally determined constants for (h/H) greater than 0.1 (*Enclosure 1 - Empirical discharge coefficients for fuse gates*) [-]

H – Height of the fuse gate [m]

The second type of discharge, over the flat horizontal sill, is determined by the ratio of head H_0 to the length of the sill W_c. According to *Table 1*, it will be decided which equation can be used for different types of weir with different water level. Weirs are divided by the ratio to the sharp crested, short crested, broad crested or long crested weirs. [5], [21]

Weir crest ratio	Type of crest	Equation
$H_0/W_c \geq 1.5$	Sharp	2.3 with $C_d = 0.611+0.075\ (H_0/P)$ where P is the height of the weir above riverbed.
$0.4 \leq H_0/W_c < 1.5$	Short	Interpolate discharge between those given 2.3 and 2.5.
$0.1 \leq H_0/W_c < 0.4$	Broad	2.5
$H_0/W_c < 0.1$	Long	Compute water surface profile to reservoir assuming critical depth at crest.

Table 1 - Classification of the weir [21]

The discharge over a broad crested weir is counted by *Equation 2.5*.

$$Q = \frac{2}{3} H_0 \cdot \sqrt{\frac{2}{3} g \cdot H_0} \cdot L_s = 1.705\ L_s \cdot H_0^{3/2} \tag{2.5}$$

Where Q – Flow over the broad crested weir [m³/s]
H_0 – Water depth above the crest [m]
L_s – Width of the flow passage [m]

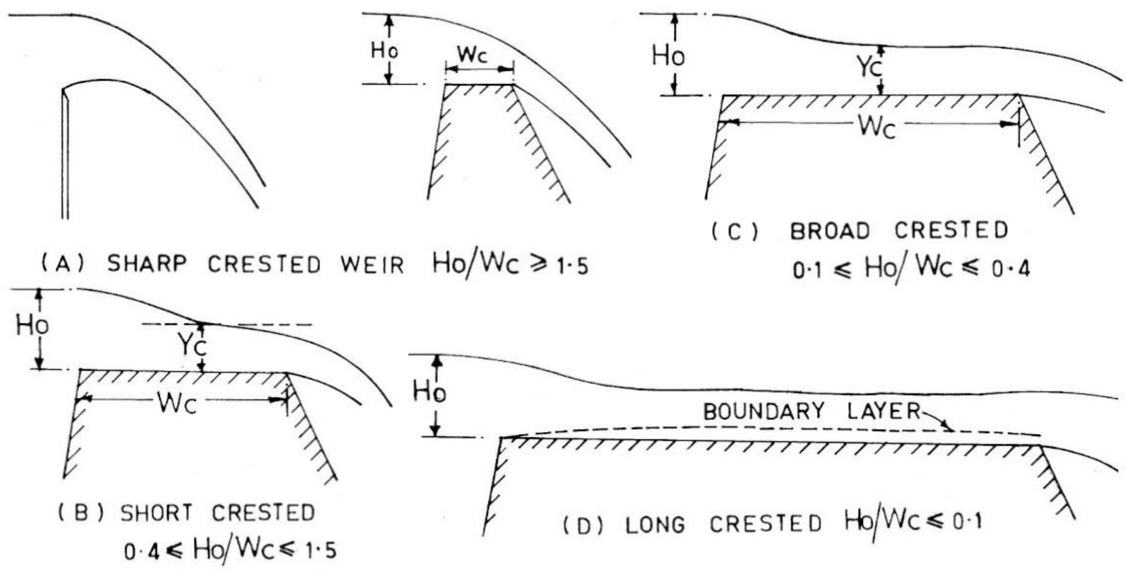


Figure 19 – Broad crested weirs [5]

The frictional resistance in the long crested weir is considerable and therefore it is usually computed by assuming critical depth at the crest. Another possible alternative is computing the discharge by multiplying *Equation 2.5* by a coefficient (*Equation 2.6*). Also, *Equation 2.7* can be used. [5], [21]

$$C = (1 - \delta^*/H_0)^{3/2} \tag{2.6}$$

Where δ^* – Maximum displacement thickness of the boundary layer (*Figure 19*) [m]

$$(1 - C) = 0.069\,(W_c/H_0 - 1 + 2.84\,R_e^{0.25})^{0.8} - R_e^{-0.2} \tag{2.7}$$

Where W_c – Length of the sill [m]
R_e – Reynolds number of flow counted with theoretical velocity v_1 [-]
$v_1 = \sqrt{2g \cdot H_0/3}$ [m/s]

When a fuse gate overturns and the gates on both sides of this fuse gate are in their positions, the discharge over the horizontal sill should be counted with side contractions (*Equation 2.8*). [5], [21]

$$L_{se} = L_s - \left(\frac{n \cdot H_0}{10}\right) \tag{2.8}$$

Where L_{se} – Effective crest width for discharge calculations [m]
n – Number of contractions [-]

2.2.6. Spillway width and sill elevation

When a fuse gate system is designed, some parameters must be given. These parameters are maximum discharge and maximum permissible reservoir level. The maximum discharge corresponds to the inflow design flood either Probable maximum flood (PMF) or Standard project flood (SPF). Then some characteristics of the spillway sill are known. So together with the parameters of the reservoir and flood, the width of the spillway and elevation of the sill are decided. Although, topography and other constraints should not be forgotten. [5], [21]

The last fuse gate in a series would tilt when the reservoir level is on the Maximum water level (MWL) or on a little bit lower water level than that. The first fuse gate would tilt with corresponding water level to a predetermined discharge value such as discharge corresponding to a 100-year flood. Between these two values of discharge, there is tilting of the intermediate fuse gates which tilt by equal steps of discharge. [5], [21]

A final configuration and a fuse gate selection by heights and widths usually require many trials and many model studies. [5]

2.2.7. Example of fuse gate solution

In an existing dam, there is an ungated and 15.75 metres wide spillway which has been designed to discharge of 25 cumecs (100-year flood) at a depth of overflow of 0.80 metres (*Figure 18*). The requirement is the spillway modified to the outflow discharge corresponding to the PMF which is 100 cumecs. Other requirements are the reservoir level which should not be increased above FRL 87.0 metres and increasing of the spillway width is not permissible, too. [5]

So, one possible solution is the existing spillway crest that would be lowered, and the fuse gate system would be installed. Another solution is a new spillway with fuse gate system that can be implemented, and the existing spillway will remain unchanged. [5]

The maximum discharge is 100 cumecs with the spillway width of 15.75 metres. So at first, treating this as a broad crested weir, maximum depth of overflow can be determined by *Equation 2.5*. [5]

$$H_0 = \left[\frac{Q}{1.705 \times L_s}\right]^{2/3} = \left[\frac{100}{1.705 \times 15.75}\right]^{2/3} = 2.4\ m$$

This means the spillway sill elevation should be at $87.0 - 2.4 = 84.6\ metres$. In this elevation, the width of the dam is about 2.0 metres and this is also the length of the sill in the direction of flow. So this gives the ratio $H_0/W_C = 2.4/2.0 = 1.2$. The ratio corresponds to a short crested weir and the *Table 1* suggests determining the flow capacity by interpolating between the sharp crested and broad crested weir. The coefficient C_d has been assumed as 1.75. After interpolating the final horizontal sill was determined at elevation 84.5 metres where the sill length is 2.03 metres and the depth of overflow is 2.5 metres which meets with the requirement of discharging 100 cumecs. [5]

The last fuse gate should tilt with the reservoir level slightly lower than elevation 87.0 metres. Assumed elevation is 86.9 metres or in other words, the depth of overflow is 2.4 metres. Then the parameters of the fuse gate can be determined. For example, the fuse gate model WLH (*Figure 16*) has an upper limit for the tilting range on value 1.6 H. So the suitable height of fuse gate is $2.4/1.6 = 1.5\ metres$. Thus, fuse gate width is $1.5 \times 1.5 = 2.25\ metres$ and the length of the crest is $1.5 \times 4.09 = 6.135\ metres$. The number of gates is $15.75/2.25 = 7$. And the length of the sill (2.03 metres) is sufficient for the fuse gates 1.5 metres wide. (*Figure* 20) [5]

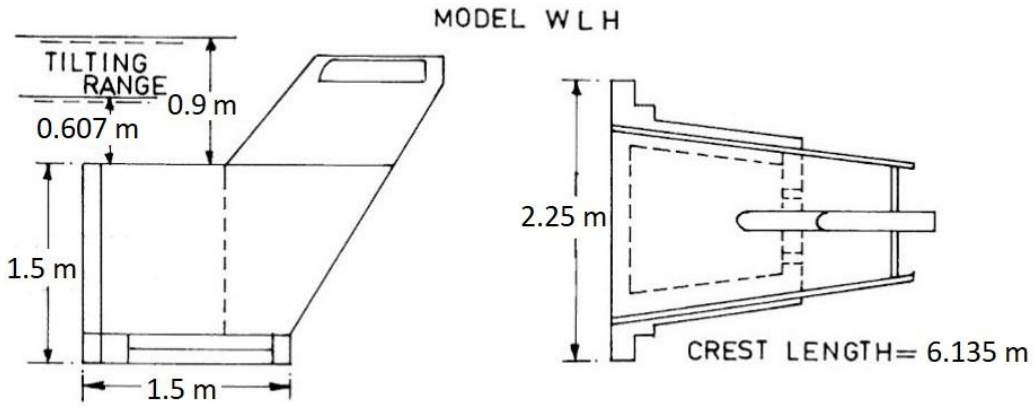


Figure 20 - Fuse gate model WLH with dimensions [5]

Information of reservoir water level and discharges for various fuse gates is needed because of determining the sequence of tilting of each fuse gate. The head-discharge relationship is determined according to *Equation 2.3, 2.4* and *Enclosure 1*. [5]

$$Q = \frac{2}{3}\sqrt{2g} \cdot C_1 \left(\frac{h}{H} - C_2\right)^{C_3} \cdot L_f \cdot h^{3/2}$$

Where $C_1 = 0.32$
$C_2 = 0.001$
$C_3 = -0.292$

$$L_f = 6.135\ m$$

At the elevation 84.5 metres, there is the truncated spillway sill and the discharge over it is given by $Q_s = 1.705\ L_{se}\ H_0^{3/2}$ where L_{se} is the effective width of the spillway segment, vacated by fuse gate accounting for the end contractions. [5]

The system is based on the idea that no two adjacent fuse gates tilt one after another. If the discharge just reaches 25 cumecs, the first fuse gate tilts. This meets with a depth of overflow of 0.607 metres over the crest of the fuse gate and it is the elevation of reservoir water level 86.607 metres. The last fuse gate tilts when the discharge reaches 90 cumecs which is the depth of 0.90 metres (reservoir water level elevation 86.90 metres). The intermediate fuse gates tilt between these two values of discharge. (*Enclosure 2*) [5], [23]

This solution with fuse gates on an existing spillway does not need another suitable site but it usually requires large-scale modifications of the existing spillway. When the existing spillway is gated, the modifications of this spillway can be very expensive. The best advantage, that the fuse gate system offers, is the close regulation. Such as, with dependence upon the flood discharge, only the required number of fuse gates would be tilted. [5]

During operation several problems can occur. There is a danger about the falling fuse gates down the crest and damage the spillway chute. The energy dissipator is also in danger. Usually, some special arrangements are needed to take the fallen fuse gates out of the tail channel. Sometimes, the recovered fuse gates are damaged too much so they cannot be reusable. Occasionally, there is a problem with the energy dissipator which was designed for the original discharge, and now it can be subjected to a much higher discharge. It also requires large-scale modifications. [5]

Even if only one fuse gate tilts, the whole entire storage behind the fuse gates is lost. Installation of a replacement unit can start after the discharge over the sill stops. So because of it, the objective should be to provide the largest possible discharge over the fuse gates without tilting. [5]

2.2.8. Fuse gates in winter conditions

The fuse gate system has been installed in Russia on the Dam of Khorobrovskaya midget power plant by Hydroplus for testing service. Steel fuse gates were used there. During winters 2001/2002, 2002/2003 and 2003/2004, the monitoring was performed. The various parameters were air temperature, ice and snow cover thickness, potential movements and deformation of the fuse gates. (*Figure 21*) [26]

In this region during winter, there are often temperatures less than -30°C. The thickness of ice in the reservoir can be more than 50 centimetres. The buckets, wells and drain holes are frozen through during this period (*Figure 22*). Despite these freezing conditions, no displacements or noticeable deformations of the fuse gates were observed there. As a result of this monitoring, fuse gate system was found as a particularly suitable solution for ice-affected dams. [26]

Figure 21 - Fuse gates in winter conditions (Khorobrovskaya Dam) [26]

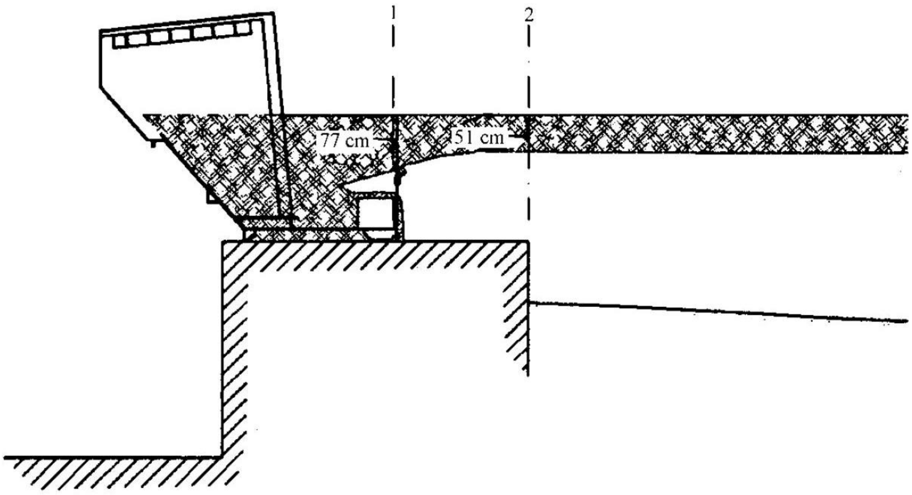


Figure 22 - Thickness of ice during winter [26]

2.2.9. Other fuse gate systems

Apart from the labyrinth crested fuse gates which are used mostly, there are other fuse gate systems which can be helpful against floods.

Straight-crested fuse gates

This fuse gate system is used due to its ability to withstand high headwaters up to 4 times of their own height. They are mostly made of concrete and their shape is optimized to the flow rate. (*Figure 23*) [17]

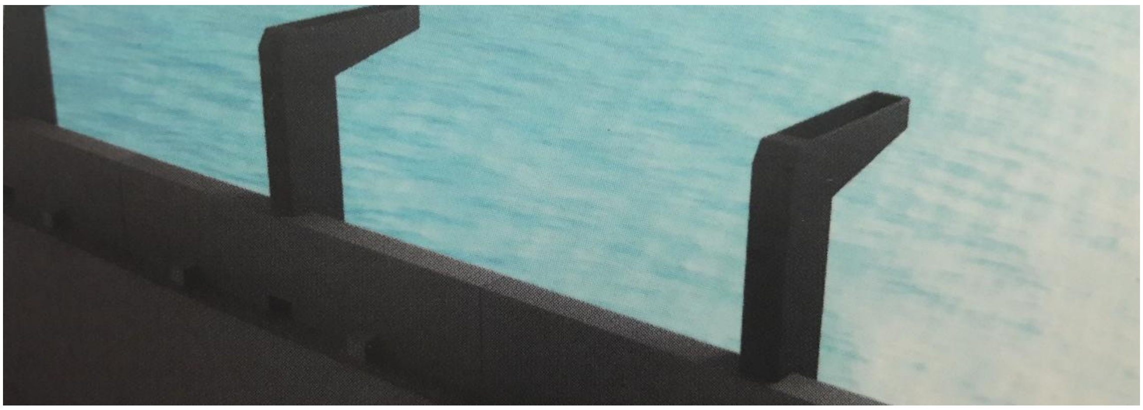


Figure 23 - Straight-crested fuse gates [17]

Folding fuse gates

This fuse gate system is based on the same triggering principles like the classic fuse gate system but it is not dragged by the flood in a tail channel. These fuse gates fold away downwards like a valve and allow to headwater discharge over the crest. After the flood has passed, gates are reinstalled in their initial position manually by the operator. [15], [17]

The gate is inclined in the downstream direction and supported by a set of hinged arms. Smaller floods are discharged over the fuse gate crest. Similarly to the solution with classic fuse gate, in this solution, there is a pressure chamber with the intake well as well and it is equipped with drainage holes. (*Figure 24* and *Figure 25*) [15]

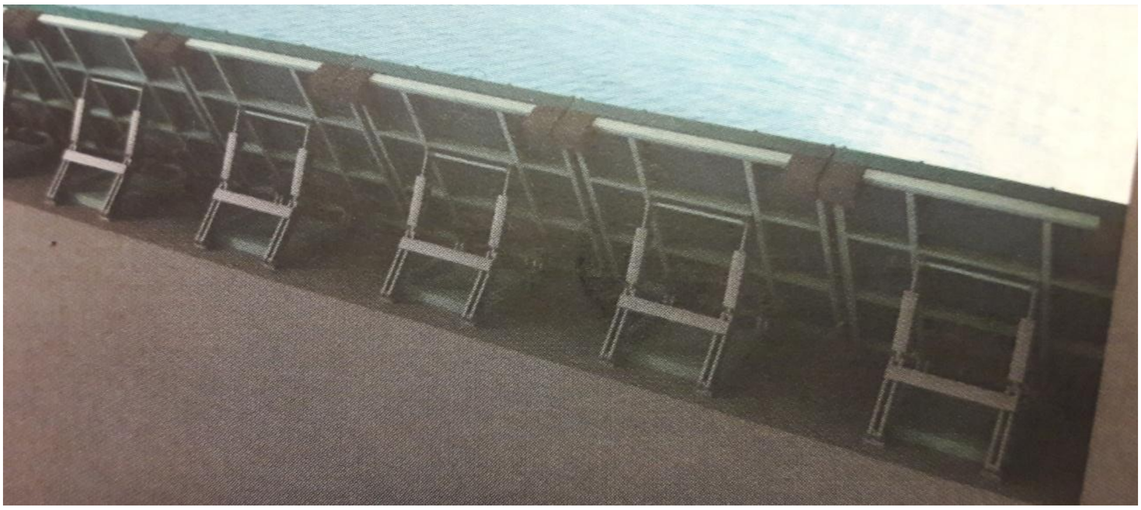


Figure 24 - Folding fuse gates [17]

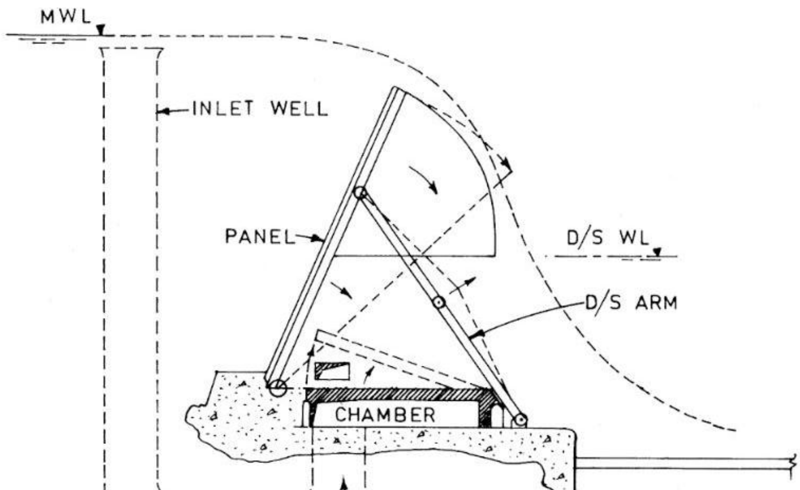


Figure 25 - Folding fuse gate schema [5]

Smart fuse gates

The construction of this fuse gate system is completely different from the others but it is also based on a simple principle. It is independent and not dragged by the flood to the tail channel. It tilts around an axis and after the flood, it comes back to the initial position by itself. The upstream water levels for the tilting and setting back in position can be adjusted during the project realization or later after completion as well. (*Figure 26*) [15]

The main part of this system is hinged panel placed across the flow. In a lower section of the panel, a counterweight is placed which ensures stability for the time, when the gate is in the closed position. When a moderate floods come the water flows over the gate crest. Another situation is during extreme floods when the water level reaches the intake of the tipping conduit. After that, the water flows through the tipping conduit to the inflow box which is in the upper section of the panel. After the box is filled by the water, it acts as a destabilising overweight and tilts the panel to the open position. [15]

When the water level starts to decrease and it drops below the intake to the closing conduit, the drains gradually release water from the inflow box. Then the counterweight takes the gate to the closed initial position. Only this fuse gate manages to capture part of the flood and keep it in the reservoir. [15]

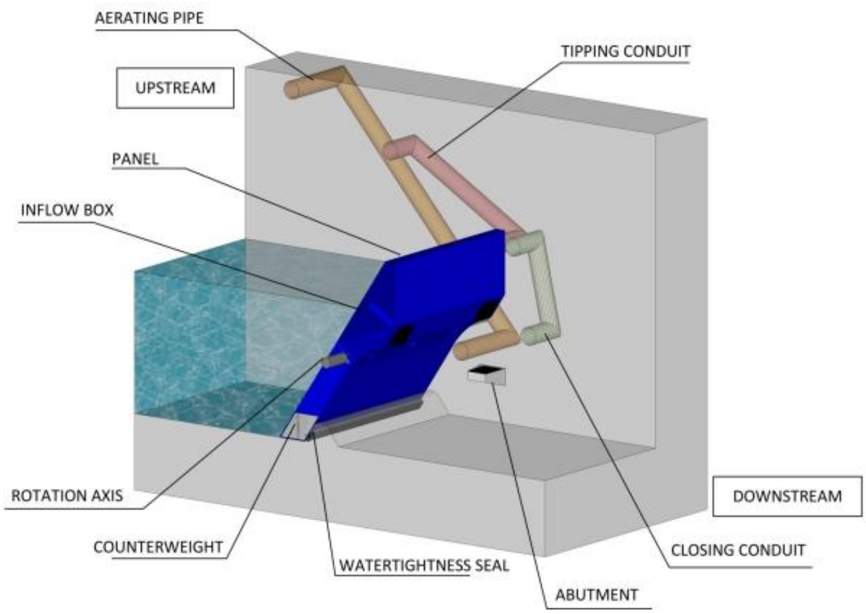


Figure 26 - Smart fuse gate schema [15]

River fuse gates

These gates are straight-crested and they are used in the river flood management. They can be easily integrated into existing levees and they are designed to moderate overtopping. (*Figure 27*) [17]

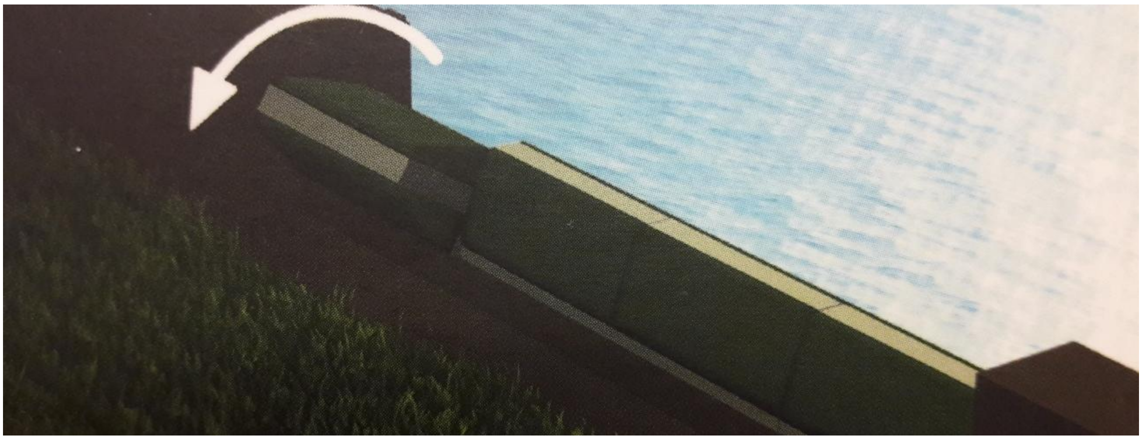


Figure 27 - River fuse gates [17]

3. Dam Ostrov nad Oslavou

The waterworks Ostrov nad Oslavou is a water reservoir in the Czech Republic. It is situated in Jihlava region and in its downstream 1.2 kilometres from the dam, there is a small town called also Ostrov nad Oslavou. In previous years, the dam safety in the Czech Republic was tightened and this dam does not meet the new requirements. Therefore, one possible solution for the dam can be an auxiliary spillway with a system of fuse plug or fuse gate. The reservoir is in the III. category of dam safety as *Table 2* shows. It means that the dam must resist against 1000-year flood without failure.

Water work category	Probability of losses	Assesment points		Probable flood [years]
		Potencial losses	Considered losses	
I.	Very high	extremely high economic losses, environmental losses and social impacts in state scale	losses of human lifes are expected	10 000
II.	High	high economic losses, environmental losses and social impacts in state/region scale	losses of human lifes are expected	10 000
			losses of human lifes are not expected	2 000
III.	Medium	considerable economic losses, environmental losses and social impacts in region scale	losses of human lifes are expected	1 000
			losses of human lifes are not expected	200
IV.	Low	low economic losses, environmental losses and social impacts in local scale	losses of human lifes are expected	200
			losses of human lifes are not expected	100
		low economic losses, only owner of the water work losses, another losses are not significant	losses of human lifes are not expected	20

Table 2 - Categories of dam safety in the Czech Republic [27]

The water reservoir is placed on a stream which is called Bohdanovský Potok. In distance of 1.2 kilometres downstream of the dam, there is the confluence with the Oslava river. The reservoir is on the west of the town and it is approximately 900 metres distanced from its edge.

3.1. Purpose of the reservoir

The water reservoir Ostrov nad Oslavou is a multipurpose dam and its purposes are [1]:

- discharge improvements in Bohdanovský Potok and Oslava river,
- flood control,
- extensive fish farming,
- sport fishing
- water accumulation for fire-fighting and other purposes,
- recreation. [1]

3.2. Technical description

This chapter contains a technical description of all important equipment of the dam Ostrov nad Oslavou. These are a dam body, drainage reservoir channel, multipurpose flow control structure, tail channel, reservoir and equipment for surveillance and measurement.

3.2.1. Dam body

It is zoned earthfill dam with a middle impervious core. The dam is straight in a plan view, and in a cross-section, it has a trapezium shape. The total volume of the dam body is 23 200 cubic metres. Shoulder of this dam contains gravel with admixtures of loam-sandy soil and sand-gravel soil. The impervious core is from clay, sandy loams and from silty soil. [1]

The gradient of the upstream slope is 1:3.2 and upstream slope is protected by a riprap of thickness from 0.09 to 0.50 metres. Under the riprap, there is a layer of a crushed aggregate of a fraction between 32 and 63 millimetres. Under the mentioned layers, there is a geotextile in all length of the dam. [1]

The gradient of the downstream slope is 1:2 and for a downstream slope revetment, grass sowing is used. In the downstream toe of the dam, the drainage blanket with drainage pipeline DN 150 is placed. [1]

The crest of the dam is 5 metres wide and 135 metres long. It can be driven but only for service purpose. There are laid concrete panels on it which are 3 metres wide and they are placed in the length of 125 metres. On the crest of the dam on the upstream side, a railing is placed which contains concrete columns and wooden stakes. [1]

Figure 28 - The crest of the dam and the upstream slope with the multipurpose control structure [28]

Parameters of the dam body (Baltic sea level) [1]:

Elevation of the crest of the dam	524.00 m a.s.l. (minimal 523.92 m a.s.l.)
Elevation of the lowest point of the valley	515.00 m a.s.l.
Elevation of the top of the impervious core	523.00 m a.s.l. (minimal 522.56 m a.s.l.)
Length of the crest of the dam	135.00 m
Width of the crest of the dam	5.00 m
Maximal height of the dam body	9.00 m
Gradient of the upstream slope	1:3.2
Gradient of the downstream slope	1:2
Total volume of the dam body	23 200 m^3

3.2.2. Drainage reservoir channel

The first part of the drainage reservoir channel is unpaved and it is approximately 38 metres long. The second part of the channel is from reinforced concrete and it is designed as a functional object for fish harvesting which is 15 metres long (*Figure 29*). [1]

Figure 29 - Drainage reservoir channel [29]

3.2.3. Multipurpose flow control structure

Purposes of the multipurpose flow control structure are:

- discharging floods over the spillway,
- discharging guaranteed minimum flow,
- emptying of the reservoir by bottom outlets. [1]

The structure is placed near to dam abutment into the right bank. The structure is made from reinforced concrete and it has 2 floors. It contains several objects. [1]

Operation shaft

Operation shaft is 8.1 metres high and its ground plan dimensions are 2x3 metres. When there are normal conditions, water should not be inside the shaft. At the upstream side, there is the intake part of the control structure which includes guiding beams for stop log barriers. The width of the stop log barriers is 2.4 metres and their total height is 7.5 metres. The first 2 pieces of the stop log barriers from the bottom are screens. There are other guiding beams for temporary plate gates if the bottom outlets would need an inspection or repair. The shaft is covered with steel trapdoor. On the top of the control structure (elevation

524.00 metres a.s.l.), control of valves of the bottom outlets is placed. The access to the shaft is provided by a steel bridge with railing. [1]

Bottom outlets

Intake to the bottom outlets is situated at the upstream side of the operation shaft.

Axis of the right bottom outlet with DN 400 is on elevation 516.50 metres a.s.l. The pipeline of this outlet is connected with a tailrace. The bottom outlet has two gate valves and they are controlled from the top of the multipurpose flow control structure. [1]

Axis of the left bottom outlet with DN 300 is on elevation 517.00 metres a.s.l. The pipeline of this outlet is lined through the tailrace and it ends in a stilling basin. The bottom outlet has one gate valve which is controlled from the top of the multipurpose flow control structure and one side gate on the end of the pipeline. [1]

Axis of the minimum guaranteed outflow outlet with DN 150 is on the elevation 516.20 metres a.s.l. The pipeline ends in the tailrace and it has two gate valves, one of these is controlled from the top of the multipurpose flow control structure and the second one is controlled from the bottom of the operation shaft. [1]

Figure 30 - Bottom outlets and the upstream side of the operation shaft [29]

There are other two bottom outlets which were used for discharging water during dam construction. These are located in the foundations of the multipurpose flow control structure, but they are used only in extraordinary situations. One outlet is from pipeline DN 400 and the other from pipeline DN 200. Both axes are on the elevation 515.30 metres a.s.l. Intake to the pipelines is covered by a screen. Each outlet has one gate valve which is controlled from the top of the multipurpose flow control structure. [1]

Spillway

The spillway was reconstructed in 2015. After the reconstruction, it has 2 ungated side spillway crests on the elevation 522.40 metres a.s.l. 9.75 metres long and 2 ungated side spillway crests on the elevation 522.20 metres a.s.l. 4.25 metres long. The spillway crests are curved (*Figure 31*). Then, the water fall in the apron which is 14.00 metres long and 2.40 metres wide. After that, water flows into the spillway chute tunnel with the length of 25.00 metres, the width of 2.40 metres and the height of 4.15 metres. The slope of the chute is 3.50 percents. The chute edge which is connected to the stilling basin is on the elevation 517.37 metres a.s.l. (*Figure 32*). [1]

Figure 31 - Curved spillway crests [30]

Tailrace

Water flows from bottom outlets through the tailrace. The tailrace has 39.00 metres in length, 2.40 metres in width and 2.20 metres in height. The slope of the tailrace is also 3.50 percents like the slope of the spillway chute. The edge of the tailrace is connected with the stilling basin on the elevation 514.41 metres a.s.l. The entrance to the tailrace is possible by steel ladders which are on both walls of the stilling basin. (*Figure 32*) [1]

Stilling basin

The stilling basin is made from reinforced concrete and it has a trapezoidal shape in a cross-section (*Figure 32*). The inclination of the walls of the stilling basin is 3:1. Length of the stilling basin is 26.00 metres and its depth is 2.50 metres. The stilling basin ends by a concrete threshold with measuring section. [1]

Figure 32 - The spillway chute (up), the tailrace (down) and a part of the stilling basin [30]

3.2.4. Tail channel

The stilling basin is connected with the tail channel which has also trapezoidal shape with the gradient of the slopes 1:1.5. In the length of 15.00 metres, the tail channel's banks are lined by stone paving. Then the stream continues in the unlined channel. (*Figure 33*) [1]

Figure 33 - The tail channel [29]

3.2.5. Reservoir

The reservoir is divided into three storages which are named as inactive storage, water supply storage and flood storage. When the water is on elevation 522.20 metres a.s.l. which is maximal water supply level, the area of the water surface is 15.70 ha, water storage is 568 000 cubic metres and the maximal depth is 7.20 metres. (*Figure 34*) [1]

Storages of the reservoir [1]:

Inactive storage	$V_S = 80\ 000\ m^3$
Water supply storage	$V_Z = 488\ 000\ m^3$
Flood storage and surcharge	$V_{RO} = 144\ 000\ m^3$
Reservoir capacity	$V_C = 712\ 000\ m^3$

Water levels and areas of the reservoir (Baltic sea level) [1]:

The inactive storage water level	$M_S = 517.80\ m\ a.s.l.$	$A_S = 64\ 000\ m^2$
Water supply storage level	$M_Z = 522.20\ m\ a.s.l.$	$A_Z = 157\ 000\ m^2$
Maximum water level	$M_{MAX} = 523.00\ m\ a.s.l.$	$A_{MAX} = 175\ 000\ m^2$

Figure 34 - Water reservoir Ostrov nad Oslavou [31]

3.2.6. Equipment for surveillance and measurement

A waterworks operator ensures measuring and surveillance of states on the dam because of its safety. If something is not usual with the waterworks, the operator has to assess the situation and decide what should be done.

The stage gauge is placed on the wall of the multipurpose flow control structure for determining the current water level in the reservoir. At the outflow of the reservoir at the end of the stilling basin, there is installed measuring section of a trapezoidal shape for measuring discharge. [1]

Seepage measurement is provided as a direct measurement of the quantity of water which discharges from the dam drainage system. The seepage is measured periodically from the two conduits which are ended in the stilling basin from both parts of the dam, left and right. For surveillance groundwater level at the downstream apron, there are two test boreholes. [1]

A dam deformation, deformation of the stilling basin, deformation of the tailrace or deformation of the multipurpose flow control structure is monitored by very precise levelling which consists of measuring levelling points. [1]

3.3. Hydrological description

In this chapter, important flows and data which can appear at the profile of the dam are mentioned. There are minimum guarantee flow, harmless flow, flood routing and hydrological data.

3.3.1. Minimum guaranteed outflow

Minimum guaranteed outflow at the downstream apron is 30 litres per second. This discharge is ensured by the minimum guaranteed outflow bottom outlet which is made from conduit DN 150. [1]

3.3.2. Harmless discharge

The harmless discharge in the village Ostrov nad Oslavou at downstream of the dam is determined to value of discharge 1.10 cumecs. When the discharge is bigger, the channel does not have the capacity for it, and water starts to flow into a floodplain area. Maximum capacity of the three bottom outlets which are used in common situations (DN 400 + DN 300 + DN 150) is 1.20 cumecs. The paved part of the channel in the downstream apron area has the capacity for 100-year flood which is 26.00 cumecs, but it is only 15 metres long. [1]

3.3.3. Floods routing

Floods routing study was implemented in the year 2015. There were performed design flood hydrographs FH 100, FH 10 and FH 1. Design flood hydrographs FH 100 and FH 10 are not influenced by the reservoir, there is no flood routing because of the volume of water which is multiple bigger than the storage in the reservoir. Only design flood hydrograph FH 1 is influenced whose discharge is decreased from 3.50 cumecs to 2.85 cumecs. This study was already calculated with the reconstructed spillway. [1]

3.3.4. Hydrological data

The hydrological data were distributed by the Czech Hydrometeorological Institute in the years 2012 and 2015. [1]

Hydrological identification number:	4 – 16 – 02 – 010
Catchment area:	48.78 km2
Average precipitation per a year:	688 mm
Average discharge:	0.33 cumecs [1]

m	[days]	30	90	180	270	355	364
m-day flow Q_m	[l/s]	750	320	160	90	30	9

Table 3 - m-day flow data [1]

N	[years]	1	2	5	10	20	50	100	200	500	1000
N-year flow Q_N	[m^3/s]	3.5	3.8	5	6.8	9.9	17.1	26	39.3	66.3	96.7
Design flood hydrograph FH N	[mil m^3]	0.58	0.87	1.35	1.79	2.3	3.08	3.76	4.52	5.65	6.62

Table 4 - N-year flow data and design flood hydrographs [1]

3.4. Manipulation during flood

The storage with which is manipulated during a flood is the flood storage.

Minimum level of flood storage: 522.20 m a.s.l.
Maximum water level: 523.00 m a.s.l.
Volume of flood storage: 144 000 m^3
Area of the water surface during MWL: 17.50 ha [1]

Water level above the elevation 522.20 metres a.s.l. is allowed only during floods and when the water surface of reservoir needs to be cleaned from suspended load and from water bloom. If the water level rises above the elevation 522.20 metres a.s.l., water flows over the lower part of the spillway. [1]

When a flood flows through the reservoir, the supply storage is filled first. When the flow raises, the bottom outlets are opened gradually until the harmless discharge which is 1.10 cumecs. The other two bottom outlets which were used for discharging water during dam construction, are used only at crisis flood events. If the inflow to the reservoir is bigger than outflow, water starts to spill over the lower crest of the spillway and the bottom outlets are fluently closed. Water is discharged only over the spillway. Then water rising in the flood storage is uncontrolled. When water achieves maximum water level elevation 523.00 metres a.s.l., the discharge over the spillway is 27 cumecs. [1]

The water level in the reservoir can be lowered before spring melting if the water is on the elevation 521.80 metres a.s.l. or higher. The drawdown of the reservoir is possible also during persistent rain period when the bigger flows are expected. The manipulation is controlled by the dispatching of Povodí Moravy, s.p. The drawdown of the reservoir is commonly done by the bottom outlets with conduits DN 300 and DN 400 which can provide discharge of 1.07 cumecs. The maximal drawdown is allowed to the water elevation 517.80 metres a.s.l. [1]

3.4.1. Levels of the flood activity

In the Czech Republic, there are three levels of the flood activity. These are watchfulness, emergency and threat.

I. Level of the flood activity – Watchfulness

This flood level is not specified at this waterworks. [1]

II. Level of the flood activity – Emergency

This flood level is announced when the discharge achieves 1.20 cumecs. This is just the flow which is discharged from 3 used bottom outlets (DN 150, DN 300, DN 400) when the water does not spill over the spillway crest. The same flow spills over the spillway crest when the water level is on the elevation 522.40 metres a.s.l., that is 20 centimetres above the lower spillway crest, and the all bottom outlets are closed. [1]

Water levels are checked every hour or the dispatching of Povodí Moravy, s.p. can order another check interval. [1]

III. Level of the flood activity – Threat

This flood level is announced when the water level achieves the elevation 522.55 metres a.s.l. The water level is 35 centimetres above the lower spillway crest. Over the spillway is discharged approximately 4 cumecs. [1]

Water levels are checked four times per hour or the dispatching of Povodí Moravy, s.p. can order another check interval. [1]

4. Design of the auxiliary spillway

The dam Ostrov nad Oslavou according to Czech requirements need to withstand 1000-year flood without failure, but nowadays, it is designed to withstand only 100-year flood. In this chapter, one possible solution how to improve the safety of this dam by an auxiliary spillway is described.

4.1. Restrictive requirements

There are several requirements which have to be abided. First of these requirements is the maximum supply water level which should be preserved, so the minimal level of the auxiliary spillway crest can be on the elevation 522.20 metres a.s.l. Nevertheless, as the next requirement, the toe of the fuse plug should not be often under the water level. So, it was decided the fuse plug toe should be above the water level of the 5-year flow.

The other requirement is that in the downstream of the auxiliary spillway, there will be no chute, no stilling basin and no tail channel. The auxiliary spillway has to be designed at dam abutment into a bank, where is not any threat of the undercutting of the dam body. The suitable location for the spillway is the left abutment into the bank.

4.2. Choice of the type of the auxiliary spillway

In the Czech Republic, fuse gate or fuse plug systems have never been used. In the year 2015, study for safety improvement of the dam Ostrov nad Oslavou was implemented, but none of these systems were included [2].

Advantages and disadvantages of these two types of spillway systems are quite similar. But in the chapter *2.2.2*, there was mentioned that fuse gates are distributed with the height from 1.5 metres to 6.5 metres and this dam would need the smaller size of fuse gates. The fuse gates which would be on turnkey order would be more expensive than fuse plug which can be constructed in any dimensions. The price of the solution is important and because of it, the fuse plug system was chosen.

4.3. Design

In this chapter the whole design of the auxiliary spillway gated with fuse plug system is introduced. For design it is needed to know the capacity of the original spillway, then the dimensions of the auxiliary spillway crest can be designed. It is supposed that some adjustments on the dam will be needed and finally, the fuse plug can be designed.

4.3.1. Original spillway capacity

At first, before the auxiliary spillway will be designed, it is needed to have a chart with original spillway capacity, because both spillways together, the original and the auxiliary, have to be able to discharge 1000-year flood. The 1000-year flood has maximum discharge 96.7 cumecs.

The first part of the capacity was counted by *Equation 4.1* for spill over an unsubmerged ogee. There are two sections with different geometry, they were counted separately and then they were summarized. The length of the spillway crest was reduced by the side contractions (*Equation 4.2*). Coefficient m was counted by equations for curved crest discovered by Kramer.

For calculation of the coefficient m of the circularly curved crest, the *Equation 4.3* with a radius r of the curving was used. For ellipse curved crest, the same equation was used but for counting the radius r, the *Equation 4.4* was used. [32]

$$Q = m \cdot b_0 \cdot \sqrt{2g} \cdot h_0^{3/2} \tag{4.1}$$

Where Q – Discharge over the spillway crest [m^3/s]
m – Coefficient of the shape of the spillway crest [-]
b_0 – The reduced length of the spillway crest [m]
h_0 – Water potential above the spillway crest [m]

$$b_0 = b - 0.1 \cdot \Sigma\xi \cdot h_0 \tag{4.2}$$

Where b – The real length of the spillway crest [m]
ξ – Loss coefficient of the side contraction [-]

$$m = \frac{2}{3} \cdot \left[1.02 - \frac{1.015}{\frac{h}{r} + 2.08} + \left[0.04 \cdot \left(\frac{h}{r} + 0.19\right)^2 + 0.0223\right] \cdot \frac{r}{s}\right] \tag{4.3}$$

Where h – Water level above the spillway crest [m]
r – Radius of the curving of the spillway crest [m]
s – Height of the spillway from the bottom of the reservoir [m]

$$r = b \cdot \left(\frac{4.57}{2 \cdot \frac{a}{b} + 1} + \frac{a}{20 \cdot b} - 0.573\right) \tag{4.4}$$

Where a – Length of the semi-major axis of the ellipse [m]
b – Length of the semi-minor axis of the ellipse [m]

The second part of the capacity was counted in two different ways. One of these ways is counting by *Equation 4.5* for spill over a submerged ogee [32]. Firstly, it was detected at which discharge the ogee starts to be submerged. It is supposed that at the edge of the spillway apron a critical depth is (*Equation 4.6*) [33]. From the critical depth, the parameter G can be counted by *Equation 4.7* and then, the maximum depth in the spillway apron can be found by the Komora's diagram (*Enclosure 3*) [12]. Finally, the water potential of the spillway crest can be counted (*Equation 4.5*) by the iteration method using Denver diagram (*Enclosure 4*). Denver diagram provides the coefficient of submerging.

$$Q = \sigma_z \cdot m \cdot b_0 \cdot \sqrt{2g} \cdot h_0^{3/2} \tag{4.5}$$

Where σ_z – Coefficient of the submerging [-]

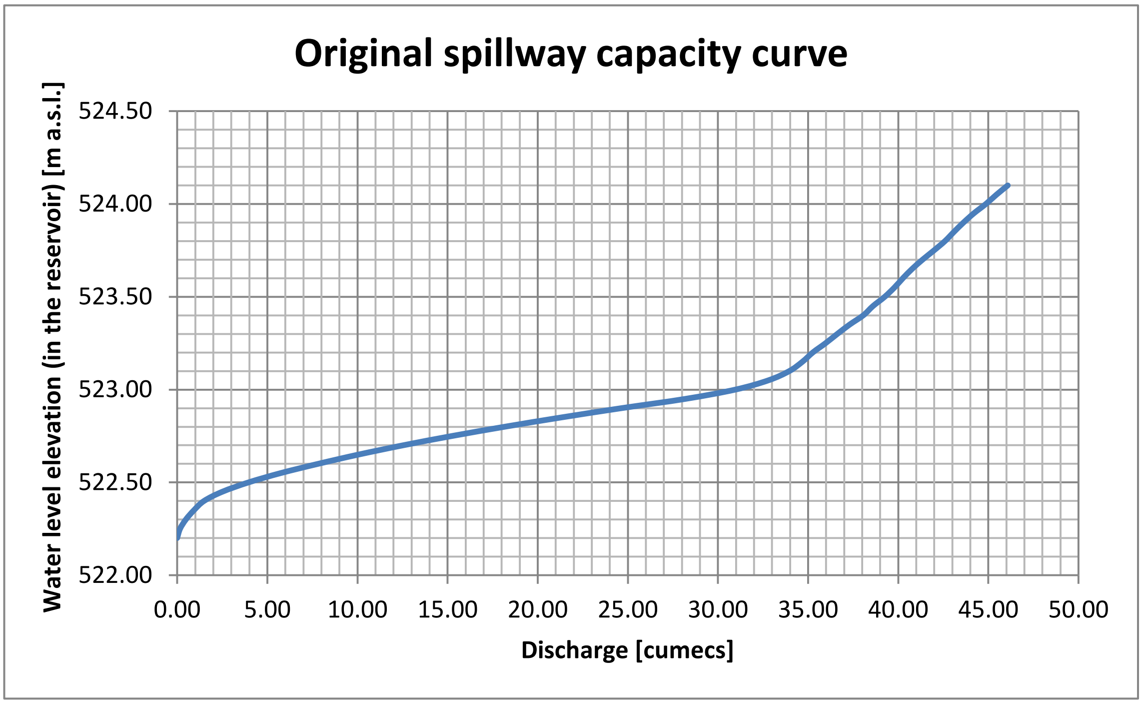
$$h_c = \sqrt[3]{\frac{\alpha \cdot Q^2}{g \cdot b^2}} \tag{4.6}$$

Where h_c – Critical depth [m]

α – Coriolis number [-]
b – Width of the rectangular channel [m]

$$G = \frac{i_s \cdot L_s}{h_c} \tag{4.7}$$

Where G – Parameter of the Komora's diagram [-]
i_s – Slope of the spillway apron [-]
L_s – Length of the spillway apron [m]
h_c – Critical depth at the edge of the spillway apron (connected with the chute) [m]



Graph 1 - Existing spillway capacity curve (second part counted as a submerged ogee)

Water level	[m a.s.l.]	522.20	522.40	522.60	522.80	523.00	523.20	523.40	523.60	523.80	524.00
Discharge	[cumecs]	0	1.49	7.83	18.19	30.98	35.27	38.07	40.27	42.60	44.87

Table 5 - Original spillway capacity (second part counted as a submerged ogee)

The second way how to calculate the second part of the spillway capacity is the Bernoulli equation for culverts (*Equation 4.8*, *Figure 35*) [34], because this spillway tunnel apron is very similar to a square culvert. This solution served only as a verification if the previous way is correct and can be used for the next calculations.

$$i_0 \cdot L_{AB} + y_h + \frac{\alpha \cdot v_h^2}{2g} = y_c + \frac{\alpha \cdot v_c^2}{2g} + \xi \cdot \frac{v_c^2}{2g} = y_c + \frac{Q^2}{2g \cdot \varphi^2 \cdot S_c^2} \tag{4.8}$$

Where i_0 – Bed slope between sections A and B [-]
L_{AB} – Distance between sections A and B [m], $L_{AB} = 0$ m
y_h – Water level at the upstream of the culvert [m]
$\frac{\alpha \cdot v_h^2}{2g}$ – Velocity head at the upstream of the culvert [m], $\frac{\alpha . v_h^2}{2g} = 0\ m$

y_c – Water level at the place where the decreased depth occurs [m],

$$y_c = 0{,}62 \cdot h$$

h – Height of the culvert

$\frac{\alpha . v_c^2}{2g}$ – Velocity head at the place where the decreased depth occurs [m]

$\xi . \frac{v_c^2}{2g}$ – Loss of the energy by the inlet to the culvert [m]

g – Gravitational acceleration [m/s^2]

φ – Velocity coefficient [-], $\varphi = \frac{1}{\sqrt{\alpha+\xi}}$, $\alpha = 1, \xi = 0.13$ [35]

S_c – Cross-section of a stream at the place where the decreased depth occurs [m^2], $S_c = 0{,}62 \cdot S_D$

S_D – Cross-section of the area of a culvert [m^2]

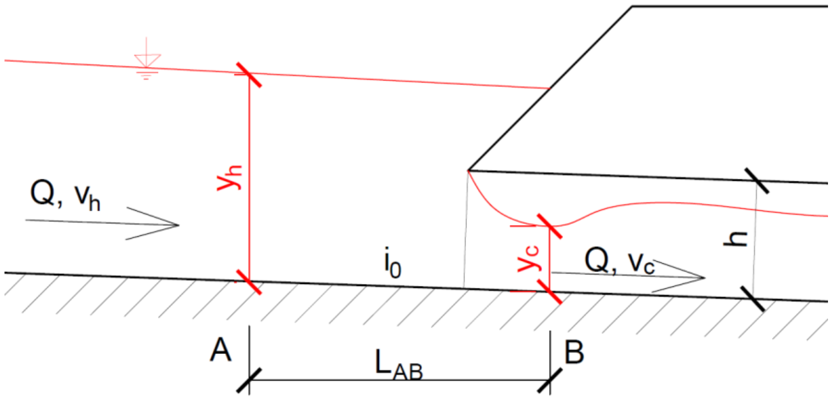
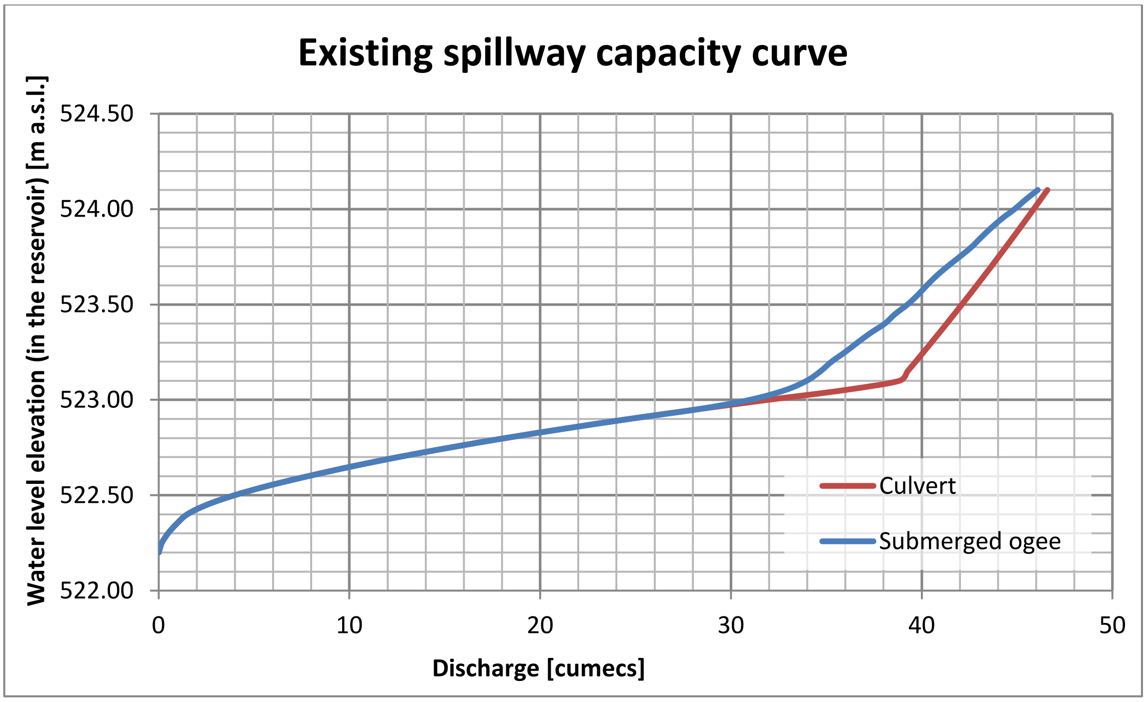


Figure 35 - Application of the Bernoulli's equation at the inlet of the culvert



Graph 2 - Existing spillway capacity curve (second part counted as a culvert, too)

Water level	[m a.s.l.]	522.20	522.40	522.60	522.80	523.00	523.20	523.40	523.60	523.80	524.00
Discharge	[cumecs]	0	1.49	7.83	18.19	31.94	39.67	41.31	42.88	44.40	45.86

Table 6 - Original spillway capacity (second part counted as a culvert)

The graphs (*Graph 1* and *Graph 2*) show that the values which are important for this design are very similar. The only difference is that culvert capacity has a little bit steeper curve than the submerged ogee and their points of congestion are different. According to safety reasons, the submerged ogee capacity was selected as the correct one. It gives lower discharges than the culvert capacity.

4.3.2. Design of the auxiliary spillway crest and channel

The auxiliary spillway crest has to be designed to meet with the restrictive requirements. The minimal level of the spillway crest can be on the elevation 522.20 metres a.s.l., but it should be above the water level of the 5-year flow. The 5-year flow is 5.0 cumecs and this flow can be discharged by the original spillway with the water level on the elevation 522.55 metres a.s.l.

The spillway crest can be placed only at the left abutment into the bank where the suitable location has only a little space.

The 1000-year flow is 96.7 cumecs. The current maximum water level of the dam is 523.00 metres a.s.l. During maximum water level the original spillway discharges 30.98 cumecs. This means that the auxiliary spillway should be able to discharge 65.72 cumecs. However, this discharge would need spillway crest more than 150 metres long and this is impossible to implement this huge spillway at this quite small dam.

One possible solution is that the maximum water level will be raised on a higher elevation. This solution was also included in the study for safety improvement of the dam Ostrov nad Oslavou [2], but with another solution of an auxiliary spillway, not fuse plug or fuse gate system.

If the maximum water level would be raised on the elevation 524.00 metres a.s.l., discharge over the original spillway would be 44.87 cumecs and then, the auxiliary spillway for 51.83 cumecs is needed. For this discharge, the spillway crest about 20 metres long is enough.

Spillway geometry

The spillway crest, it means the part where the fuse plug will be placed, will be made from reinforced concrete. The elevation of the spillway crest will be 522.55 metres a.s.l. The bottom of the crest will be 10 metres long in the direction of flow, its width will be 19.0 metres and its thickness will be 0.3 metres. Its edge will be ended by the cut off wall, it will be 1 meter high and its function will be prevention to retrogressive erosion. The similar wall as the wall at its edge will be at its beginning.

Sidewalls of the spillway crest will be done in the slope 10:1 on their both sides. So the width of the sidewalls will be at its top 0.3 metres, at the spillway crest level 0.59 metres and at its toe 0.69 metres. The slope will be there because of the joint of the impervious core of the original dam to the concrete constructions, and joint of the fuse plug to the concrete constructions.

The spillway crest will be connected with the spillway channel which leads the discharge around the dam until the place where the discharge cannot endanger the dam. The channel consists of the right turn with the radius 25.50 metres and the short straight. The length of the axis of the channel is approximately 63 metres and the width of the channel is 19 metres. The channel has 1% slope. As the right bank of the channel, the same sidewall from the spillway crest with slope 10:1 continues. But the right bank transfer from the sidewall to the paved bank with a riprap and with slope 1:1.

At the place where the channel ends, the slope of the terrain raises. It means that there will be a critical depth. Then, water will continue to the valley which is under water during bigger floods, without leading by any channel.

Spillway capacity

The spillway was designed for the maximum flow which is almost 52 cumecs. It was verified that the maximum flow can be discharged without failure by this spillway. For the spillway crest, equations for broad crested weirs were used (*Equations 4.9 – 4.12; Figure 36*) [32].

$$h_1 = h \cdot \varepsilon_1 \tag{4.9}$$

Where h_1 – The first decreased depth on the spillway crest [m]
h – Height of the water in the reservoir above the spillway crest [m]
ε_1 – Coefficient of the decrease for h_1 [-], $\varepsilon_1 = 0.6$

$$h_2 = h \cdot \varepsilon_2 \tag{4.10}$$

Where h_2 – The second depth on the spillway crest (at its end) [m]
ε_2 – Coefficient of the decrease for h_2 [-], $\varepsilon_2 = 0.73$

If the h_2 is bigger than the depth downstream of the spillway crest h_σ, the discharge can be calculated by *Equation 4.11* for an unaffected flow. In other cases, if the h_2 is smaller than the h_σ, the *Equation 4.12* must be used. The depth h_σ is known by the next step where the depth in the spillway channel is calculated by equations for steady flow (*Equations 4.13 – 4.18; Figure 37*) [33].

$$Q = \varphi \cdot S_1 \cdot \sqrt{2g \cdot (h_0 - h_1)} \tag{4.11}$$

Where φ – Discharge coefficient [-], $\varphi = 0.95$
S_1 – Flow cross-section area at the place with depth h_1 [m^2]
h_0 – Water potential above the spillway crest [m]

$$Q = \varphi \cdot S_\sigma \cdot \sqrt{2g \cdot (h_0 - h_\sigma)} \tag{4.12}$$

Where S_σ – Flow cross-section area at the place with depth h_σ [m^2]
h_σ – Depth downstream of the spillway crest which negatively influences the discharge[m]

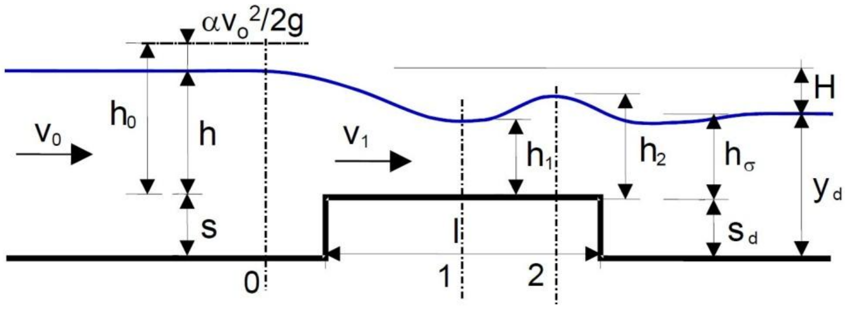


Figure 36 - Schema of a broad crested weir

$$R = \frac{S}{O} \tag{4.13}$$

Where R – Hydraulic radius [m]
S – Flow area [m^2]
O – Wetted perimeter [m]

$$C = \frac{1}{n} \cdot R^{1/6} \tag{4.14}$$

Where C – Manning coefficient [m$^{1/2}$/s]
n – Manning roughness [s/m$^{1/3}$]

$$v = C \cdot \sqrt{R \cdot i_E} \tag{4.15}$$

Where v – Flow velocity [m/s]
i_E – Energy gradient [-]

$$Q = S \cdot v \tag{4.16}$$

$$Fr = \frac{v}{\sqrt{g \cdot \frac{S}{B}}} \tag{4.17}$$

Where Fr – *Froude* number [-]

$$i_0 \cdot \Delta L + y_1 + \frac{\alpha \,.\, v_1^2}{2g} = y_2 + \frac{\alpha \,.\, v_2^2}{2g} + i_E \cdot \Delta L \tag{4.18}$$

Where i_0 – Bed slope between sections 1 and 2 [-]
ΔL – Distance between sections 1 and 2 [m]
y_1 – Water level at the upstream section [m]
$\frac{\alpha \cdot v_1^2}{2g}$ – Velocity head at the upstream section [m]
y_2 – Water level at the downstream section [m]
$\frac{\alpha \,.\, v_2^2}{2g}$ – Velocity head at the downstream section [m]
i_E –Energy gradient between sections 1 and 2 [-]

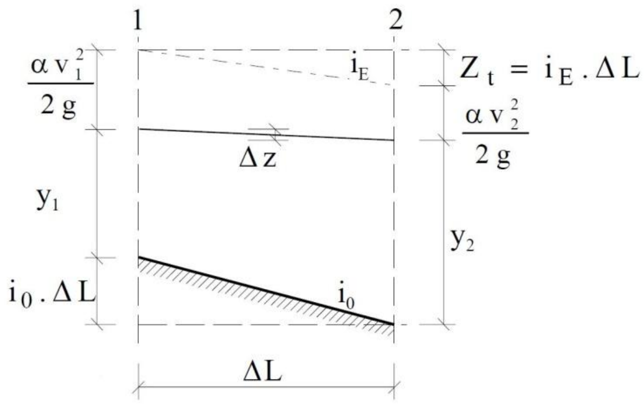
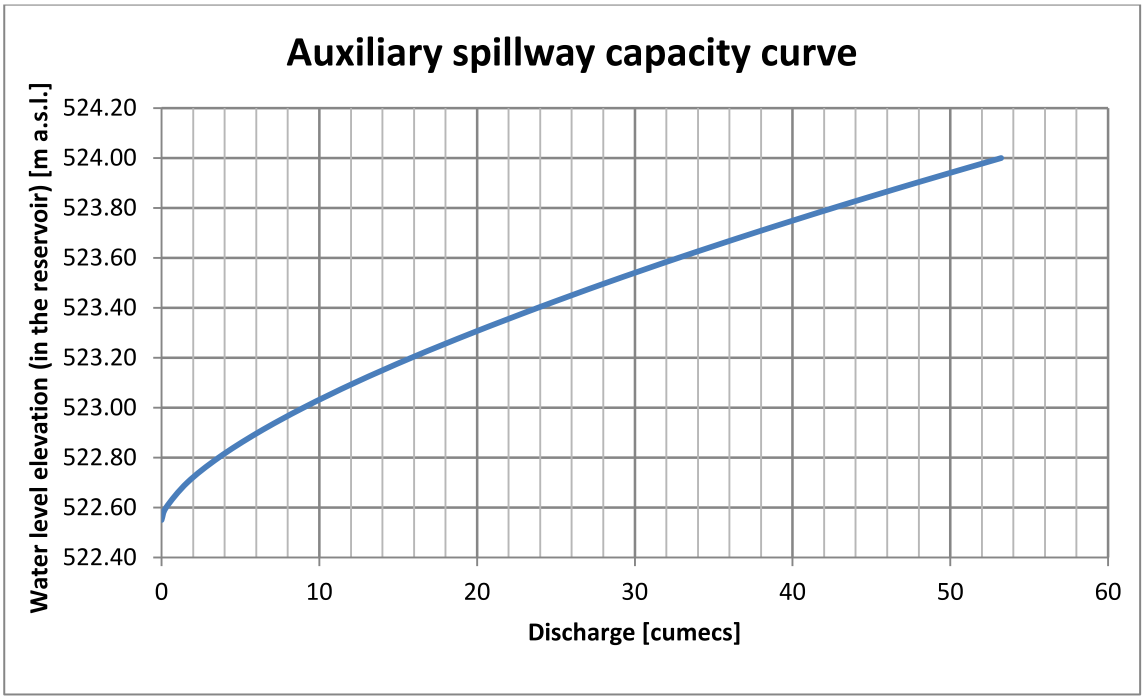


Figure 37 - Schema of the solution of steady flow by Bernoulli's equation

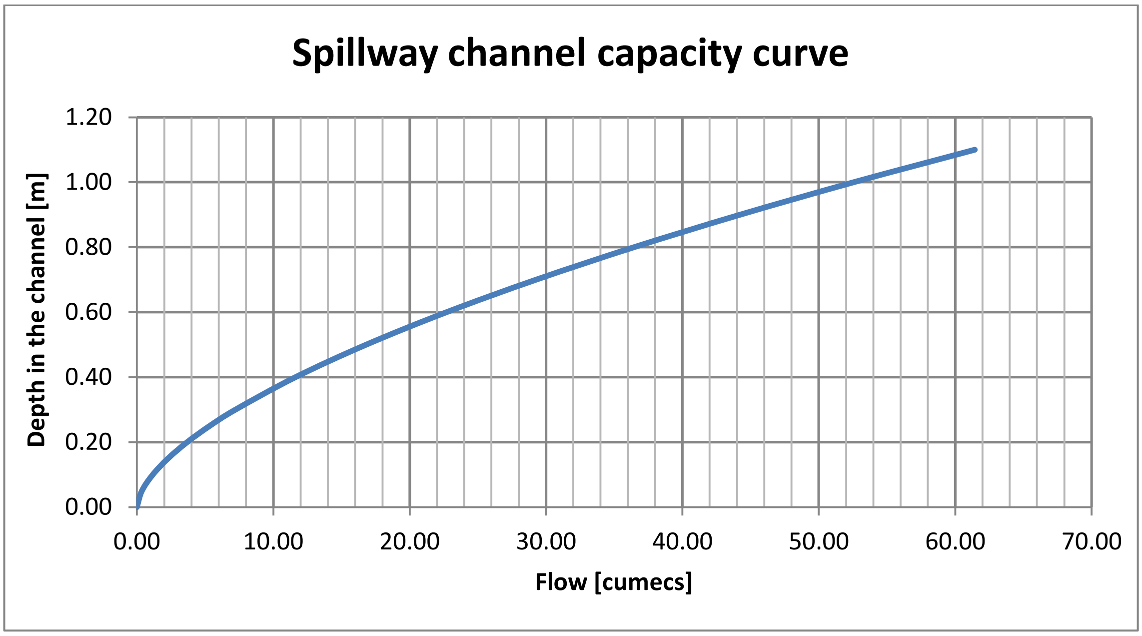
Results are shown in charts and tables. The relationship between the water level in the reservoir and discharge over the spillway crest is shown in *Graph 3* and *Table 7*. The spillway channel capacity is in *Graph 4* and *Table 8*. Finally, water surface levels were counted from the reservoir until the end of the spillway channel during the several discharges over the spillway crest. It is shown in *Graph 5* and *Table 9*.



Graph 3 - Auxiliary spillway capacity curve

Water level	[m a.s.l.]	522.55	522.60	522.70	522.80	523.00	523.20	523.40	523.60	523.80	524.00
Discharge	[cumecs]	0	0.29	1.63	3.62	8.99	15.83	23.84	32.75	42.57	53.22

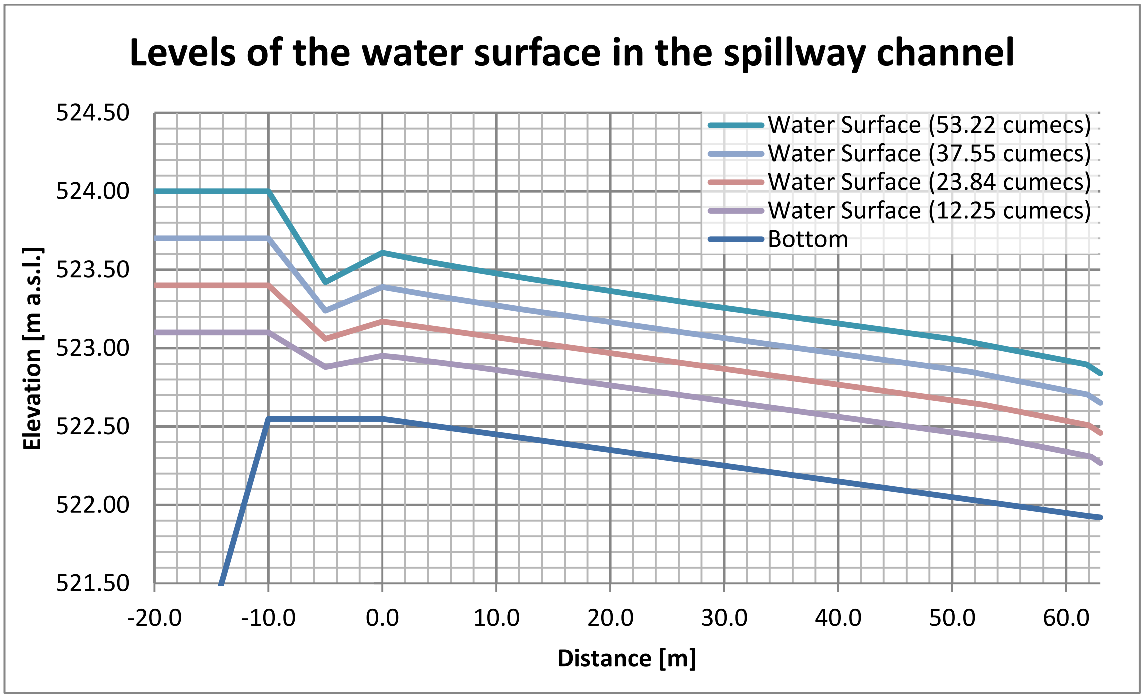
Table 7 - Auxiliary spillway capacity



Graph 4 - Spillway channel capacity curve

Water depth	[m]	0.00	0.05	0.10	0.20	0.30	0.40	0.50	0.70	0.90	1.10
Discharge	[cumecs]	0	0.37	1.17	3.69	7.22	11.63	16.81	29.26	44.22	61.43

Table 8 - Spillway channel capacity



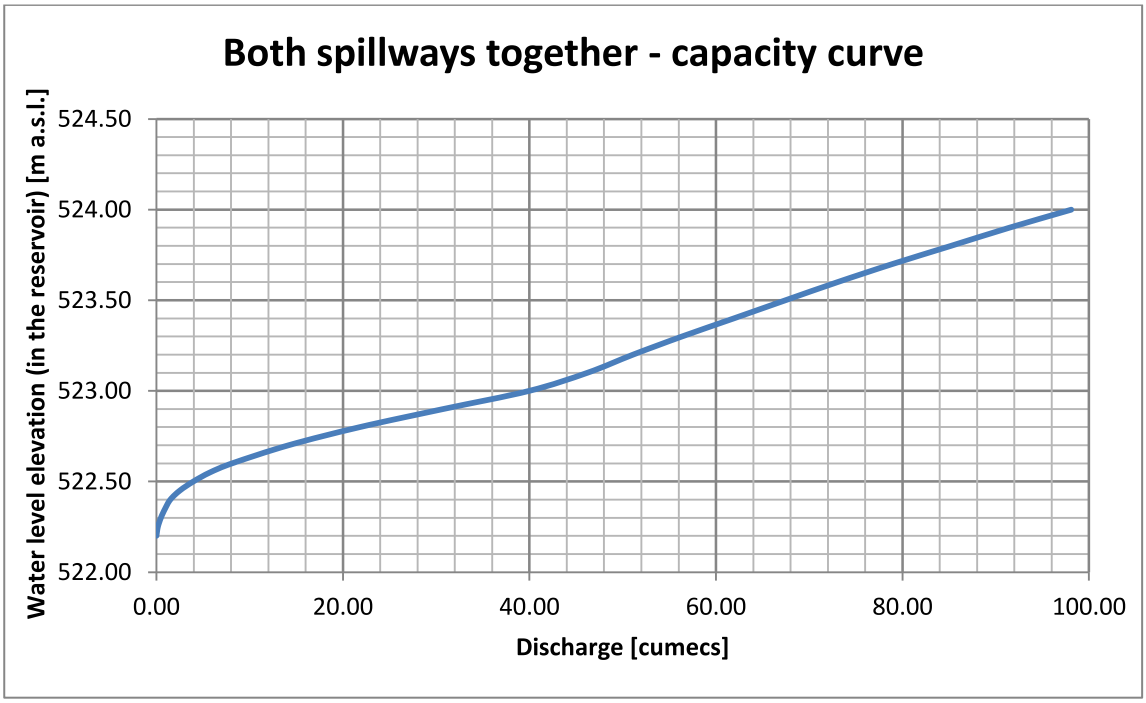
Graph 5 – Longitudinal section of the levels of the water surface in the spillway channel during several flows

Distance [m]	Bottom [m a.s.l.]	Depth [m]	Water Surface [m a.s.l.] (53.22 cumecs)
-20.0	520.00	1.45	524.00
-10.0	522.55	1.45	524.00
-5.0	522.55	0.87	523.42
0.0	522.55	1.06	523.61
4.5	522.51	1.04	523.55
8.0	522.47	1.03	523.50
13.2	522.42	1.02	523.44
28.7	522.26	1.01	523.27
50.7	522.04	1.01	523.05
61.8	521.93	0.96	522.90
63.0	521.92	0.92	522.84

Table 9 - Level of the water surface in the spillway channel during maximum flow

The spillway channel can discharge the design flow without any problems. After that, the discharges through both spillways were summarized and it is shown in *Graph 6* and *Table 10*.

Maximum discharge, which flows through both spillways together, is 98.09 cumecs. This discharge would occur when the water level would be on the elevation 524.00 metres a.s.l. It must be said that the discharge curves and tables are valid only in the case when the fuse plug has already been washed away.



Graph 6 - Both spillways together - capacity curve

Water level	[m a.s.l.]	522.20	522.40	522.60	522.80	523.00	523.20	523.40	523.60	523.80	524.00
Discharge	[cumecs]	0	1.49	8.12	21.81	39.98	51.10	61.91	73.02	85.17	98.09

Table 10 - Capacity of both spillways together

4.3.3. Necessary adjustments of the dam

As it was said previously, the maximum water level has to be raised. The design in the previous chapter assumed the maximum water level on the elevation 524.00 metres a.s.l. This solution has already been introduced in the study for safety improvement of the dam Ostrov nad Oslavou [2]. So the dam can be adjusted in the same way the study says.

The point is making the dam impervious until the crest of the dam. It was detected that the minimum level of the top of the impervious core along the dam is on the elevation 522.60 metres a.s.l. [2]

Firstly, the earth above the top of the impervious core will be removed to the elevation 523.00 metres a.s.l. There will be dug a groove to the impervious core along the dam which will be 1.00 meter deep and 0.60 metres wide. In this groove a vertical sealing element (e.g. sealing foil or copper sheet) will be inserted. The element will be sealed by the clay-cement mixture. At the upstream part of the crest of the dam, the concrete foundation for the precast concrete wave wall will be implemented. The foundation will be connected with the vertical sealing element by using anchors. The foundation will be 0.65 metres wide and 0.90 metres high. On the foundation, the precast concrete wave wall will be placed and connected with it. The wave wall has its top on the elevation 524.70 metres a.s.l. [2]

The service road on the crest of the dam will include surface from cement-concrete or asphalt-concrete with the load capacity of 30 tons. The road will be 3.80 metres wide. In the road base layer, the cable protector will be placed for the power line to the multipurpose flow control structure. A crash barrier will be located at the downstream part of the crest of the dam. [2]

The transport on the crest of the dam was interrupted by the spillway channel. Due to the interruption and to ensure that maintenance equipment can be transported to the spillway channel and downstream of the dam, the earthen ramp was designed. The ramp has a slope of 17 % and during the flood, which will wash the fuse plug away, the ramp will be washed away, too.

4.3.4. Design of the fuse plug

Finally, after the spillway has been designed, the fuse plug can be also designed. In this chapter, dimensions of the fuse plug, its materials, the pilot channel, the impervious core, the slope protection and the lateral erosion rate will be discussed.

Dimensions of the fuse plug

The fuse plug will be placed on the concrete auxiliary spillway crest. The fuse plug will be 1.45 metres high and the crest of the fuse plug will be on the elevation 524.00. The crest of the fuse plug will be 2 metres wide, which is enough for maintenance (e.g. grass cutting). The length of the crest of the fuse plug will be 19.29 metres long, but in this dimension the width of the pilot channel is included.

The fuse plug will have the same gradients of its slopes as the main dam has. It means that the gradient of the upstream slope will be 1:3.2 and the gradient of the downstream slope will be 1:2. The dimension of the toe of the fuse plug will be 9.54 metres. It means that the concrete crest is almost a half meter longer than the fuse plug toe in the flow direction is.

Pilot channel

The pilot channel will be situated in the middle of the fuse plug. Its dimensions are designed for flowing sufficient discharge to begin the washout. *Equations 4.9, 4.10* and *4.11* for broad crested unaffected weirs were used for discharge calculation and depth determination. Then, the shear stress was calculated because it is important to know what size of the effective grain would be washed out. The shear stress was calculated by *Equation 4.19* [33]. The critical shear stress, the stress limit which the effective grain can withstand was calculated by *Equation 4.20* [33].

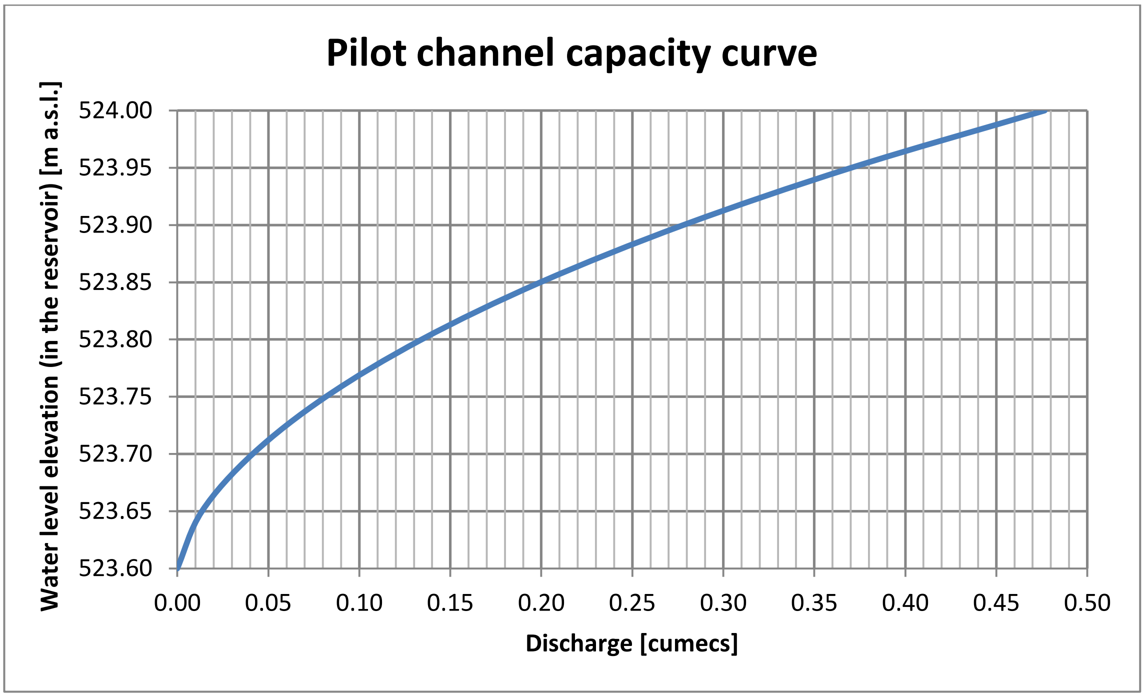
$$\tau_0 = \rho \cdot g \cdot h \cdot i_E \tag{4.19}$$

Where τ_0 – Shear stress at the bottom of the channel [Pa]
ρ – Density of water [kg/m^3]
h – Water depth [m]
i_E – Energy gradient [-]

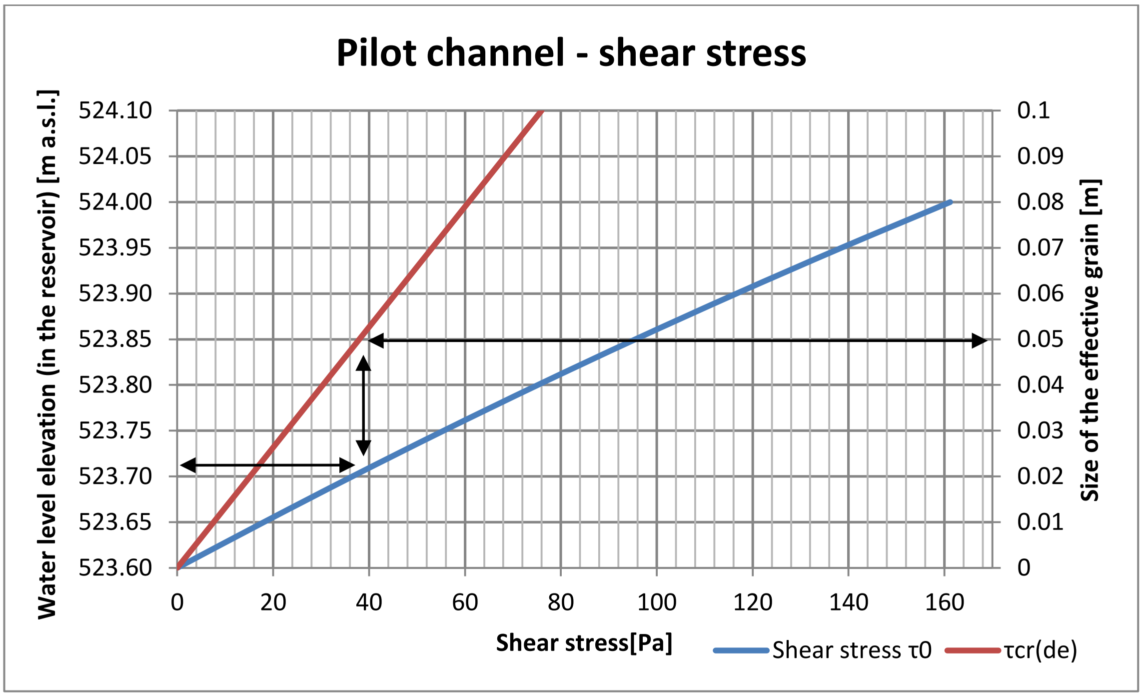
$$\tau_{cr} = 760 \cdot d_e \tag{4.20}$$

Where τ_{cr} – Critical shear stress [Pa]
d_e – Size of the effective grain [m]

The pilot channel discharge capacity is shown in *Graph 7* and *Table 11*. *Graph 8* shows the shear stress on the bottom of the pilot channel and also the critical shear stress for different sizes of effective grain.



Graph 7 - Pilot channel capacity curve



Graph 8 - Pilot channel - shear stress and critical shear stress of the effective grain

Water level	[m a.s.l.]	523.60	523.70	523.80	523.90	524.00
Discharge	[cumecs]	0	0.041	0.134	0.278	0.477
Shear stress τ_0	[Pa]	0	36.5	75.2	116.6	161.2

Table 11 - Pilot channel - discharge and shear stress

The width of the pilot channel should be about a half of the fuse plug height, so design width in the bottom is 0.70 metres. The slope of the bottom is 5 percents and gradient of both channel banks is 1:2. The elevation of the beginning of the pilot channel is 524.60 metres a.s.l., it means, 40 centimetres lower than the crest of the dam and the fuse plug, too. The existing spillway discharges over 40 cumecs during water level on the elevation 523.60 metres a.s.l. and this is more than 200-year flow. Material of the bottom of the pilot channel should contain the effective grain about 50 millimetres, it means approximately the fraction 32-63. Erosion of this material should begin roughly when the water level in the reservoir reaches the elevation 523.70 metres a.s.l.

Impervious core

In the fuse plug, there must be an impervious core to prevent the flow of water through the fuse plug. The impervious core will be inclined in the downstream direction and the angle is 1:1. The core will be made from a fine cohesive soil. Its coefficient of permeability should be $K = 10^{-7}$ m/s and lower.

The top of the impervious core is on the elevation 523.60 metres a.s.l., only at the part of the fuse plug where the pilot channel is, it is a little bit lower. The thickness of the impervious core is 50 centimetres and at its both sides (upstream and downstream), there can be filters with the width of 20 centimetres to prevent piping of the core. However, it is not necessary, because the fuse plug parts will be not under water level for a long-time period so the piping does not have to happen at all.

The impervious core will be joined with the sidewalls of the concrete auxiliary spillway structure. The sidewalls has slope 10:1 just because of the joint between the walls and the impervious core. The joint will be provided by clay slush.

Slope protection

The slope protection is necessary part of the fuse plug. It protects the fuse plug against external influences such as waves, rain, wind, snow melting etc. Without a slope protection, the fuse plug could breach earlier than it is required.

The slope protection will be implemented on both slopes of the fuse plug. The gradient of the upstream slope of the fuse plug is 1:3.2 and the gradient of the downstream slope is 1:2. The thickness of the layer of the slope protection will be 20 centimetres. Grains of the slope protection should have diameter between 70 and 150 millimetres as it is shown on the *Figure 7* in the *chapter 2.1.2*. So the fraction 63-125 suits best.

On the crest of the fuse plug, the same material like on the fuse plug slopes is not needed because the crest of the dam and the bottom of the pilot channel have only low gradient. So on the crest of the fuse plug and on the bottom of the pilot channel will be provided by the 100 millimetres thick layer, which is called gravel surfacing, with effective grain size about 50 millimetres which is the fraction 32-63.

The biggest part of the fuse plug, the stabilizing part, will be made from sand and gravel (coarse soils). Dimensions of the fuse plug or fuse plug parts can be adjusted according to possibilities of used machines.

Rate of the lateral erosion

Firstly, the fuse plug is eroded at the place where the pilot channel is. When the part of the fuse plug under the pilot channel is washed out then, the washout continuous with the lateral erosion. The rate of the lateral erosion can be counted by the empirical *Equation 2.2*. For this fuse plug, 1.45 metres high, the rate of the lateral erosion is 69.17 metres per hour. The pilot channel is in the middle of the fuse plug and the lateral erosion goes from the pilot channel to the both sides. So the whole fuse plug should be washed out in approximately 8 and a half minute by the lateral erosion.

Before the beginning of the lateral erosion, the part of the fuse plug under the pilot channel must be washed out, and it also takes some time. This time was estimated for 10 minutes. But for this issue the model study would be needed.

4.3.5. Volume of materials

There is a large amount of materials which has to be removed, replaced or used for reconstruction of the dam. The volume of materials was approximately calculated according to technical drawings which are attached in this thesis in the last chapter (*8.2 Technical enclosures*).

Materials will be used for the reconstruction of the crest of the dam, for the building of the auxiliary spillway and its channel and for the realization of the fuse plug.

List of needed materials:

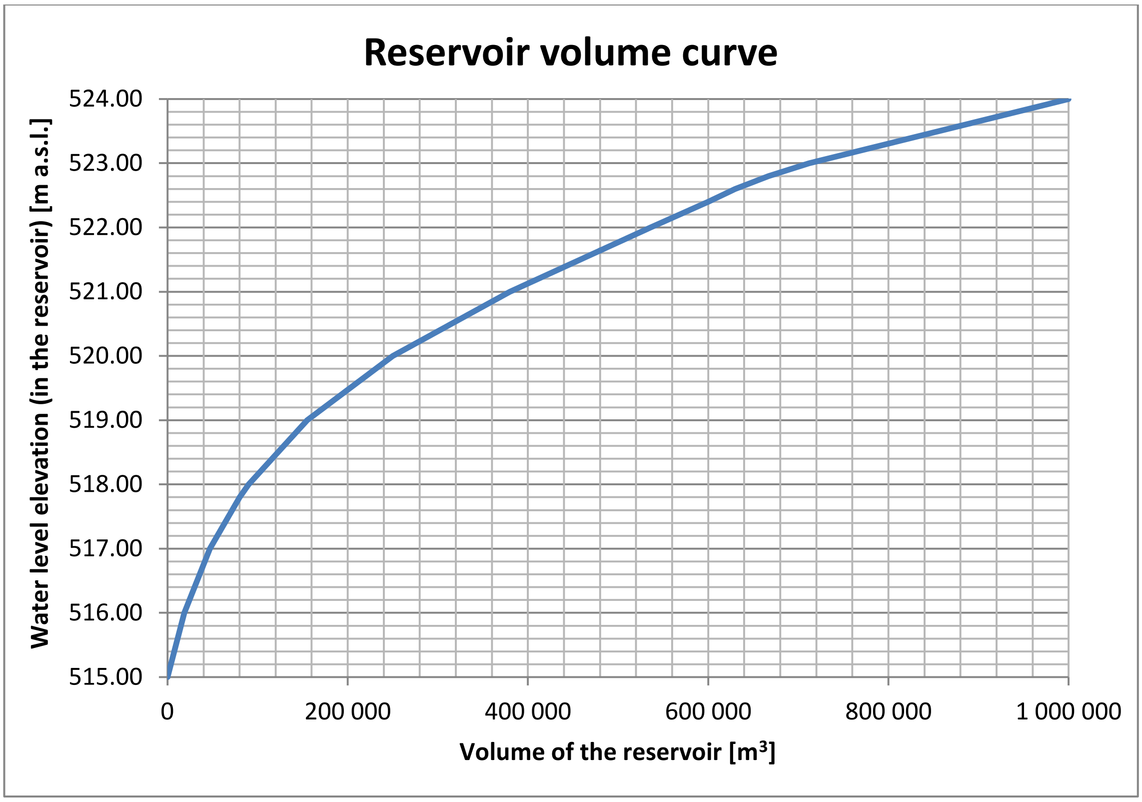
• Concrete	90	m^3
• Reinforced concrete	146	m^3
• Precast wave wall	116	m
• Sealing element	209	m^2
• Clay-cement mixture	77	m^3
• Crash barrier	120	m
• Grass restoration	252	m^2
• Construction of the service road	456	m^2
• New soil for the crest of the dam	289	m^3
• Clay slush	16	m^2
• Fine cohesive soil for the impervious core	14	m^3
• Filters	11	m^3
• Sand and gravel (coarse soils)	100	m^3
• Gravel 63-125	31	m^3
• Gravel 32-63	3	m^3
• Riprap	329	m^3
• Soil for the ramp	38	m^3

List of removed materials:

• Precast concrete panels	360	m^2
• Upper part of the crest of the dam	507	m^3
• Soil (which will be replaced by the auxiliary spillway and its channel)	3321	m^3

5. Assessment of the 1000-year flood

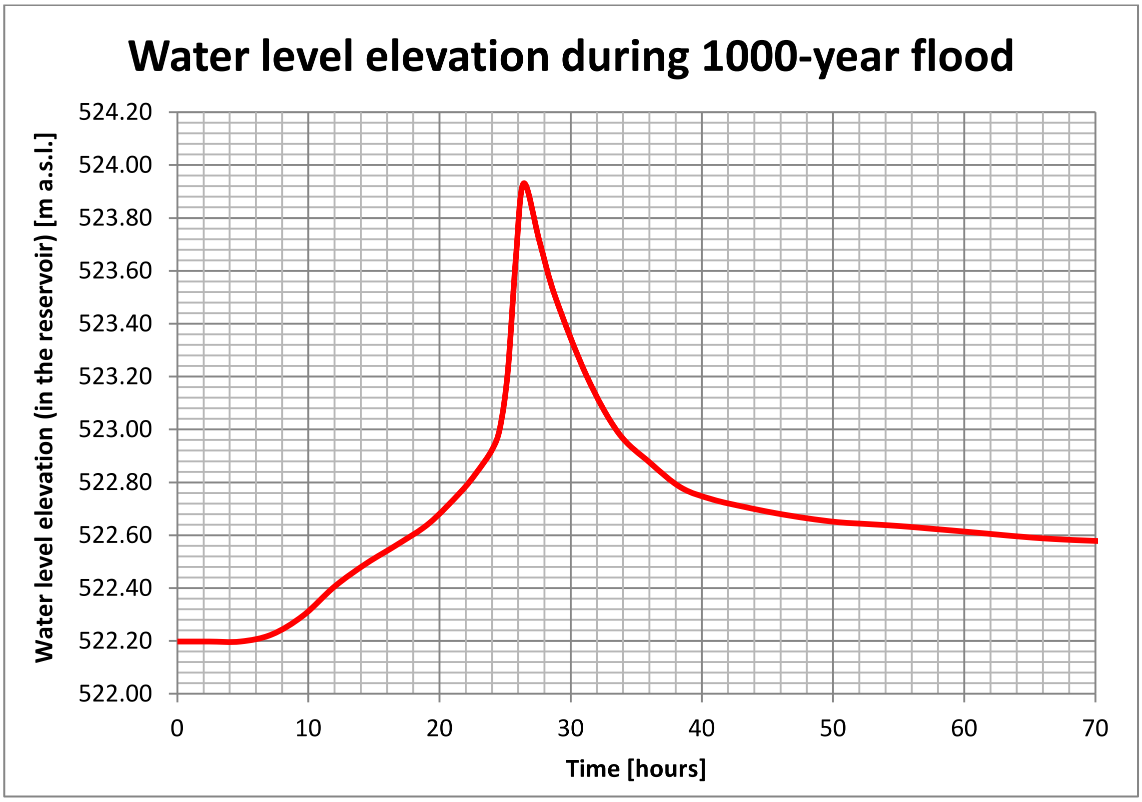
Now, it is time to assess the process of the 1000-year flood through the reconstructed dam. The theoretical 1000-year flood was provided by the Czech Hydrometeorological institute. The next necessary point that was needed was the reservoir volume curve (*Graph 9*). The reservoir volume curve shows an exact volume of water in the reservoir during a certain water level elevation. For the calculation of the assessment of the 1000-year flood, the Excel programme was used.



Graph 9 - Reservoir volume curve

Unfortunately, the theoretical flood has the very coarse time-scale, so the process of the fuse plug washout is invisible in the result. However, before the reconstruction, the dam would be overflowed by the theoretical 1000-year flood. After the reconstruction, the dam would not be overflowed what shows the *Graph 10* but moreover, the discharge is decreased from the 96.7 cumecs to 79.8 cumecs.

The assessment was counted according to rules which are described in the *chapter 3.4-Manipulation during flood*. The bottom outlets were used only before and after the flood for discharging maximum harmless flow, and during the peak flow, they were closed. The resultant graph is attached in the last chapter (*8.2 Technical enclosures*, *TE.09.*).



Graph 10 - Water level elevation during 1000-year flood

6. Conclusion

Before the writing of this thesis was started, the purposes of the work were the theoretical part which includes fuse plug and fuse gate system description and their utilization in dam safety improvement measures, and the practical part which is focused on a solution with safety improvement of the Ostrov nad Oslavou dam. The dam safety was supposed to increase from the resistance against 100-year flood to the resistance against 1000-year flood. The design was supposed to include the safety improvement by an auxiliary spillway with a fuse plug or a fuse gate system.

In the part of the thesis where the theory of fuse plug and fuse gate systems was described, there was mentioned everything important about their parts, advantages, disadvantages, their design and their hydraulic. These systems have never been used in the Czech Republic so far. The main reason for it is that it is not possible to test these systems in a trial operation in the real waterworks and it is also their main disadvantage. Their utilization is mostly in dams, where it is required simultaneously to increase the water level in the reservoir (or capacity of the reservoir) and to improve the safety of the dam.

Next section was dedicated to the dam Ostrov nad Oslavou which is placed in the Jihlava region in the Czech Republic. The equipment of the dam, its hydrological data and its manipulation during flood events were depicted.

The design started with restrictive requirements and with the decision which system will be considered further. The fuse plug system was chosen for its variability and for lower costs. The capacity of the existing spillway was calculated, and after that, the design of the auxiliary spillway crest could be performed. The crest was designed as a broad crested weir connected with the spillway channel which drains the flow to the safety distance from the dam. The auxiliary spillway was designed to discharge the peak flow of the 1000-year flood with the existing spillway together. The necessary adjustments of the upper part of the dam were discussed next. The maximum water level had to be increased from the elevation 523.00 metres a.s.l. to the elevation 524.00 metres a.s.l.

The final design of the fuse plug is based on the theoretical part of this thesis. It contains dimensions and material requirements in the appropriate level of detail. The rate of the lateral erosion was counted, and the time needed for the washout of the part of the fuse plug under the pilot channel was estimated. The thesis was finished by the lists of the materials (needed and removed) and by the assessment of the process of the 1000-year flood through the dam after its reconstruction. The result of the assessment was satisfying.

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8. Enclosures

8.1. Theoretical enclosures

Model	C_1	C_2	C_3	R^2
WLH(NHL)	0.32 ±0.004	0.001	−0.292±0.006	1.00
WLH(TVA)	0.306±0.009	0.001	−0.360±0.013	0.98
NLH	0.254±0.012	0.0005	−0.319±0.018	0.94
WHH	0.315±0.009	0.0375	−0.258±0.005	0.99

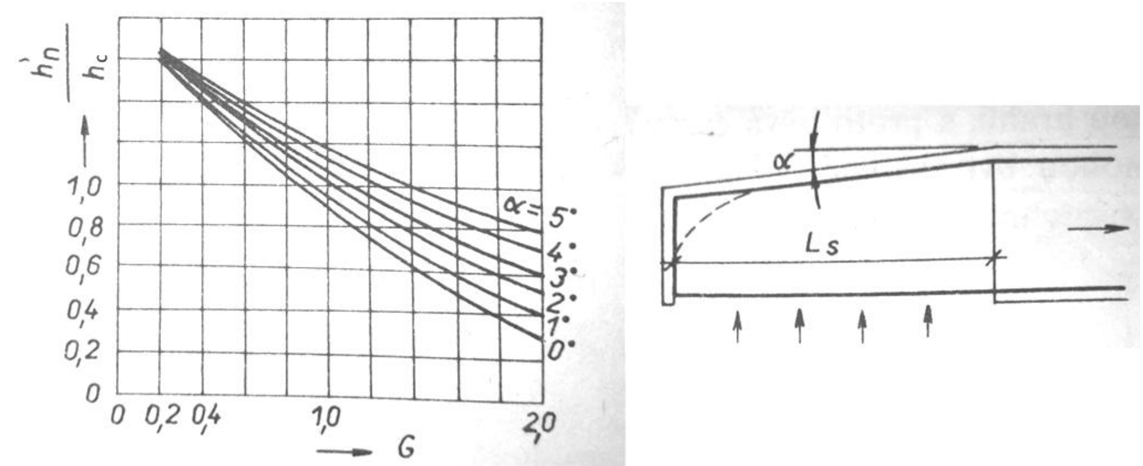
NHL Experiments at National Hydraulic Laboratory, Chatou, France.
TVA Experiments at Tennessee Valley Authority Laboratory, US.
R = Correlation coefficient

Enclosure 1 - Empirical discharge coefficients for fuse gates [21]

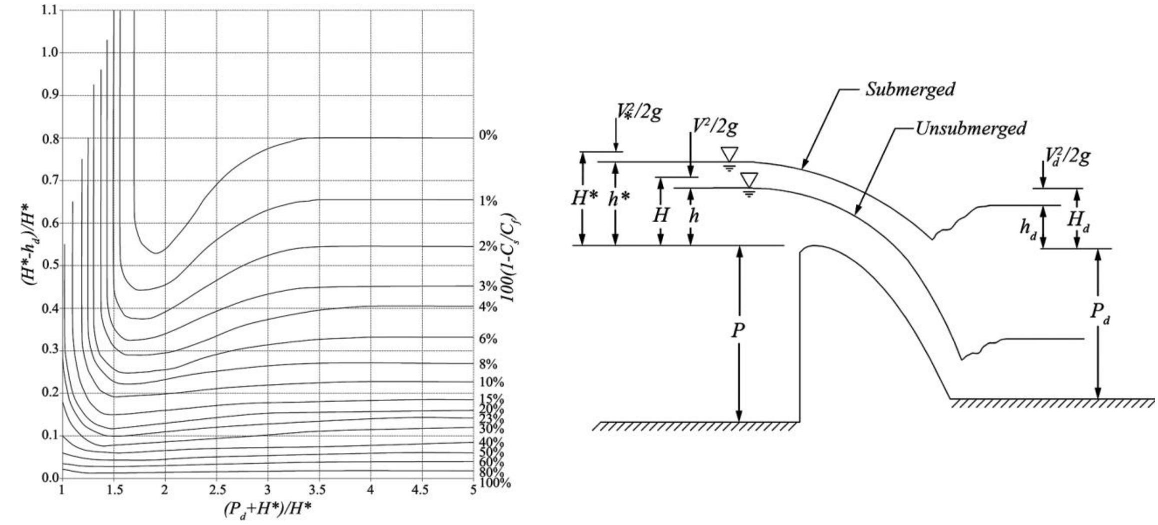
	Number of fuse gates in position Discharge in cumec							
Head on fuse gate h	0	1	2	3	4	5	6	7
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	50.63	42.43	34.24	27.0	18.81	12.54	6.27	0
0.2	61.08	51.96	42.86	35.06	25.96	19.48	13.08	6.54
0.4	72.18	62.29	52.42	44.23	34.34	27.92	21.51	15.10
0.6	83.87	73.17	62.47	54.01	43.33	37.10	30.87	24.65
0.8	96.14	84.58	73.02	64.28	52.72	46.78	40.83	34.90
1.0	108.95	96.46	83.95	74.92	62.42	56.84	51.27	45.68
Side contraction	0	2	4	4	6	4	2	0
Head at tilting	–	0.90	0.85	0.80	0.75	0.70	0.65	0.607
Discharge	100.0	90.5	75.8	64.3	50.4	41.9	33.4	25.0
Delta Q	9.5	14.7	11.5	13.9	8.5	8.5	8.4	0

(1) Discharges in Col (2) correspond to flow only over the spillway, with all the 7 fuse gates tilted.
(2) Discharges in Col (9) correspond to floe over the crest of all the 7 fuse gates in position.
(3) All other discharges comprise combination of discharges over the fuse gates and spillway segments.

Enclosure 2 - Discharges over the fuse gates and spillway (example solution) [5]



Enclosure 3 - Komora's diagram [12]

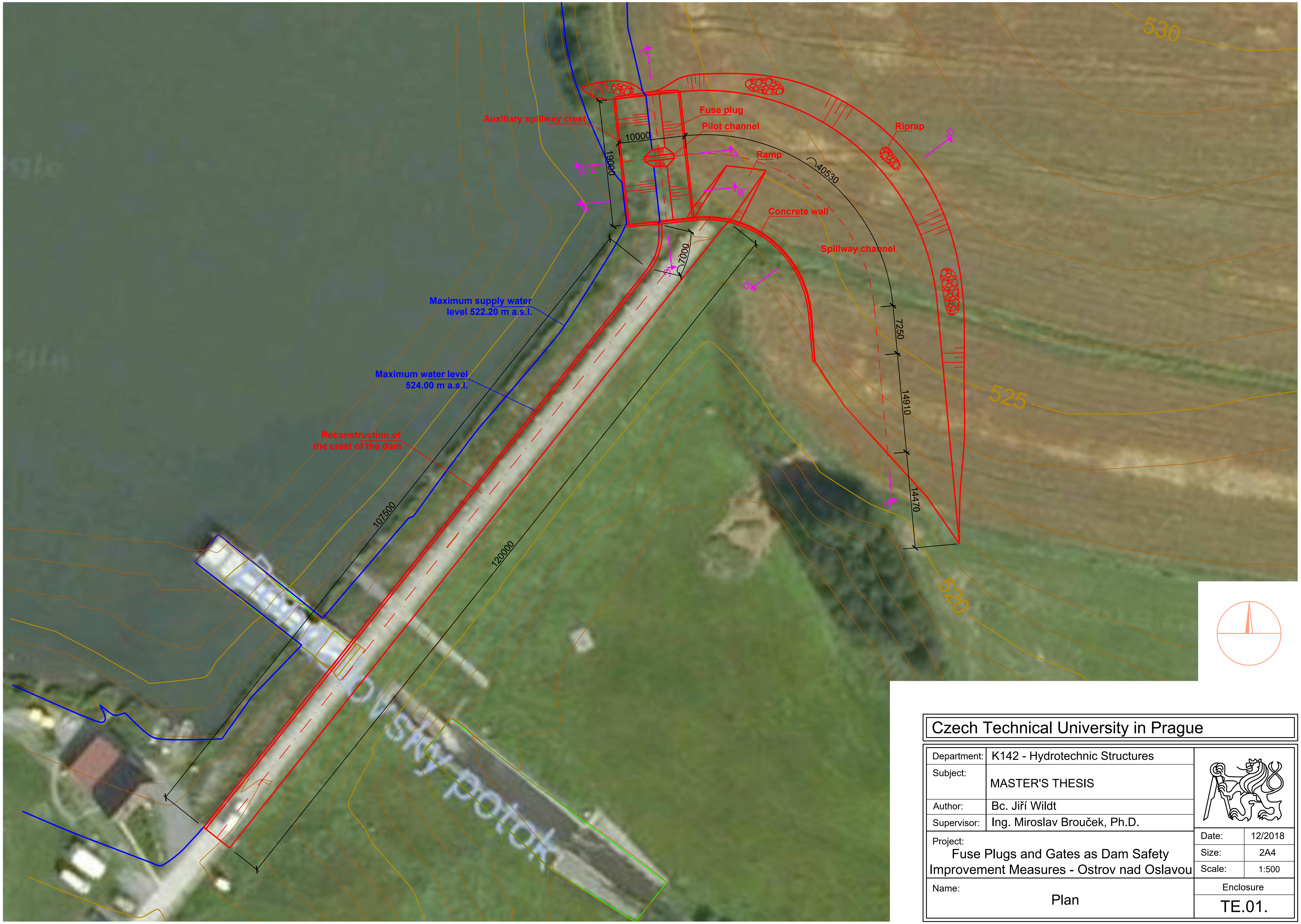


Enclosure 4 - Denver diagram [36]

8.2. Technical enclosures

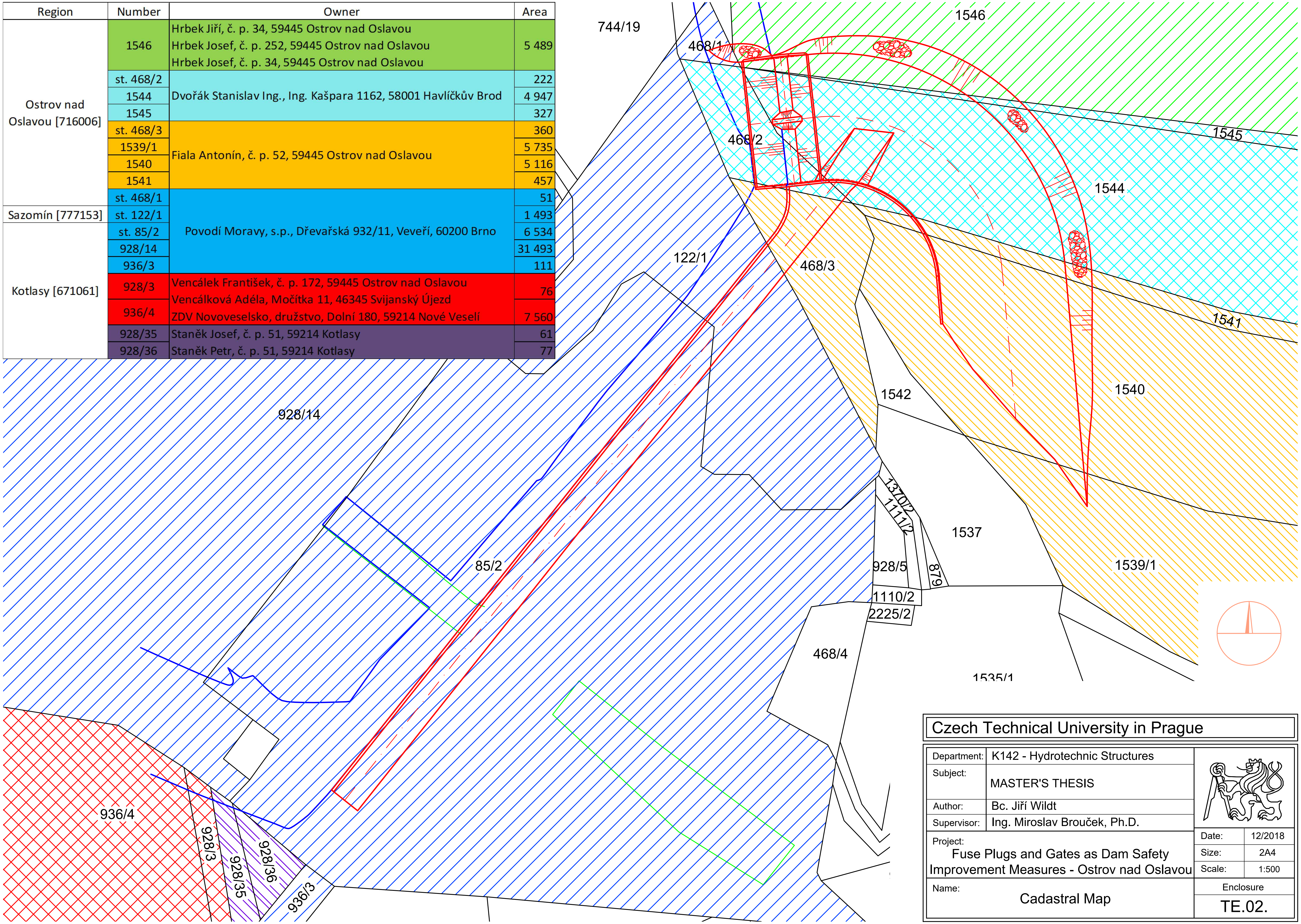
List of the technical enclosures:

• TE.01.	Plan	M 1:500
• TE.02.	Cadastral Map	M 1:500
• TE.03.	Plan – Auxiliary Spillway	M 1:250
• TE.04.	Typical Cross-Section – Crest of the Dam	M 1:100
• TE.05.	Cross-Sections A-A' and B-B' Fuse Plug and Pilot Channel (with Filters)	M 1:50
• TE.06.	Cross-Sections A-A' and B-B' Fuse Plug and Pilot Channel (without Filters)	M 1:50
• TE.07.	Cross-Sections C-C' and D-D' Spillway crest and Spillway Channel	M 1:100
• TE.08.	Longitudinal section E-E' Pilot Channel + Spillway Channel	M 1:50/500
• TE.09.	Assessment of the 1000-year flood – graph	



Czech Technical University in Prague			
Department:	K142 - Hydrotechnic Structures		
Subject:	MASTER'S THESIS		
Author:	Bc. Jiří Wildt		
Supervisor:	Ing. Miroslav Brouček, Ph.D.	Date:	12/2018
Project:	Fuse Plugs and Gates as Dam Safety Improvement Measures - Ostrov nad Oslavou	Size:	2A4
		Scale:	1:500
Name:	Plan	Enclosure	TE.01.

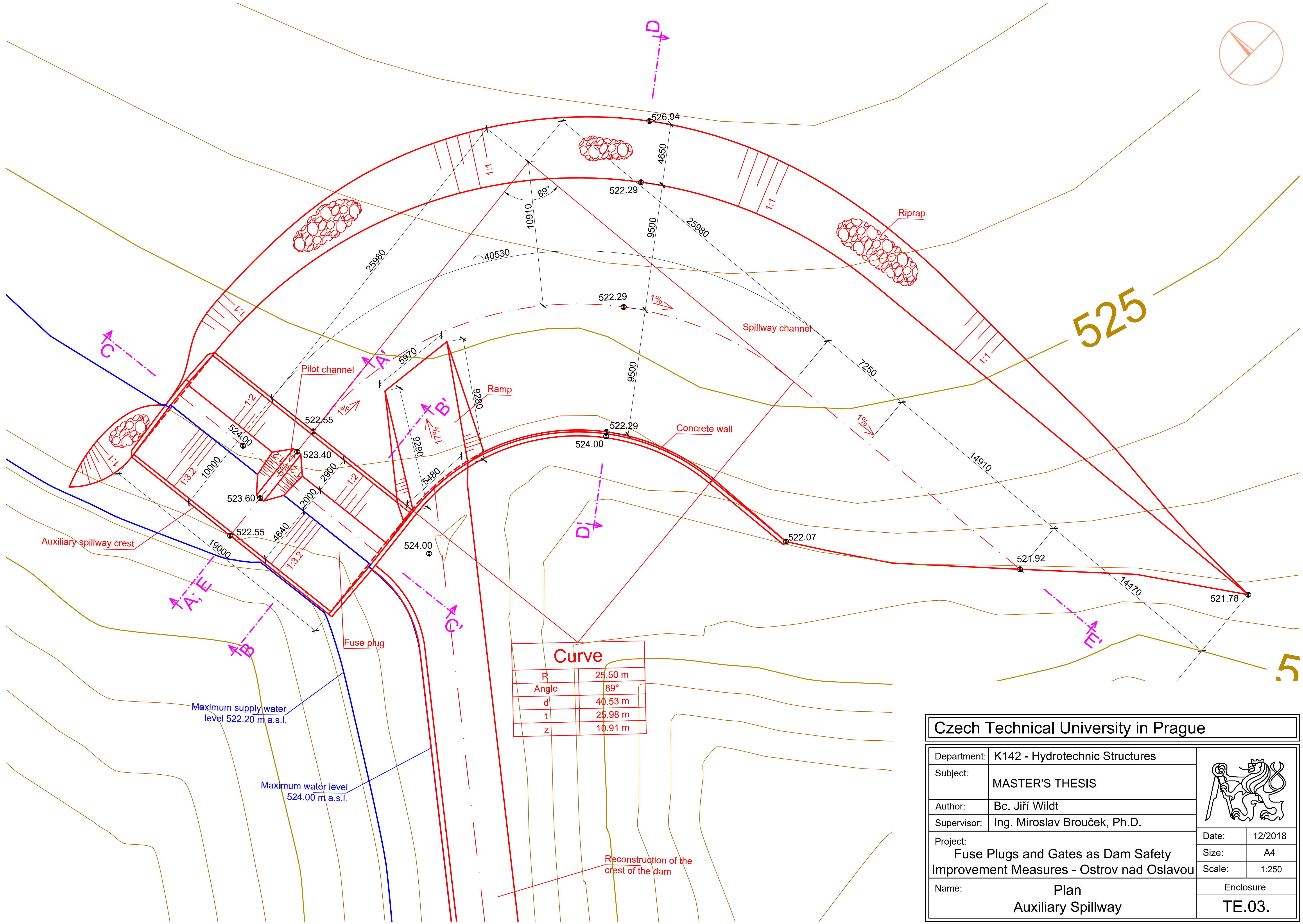
Region	Number	Owner	Area
Ostrov nad Oslavou [716006]	1546	Hrbek Jiří, č. p. 34, 59445 Ostrov nad Oslavou Hrbek Josef, č. p. 252, 59445 Ostrov nad Oslavou Hrbek Josef, č. p. 34, 59445 Ostrov nad Oslavou	5 489
	st. 468/2	Dvořák Stanislav Ing., Ing. Kašpara 1162, 58001 Havlíčkův Brod	222
	1544		4 947
	1545		327
	st. 468/3	Fiala Antonín, č. p. 52, 59445 Ostrov nad Oslavou	360
	1539/1		5 735
	1540		5 116
	1541		457
	st. 468/1	Povodí Moravy, s.p., Dřevařská 932/11, Veveří, 60200 Brno	51
Sazomín [777153]	st. 122/1		1 493
Kotlasy [671061]	st. 85/2		6 534
	928/14		31 493
	936/3		111
	928/3	Vencálek František, č. p. 172, 59445 Ostrov nad Oslavou Vencálková Adéla, Močítka 11, 46345 Svijanský Újezd ZDV Novoveselsko, družstvo, Dolní 180, 59214 Nové Veselí	76
	936/4		7 560
	928/35	Staněk Josef, č. p. 51, 59214 Kotlasy	61
	928/36	Staněk Petr, č. p. 51, 59214 Kotlasy	77



Czech Technical University in Prague

Department:	K142 - Hydrotechnic Structures
Subject:	MASTER'S THESIS
Author:	Bc. Jiří Wildt
Supervisor:	Ing. Miroslav Brouček, Ph.D.
Project:	Fuse Plugs and Gates as Dam Safety Improvement Measures - Ostrov nad Oslavou
Name:	Cadastral Map

Date:	12/2018
Size:	2A4
Scale:	1:500
Enclosure	TE.02.



Curve

R	25.50 m
Angle	89°
d	40.53 m
t	25.98 m
z	10.91 m

Czech Technical University in Prague			
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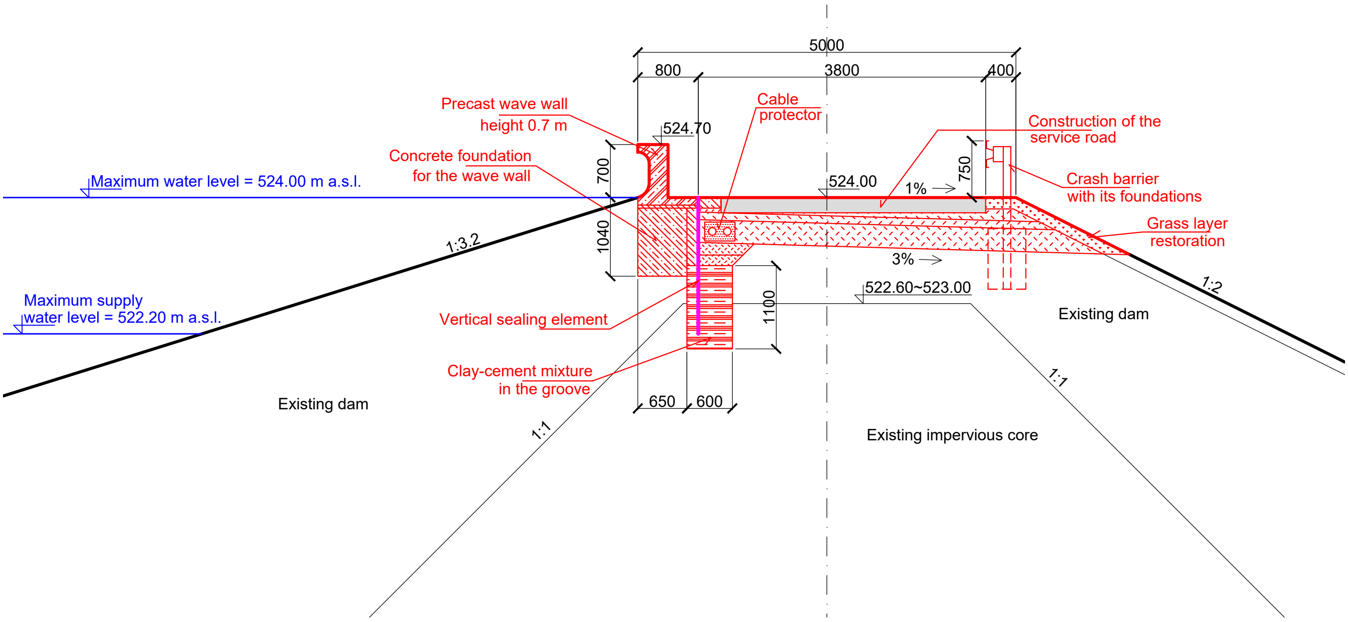
Subject:	MASTER'S THESIS		
Author:	Bc. Jiří Wildt		
Supervisor:	Ing. Miroslav Brouček, Ph.D.	Date:	12/2018
Project:	Fuse Plugs and Gates as Dam Safety Improvement Measures - Ostrov nad Oslavou	Size:	A4
		Scale:	1:250
Name:	Plan Auxiliary Spillway	Enclosure	TE.03.

Typical Cross-Section
CREST OF THE DAM
M 1:100

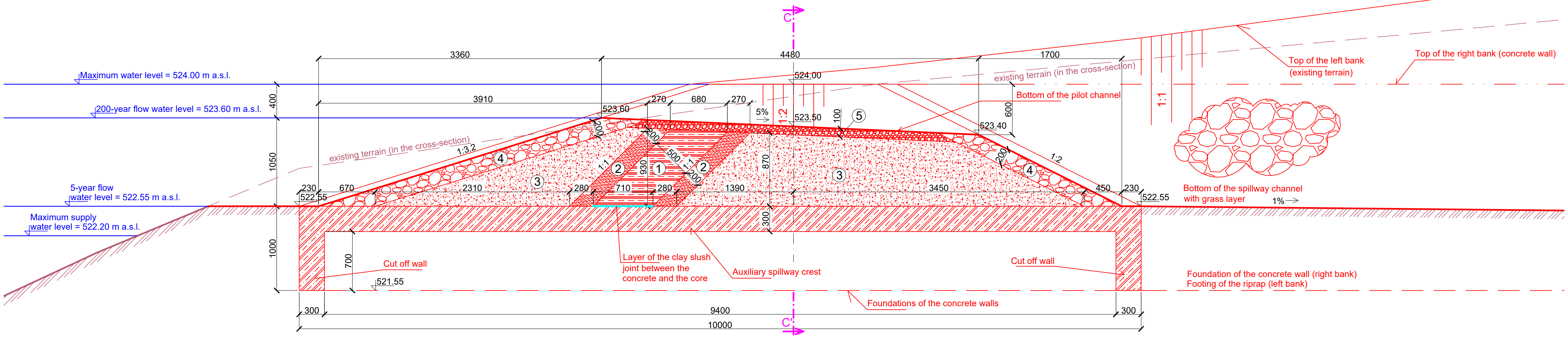
This technical drawing was taken from Study for safety improvement of the dam Ostrov nad Oslavou [2]



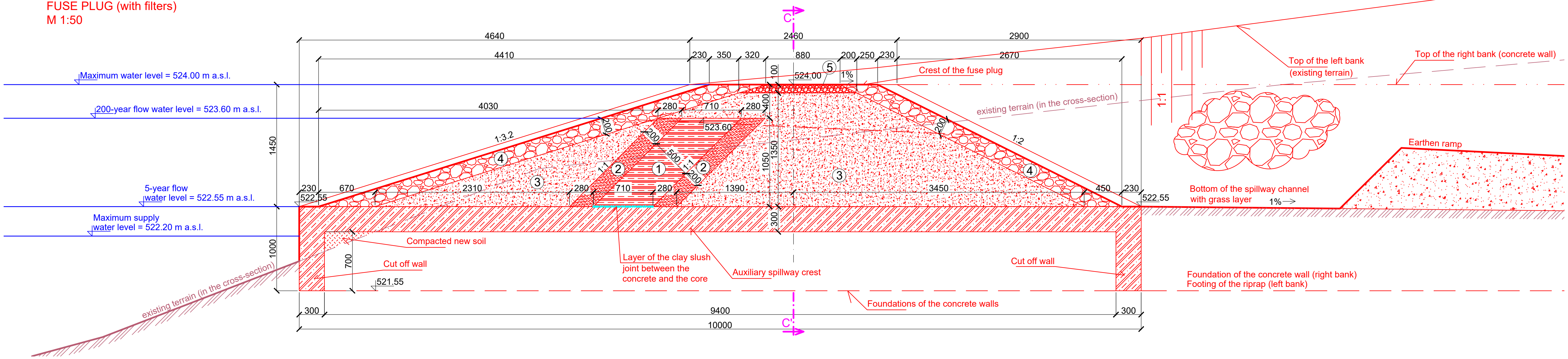
Czech Technical University in Prague

Department:	K142 - Hydrotechnic Structures		
Subject:	MASTER'S THESIS		
Author:	Bc. Jiří Wildt		
Supervisor:	Ing. Miroslav Brouček, Ph.D.		
Project: Fuse Plugs and Gates as Dam Safety Improvement Measures - Ostrov nad Oslavou		Date:	12/2018
		Size:	A4
		Scale:	1:100
Name:	Typical Cross-Section Crest of the Dam	Enclosure	TE.04.

Cross-section A-A'
PILOT CHANNEL (with filters)
M 1:50



Cross-section B-B'
FUSE PLUG (with filters)
M 1:50



① Impervious core - fine cohesive soil; K = 10^{-7} m/s or lower; PS 95%
② Filters
③ Stabilizing part - sand and gravel (coarse soils); I_d > 0.8
④ Slope protection - fraction 63-125 mm
⑤ Gravel surfacing - fraction 32-63 mm

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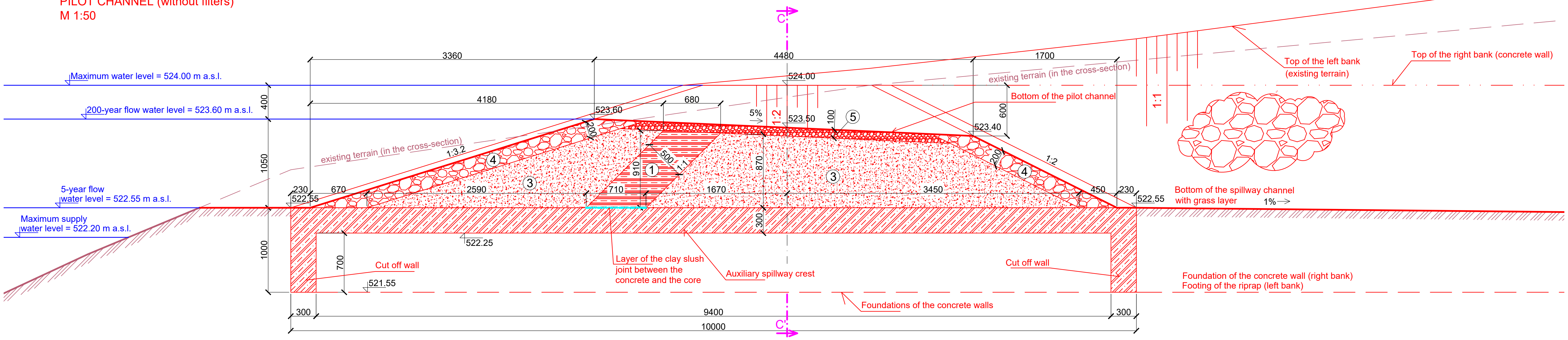
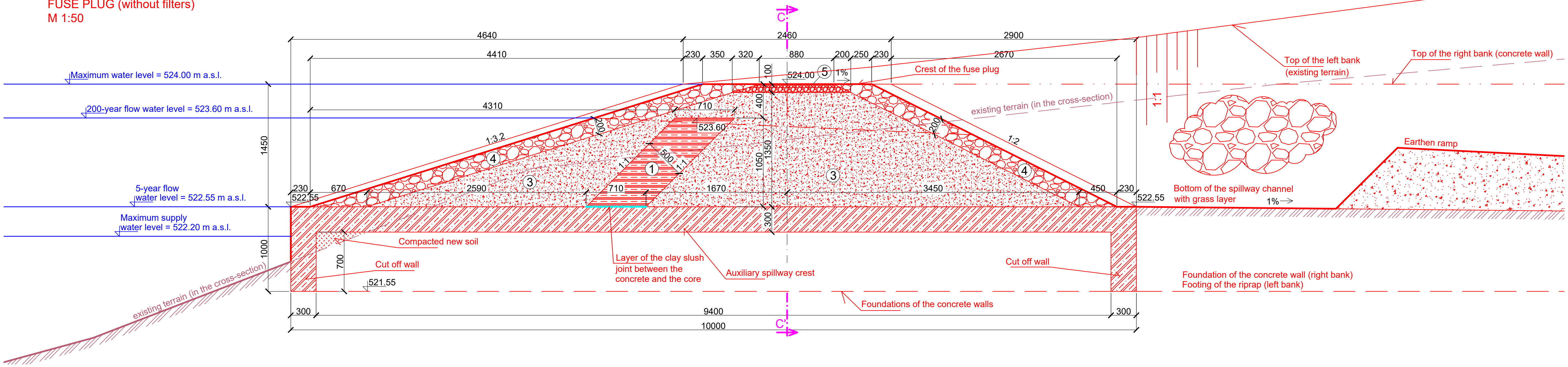
Subject:	MASTER'S THESIS		
Author:	Bc. Jiří Wildt		
Supervisor:	Ing. Miroslav Brouček, Ph.D.	Date:	12/2018
Project: Fuse Plugs and Gates as Dam Safety Improvement Measures - Ostrov nad Oslavou		Size:	2A4
		Scale:	1:50
Name: Cross-Sections A-A' and B-B' Fuse Plug and Pilot Channel (with Filters)		Enclosure	TE.05.

Cross-section A-A'
PILOT CHANNEL (without filters)
M 1:50



Cross-section B-B'
FUSE PLUG (without filters)
M 1:50

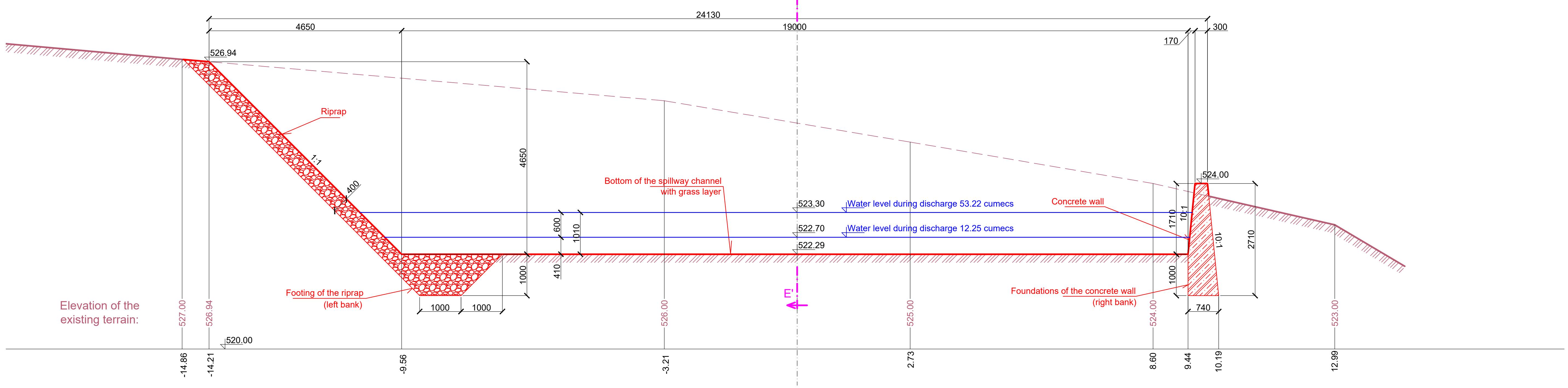


1 Impervious core - fine cohesive soil; K = 10^{-7} m/s or lower; PS 95%
3 Stabilizing part - sand and gravel (coarse soils); I_d > 0.8
4 Slope protection - fraction 63-125 mm
5 Gravel surfacing - fraction 32-63 mm

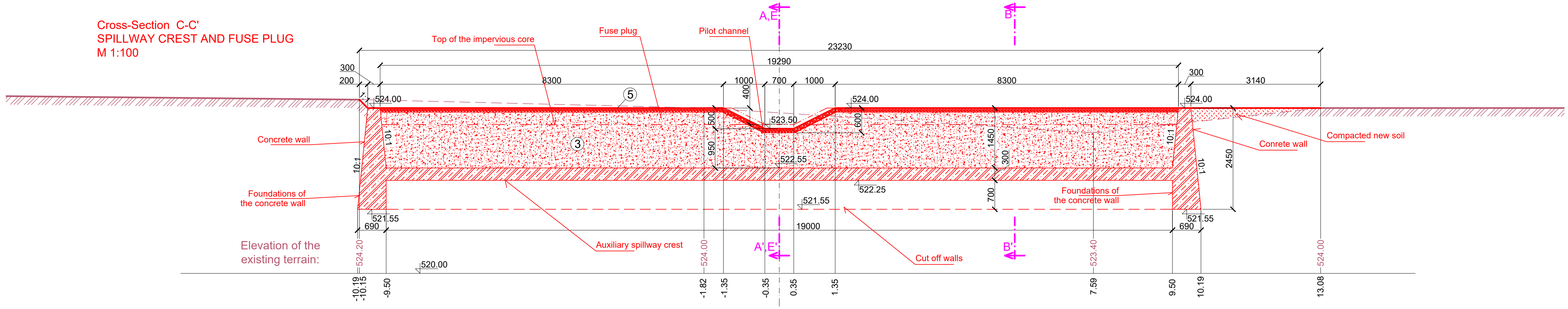
Czech Technical University in Prague

Department:	K142 - Hydrotechnic Structures		
Subject:	MASTER'S THESIS		
Author:	Bc. Jiří Wildt		
Supervisor:	Ing. Miroslav Brouček, Ph.D.	Date:	12/2018
Project:	Fuse Plugs and Gates as Dam Safety Improvement Measures - Ostrov nad Oslavou	Size:	2A4
		Scale:	1:50
Name:	Cross-Sections A-A' and B-B' Fuse Plug and Pilot Channel (without Fil.)	Enclosure	TE.06.

Cross-Section D-D'
SPILLWAY CHANNEL
M 1:100



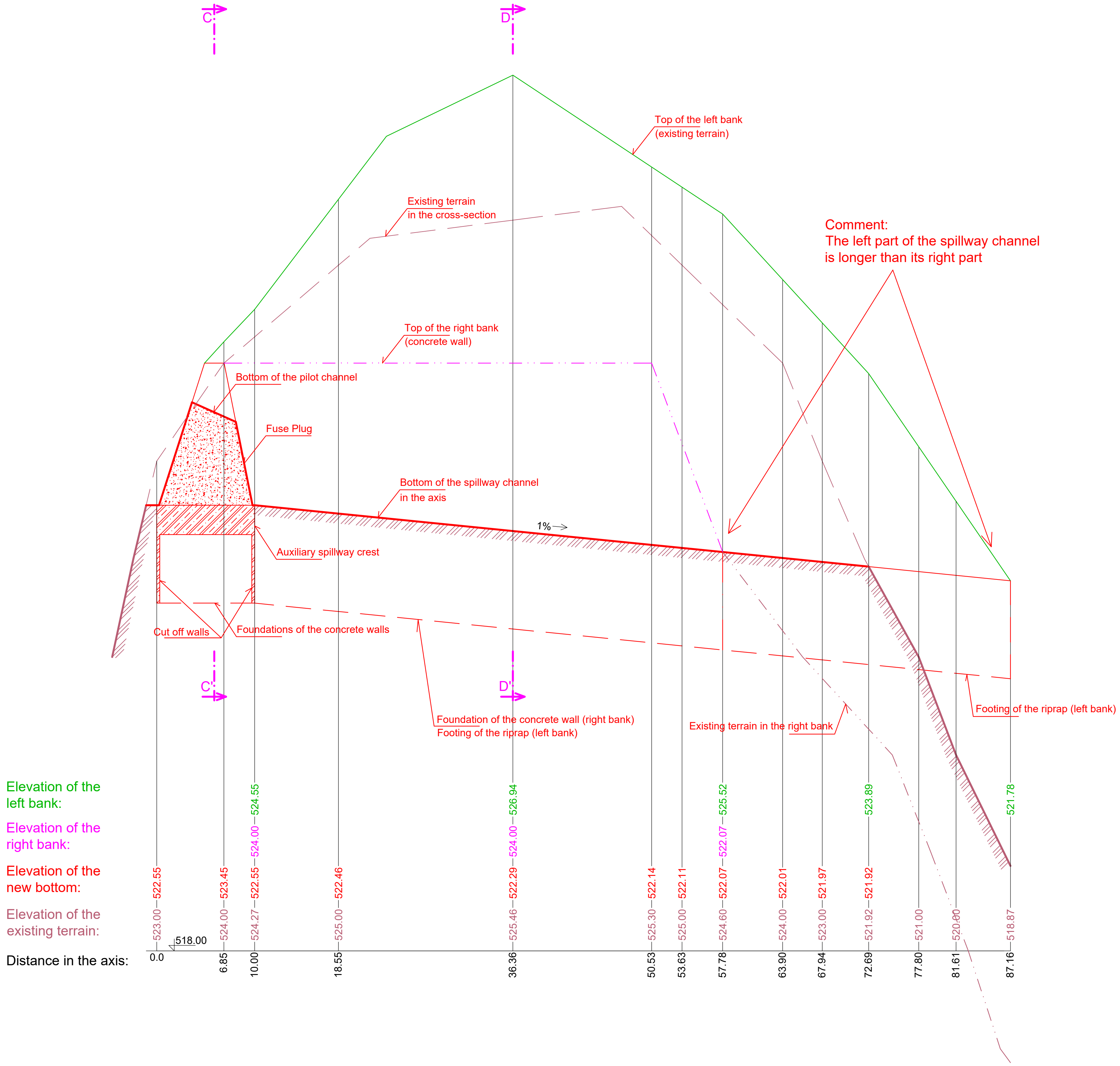
Cross-Section C-C'
SPILLWAY CREST AND FUSE PLUG
M 1:100



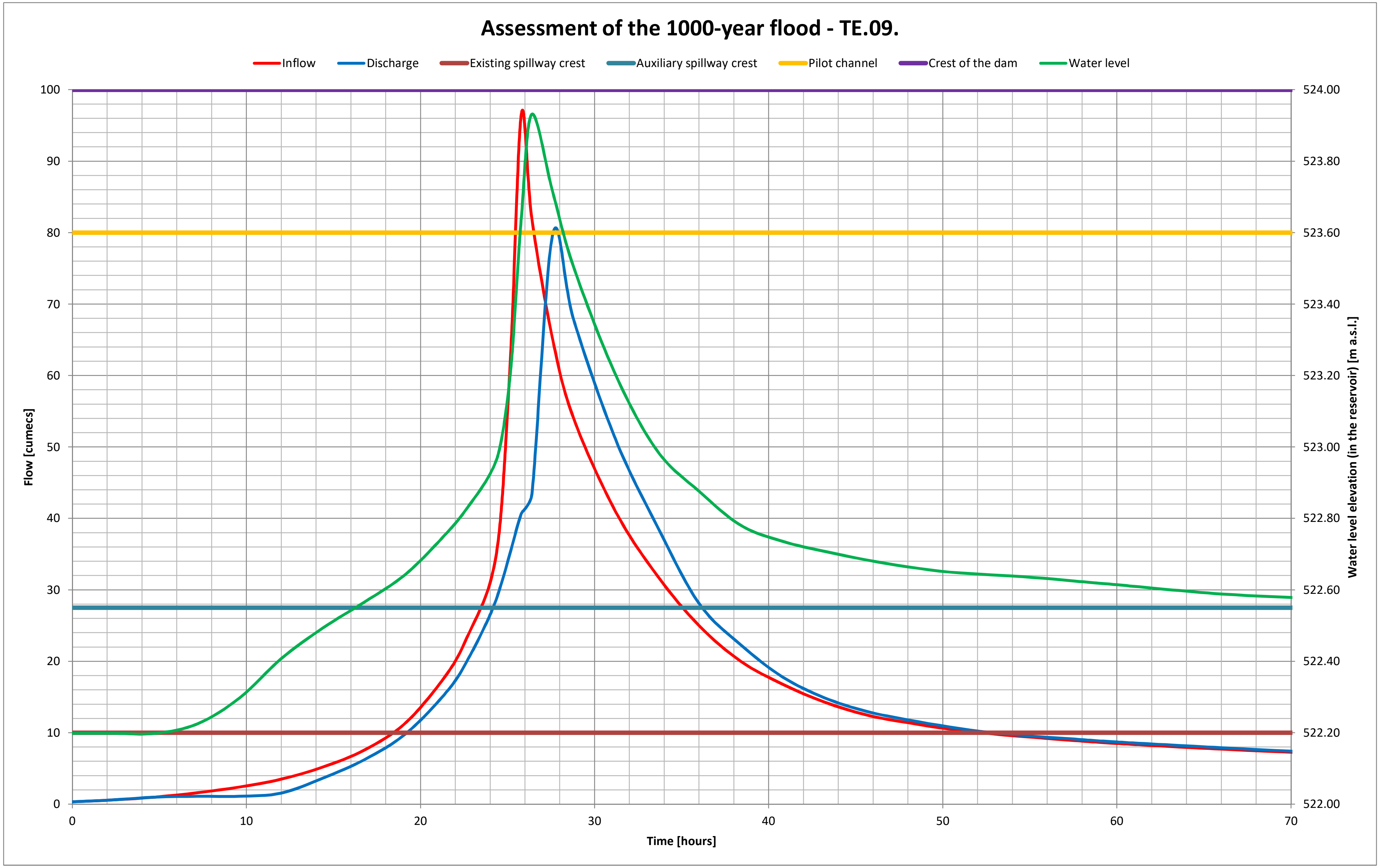
③ Stabilizing part - sand and gravel (coarse soils); I_d > 0.8
⑤ Gravel surfacing - fraction 32-63 mm

Czech Technical University in Prague	
Department:	K142 - Hydrotechnic Structures
Subject:	MASTER'S THESIS
Author:	Bc. Jiří Wildt
Supervisor:	Ing. Miroslav Brouček, Ph.D.
Project:	Fuse Plugs and Gates as Dam Safety Improvement Measures - Ostrov nad Oslavou
Name:	Cross-Sections C-C' and D-D' Spillway crest and Spillway Channel
Date:	12/2018
Size:	2A4
Scale:	1:100
Enclosure	TE.07.

Longitudinal-section E-E'
PILOT CHANNEL + SPILLWAY CHANNEL
M 1:50/500



Czech Technical University in Prague	
Department:	K142 - Hydrotechnic Structures
Subject:	MASTER'S THESIS
Author:	Bc. Jiří Wildt
Supervisor:	Ing. Miroslav Brouček, Ph.D.
Project:	Fuse Plugs and Gates as Dam Safety Improvement Measures - Ostrov nad Oslavou
Name:	Longitudinal section E-E' Pilot Channel + Spillway Channel
Date:	12/2018
Size:	2A4
Scale:	1:50/500
Enclosure	TE.08.


Assessment of the 1000-year flood - TE.09.
Inflow
Discharge
Existing spillway crest
Auxiliary spillway crest
Pilot channel
Crest of the dam
Water level
Flow [cumecs]
Water level elevation (in the reservoir) [m a.s.l.]
Time [hours]
0
10
20
30
40
50
60
70
80
90
100
522.00
522.20
522.40
522.60
522.80
523.00
523.20
523.40
523.60
523.80
524.00