CHAPTER 5

TECHNICAL ANALYSIS

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ABBREVIATIONS AND ACRONYMS

AC	Asbestos Cement
BACF	Biological activated carbon filters
BOT	Built-Operate-Transfer
BOD	Biochemical oxygen demand
BPT	Breaking Pressure Tank
CBA	Cost Benefit Analyses
COD	Chemical oxygen demand
DAF	Dissolved air flotation
DI	Ductile iron pipe
DWD	Drinking Water Directive
DN	Diameter Nominal
EIA	Environmental Impact Assessment
EIB	European Investment Bank
EK	Economic benefit
EPA	Environmental Protection Agency
EU	European Union (from 1992)
EUR	European Euro
FIDIC	International Federation of Consulting Engineers
GAC	Granular Activated Carbon
GoS	Government of Serbia
GDP	Gross Domestic Product
GRP	Glass-reinforced-plastic pipe
HDPE	High density polyethylene pipes
HRT	Hydraulic Retention Time
IRR	Internal Rate of Return
LEAP	Local Environmental Action Plan
MAC	Maximum allowable concentration
masl	Meters above mean sea level
MLSS	Mixed Liquor Suspended Solids
MRB	Measurement and control block
NEAP	National Environmental Action Plan
NES	National Environmental Strategy
NOM	Natural organic matter
O&M	Operation and Maintenance
OCK	Opportune Costs of Capital
OGRS	Official Gazette of Republic of Serbia
PE	Population Equivalent / Polyethylene
PET	Polyethylene terephthalate
PIP	Project Implementation Plan
PP	Prefabricated Polypropylene
PRAG	Practical Guide to Contract Procedures financed from the general budget of the EC in the context of external actions
PS	Pumping Station
PUC	Public Utility Company
PVC	Polyvinyl Chloride
SBR	Sequencing Batch Reactor
SCADA	Supervisory control and data acquisition
SPS	Sewage Pumping Station
SRT	Sludge Retention Time
SS	Suspended solids
SVI	Sludge Volume Index
TA	Technical Assistance
TDS	Total Dissolved Solids
THM-s	Tri Halo Methanes
TKN	Total Kjeldahl Nitrogen

TOC	Total Organic Carbon
TOD	Total Oxygen Demand
TOR	Terms of reference
TSS	Total Suspended Solids
UV	Ultra Violet
uPVC	Unplasticized polyvinyl chloride pipes
UV	Ultra violet light
UFW	Unaccounted For Water
VAT	Value Added Tax
WAS	Waste Activated Sludge
WFD	Water Framework Directive
WHO	World Health Organisation
WMMP	Water Management Master Plan of the Republic of Serbia
WT	Water Tank
WTP	(Drinking) Water Treatment Plant
WWTP	Wastewater Treatment Plant

5

TECHNICAL ANALYSIS

5.1 Potable water supply

5.1.1 Introduction

This section presents main objectives of the technical study of the potable water supply system at Vlasina. The main objectives of the water supply system technical study are set as follows:

- ⇨ To collect, supplement and systematize the inventory and all available data on the existing water supply system including data on the system layout, system components (water sources, transmission mains, pumping facilities, water storage facilities, water distribution network, etc.), water balances in the system, control and operation of the system;
- ⇨ To perform an analysis and corresponding assessment of the existing water supply system outlining its major deficiencies;
- ⇨ To study and analyse current urban development master plans and development projections in relation to planning of the future water supply system development;
- ⇨ To analyse and assess adequacy of the existing water sources in relation to available capacities, their water quality, sanitary protection and sustainability;
- ⇨ To propose inclusion of new water sources into the future water supply scheme in order to meet estimated potable water demand in the Project Area;
- ⇨ To devise and present a technical concept of the future water supply scheme in the Project Area including: water treatment facilities, transmission mains, water storage tanks, pumping stations and other components, as required to provide adequate level of service and reliable and safe potable water supply to the consumers in the Project Area;
- ⇨ The said concepts of the future water supply system development shall incorporate corresponding contemporary positive engineering practices and standards, especially these related to adequate water quality, sanitary and environmental protection;

5.1.2 Existing status

This section outlines the existing status of potable water supply in the project area.

In order to provide regular potable water supply for permanent population, visitors and tourist facilities, local water supply system has been developed and constructed. The system covers and supplies mostly users in Vlasina Rid area, and it is located on the outskirts of Cemernik, to the west from the Vlasina dam.

The system consists of the following main elements:

- Water intake called Plavilo

- Raw water pipeline from the intake to the water treatment plant, including break pressure tanks on the route
- Water treatment plant of 15 l/s nominal capacity
- Treated water storage tank of 500m^3 within the plant
- Treated water distribution network supplying Vlasina Rid area

The system has been constructed in accordance with the detail project design prepared by Hidrosanitas, 1983 (reference documentation No. 17). The original project design considers the aforesaid system components to be the first stage of planned development, the second stage being additional water intake called Cvejina dolina, additional treatment capacity of 15l/s, additional water storage capacity of 500m^3. Only the first stage of the water supply system based on the intake located approximately 1 km downstream the Plavilo spring has been realized.

Water intake Plavilo is located at app. 1.450 masl. In accordance with the measurements conducted for the purpose of the original design preparation, the following flows were recorded on Plavilo and Cvejina Dolina creeks:

Table 5.1: Minimal recorded discharges – Q (l/s)

Source name	08/ 1982	09/ 1982	10/ 1982	11/ 1982	08/ 1983	09/ 1983	10/ 1983	11/ 1983
Creek Plavilo	40	40	35	30	30	35	30	25
Creek Cvejina Dolina	12	13	12	9	10	10	8	7
Merged stream	75	80	70	60	65	75	65	55

These discharges are considered to have been recorded in extremely dry seasons, and were therefore assumed to represent the minimal operational capacities.

For the purpose of further planning the following minimal operational capacities, corresponding to month of August (the period of highest water demand), of considered sources were adopted:

- **Creek/intake Plavilo - Q_{min} = 30 l/s**
- **Creek/intake Cvejina dolina - Q_{min} = 10 l/s**
- ***Total - $Q_{min\ total}$ = 40 l/s***

Consequently, the aforesaid total minimal capacity has been selected as the design flow for appurtenant water transport and water treatment facilities.

With regard to water quality in the identified water sources, the analysis conducted for the purpose of the original design preparation showed that water quality generally corresponded to the first class waters, as defined in the Rulebook on classification of waters. The results of the physical and chemical analysis are shown in Annex 5.6.

The raw water pipeline operates by gravity and transports abstracted raw water to the water treatment plant.

Main technical features of the raw water pipeline from water intake Plavilo to the existing water treatment plant:

- Total available head - 1450 – 1299 masl – 151m
- Nominal diameter - DN200/L=1280m; DN150/L=1720m
- Pipeline length - 3.000 m
- Break pressure tanks at - 1430masl, 1+1280km; 1360masl, 2+180km
  1299masl, 2+990 km
- Design capacity - 30 l/s
- Pipe material - PVC, PN10

The raw water pipeline supplies a WTP, called Vlasina in this report, located in Vlasina Rid zone, as shown in the enclosed layout drawing.

Hydraulic calculation for the concerned raw water pipeline indicated that its design capacity is exceeded, while the actual operational capacity of the pipeline could be as much as 47 l/s – depending on the actual effective hydraulic roughness. However, since the pipeline has been in operation for more than 20 years, and it is also subject to variable raw water quality, in order to determine the actual operational capacity it would be necessary to conduct measurements of hydraulic parameters in the pipeline.

Figure 5.1: Hydraulic Profile of raw water pipeline Intake – WTP Vlasina (design capacity Q= 30 l/s)

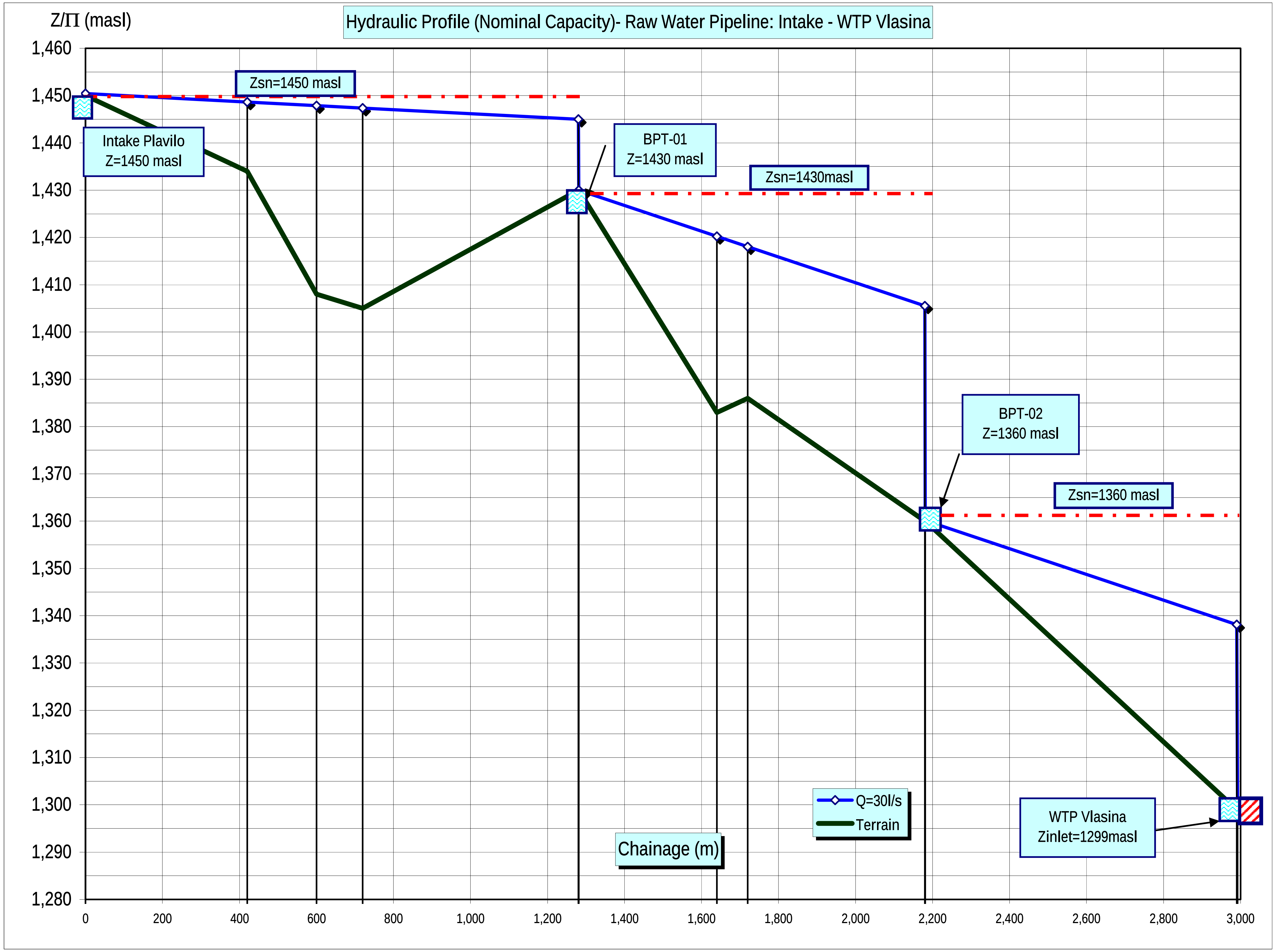
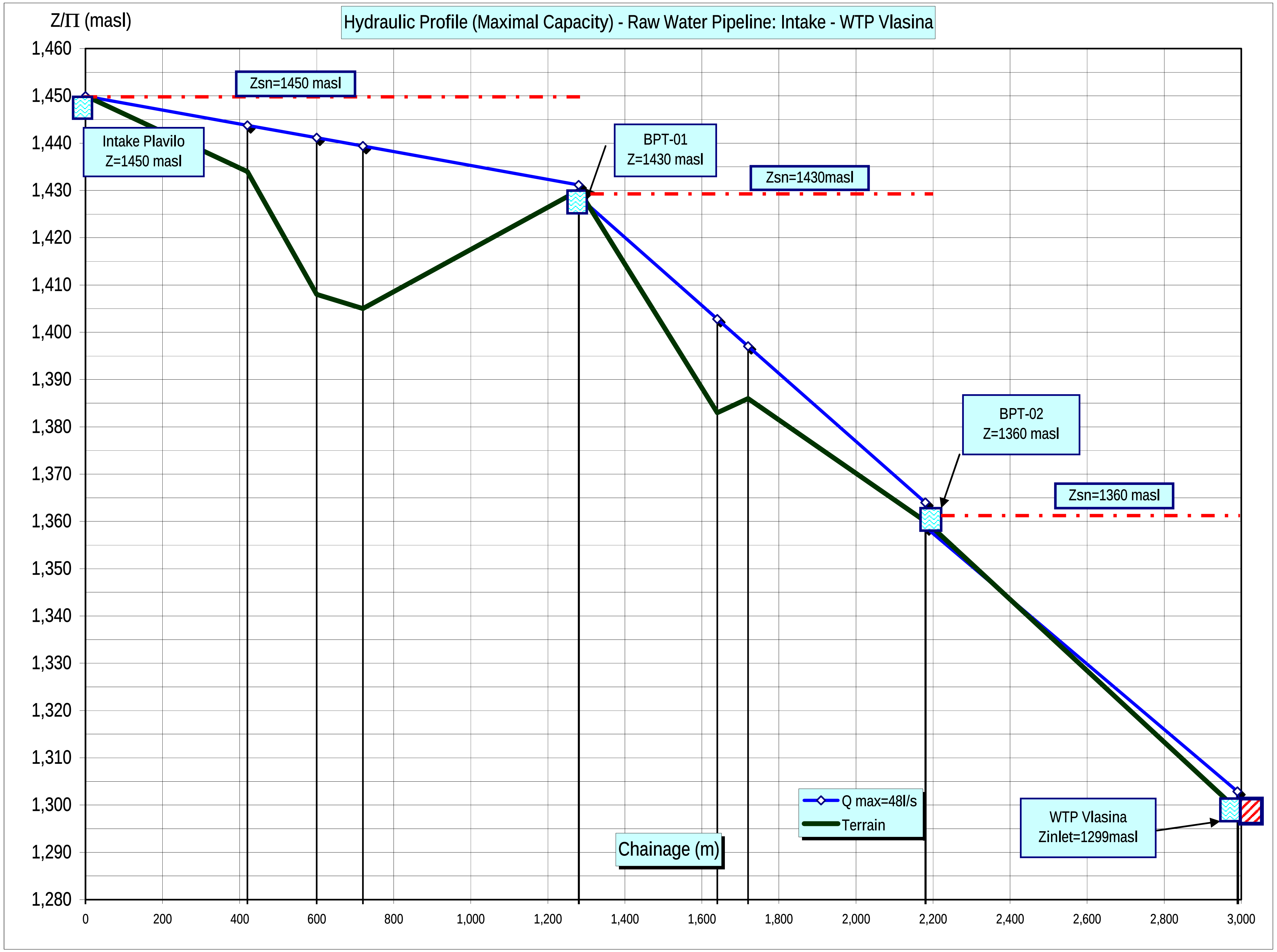


Figure 5.2: Hydraulic Profile of raw water pipeline Intake – WTP Vlasina (maximum capacity Q= 48 l/s)



Current operational capacity of the plant is 15 l/s. The plant has been designed for a phased development. The first phase of 15 l/s capacity has been already constructed, as well as the first phase water intake called Plavilo. The second phase would include additional capacity of 15 l/s as well as construction of the second water intake called Cvejina Dolina.

In accordance with the original design parameters, water quality data related to the abovementioned intakes corresponded to the first category water, as described in the Rulebook on Classification of Waters (OGRS 5/68). However, it was indicated that problems with increased water turbidity could be expected during and after intensive rainfalls.

This assumption proved to be correct, and abrupt increases of turbidity of raw water represents one of major operational problems of the WTP. Namely, as witnessed by the plant operators, the plant can not handle sudden surges of turbidity, and during these periods, the raw water inlet is closed, and the plant can provide only as much water as there is in its clear water storage. These phenomena have been recorded in a WTP logbook.

The plant layout includes a control room and filter building. Implemented process technology includes filtration at slow sand filters and compulsory disinfection by means of chlorination.

There are 3 slow sand filters, sized 5.0m x 10.10.0m (effective filtration area of each filter is estimated at 48 m^2).The filters consist of perforated collection pipes laid on filter bottom, supporting gravel layer, and finally 1 m deep fine filtering sand (0.25-0.35 mm).

These slow sand filters operate at filtration rate of 0.5 to 1.5 m/h, as a function of the plant operational capacity and number of filters in operation.

Along the filters, there is corresponding filter piping gallery containing inlet, outlet and so called first filtrate piping. Therefore, filtration rate for the plant design capacity is 0.5 m/h.

It is originally planned that one filtration cycle should last 30 to 50 days, and after that filters should be cleaned. However, due to the increased turbidity, filtration cycle is usually much shorter, and the plant has to be put out of operation. Based on the information by the PUC Vodovod Surdulica, the plant dominantly provides treated water compliant with the required potable water quality. However, rather frequent switching off the plant during storm episodes is not acceptable, because normal potable water is interrupted.

Figure 5.3: Water Treatment plant Vlasina

Figure 5.4: Slow sand filtration units and raw water inlet chamber

Figure 5.5: WTP Vlasina – Filters

Figure 5.6: WTP Vlasina – Filter box

The plant also includes a treated water storage tank of 500 m^3, minimum operational level being 1.292,50 masl, and the maximum operational level is 1.295 masl. The tank is connected to the main outlet pipe of DN200, which supplies consumers in the lower supply zone in Vlasina Rid area.

The plant is equipped with basic hydro-mechanical equipment, piping, including steel pipes and fittings, as well as conventional valves enabling normal filtration, first filtrate discharge, switching on or off filters, etc.

All of the abovementioned valves are manually operated, which means that the overall level of automation of the plant is at a low level, and all major operations are carried out manually. The piping and valving seem to be at the end of their operational lifetime, and should certainly be replaced within the scope of the WTP upgrade.

Figure 5.7: WTP Vlasina – Pipework **Figure 5.8: WTP Vlasina – Flowmeter**

Plant operational capacity is supposed to be recorded at a volumetric flowmeter. However, no information was given regarding the flowmeter reliability, calibration, and based on the site visit and actual status of the flowmeter, the recordings could be considered as indicative.

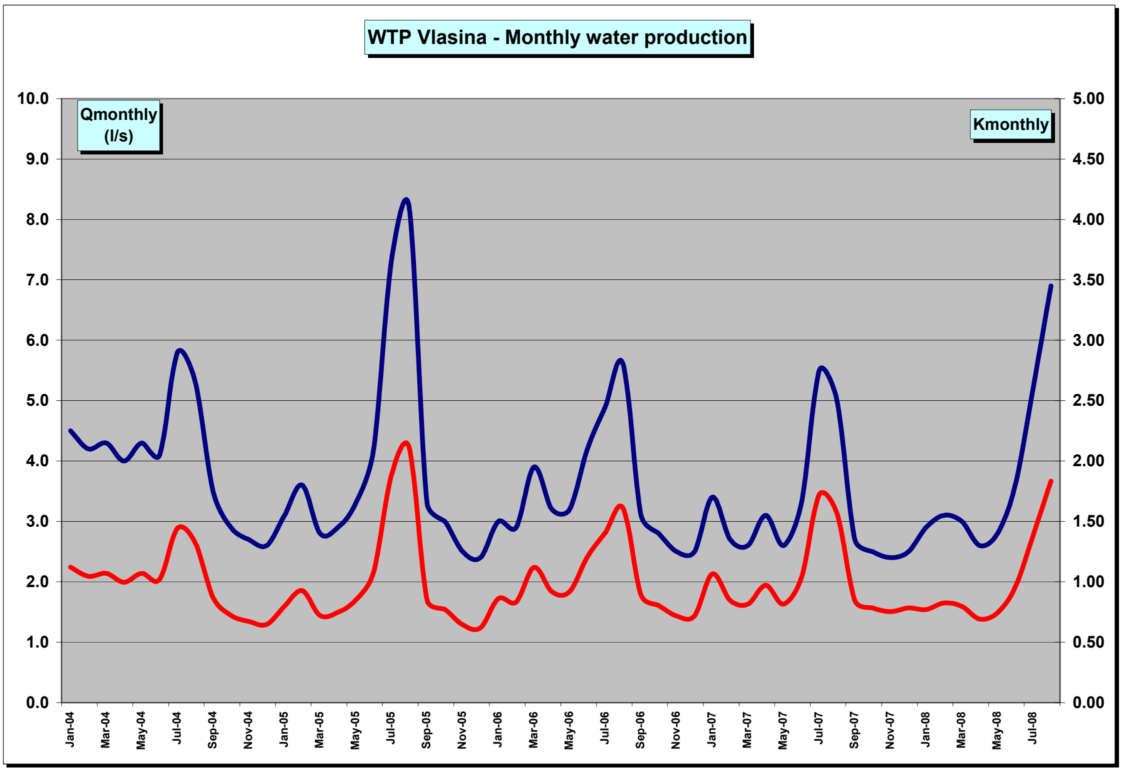
Based on the site visit it is also recommended to consider and implement substantial structural rehabilitation of the plant structures. Among other problems significant leakage from the raw water inlet / distribution chamber was observed that can endanger this unit and consequently operation of the whole plant.

Operation of the plant is regularly recorded in the plant log-book where all operations and results of significance (flows, quality, operational problems, maintenance, repairs) are included. Based on the logbook data, it was possible to generate daily water production, actually daily flows that were distributed in the network.

On the other hand, there are no records of raw water inflows coming from the intake, which would have been of importance to establish the intake operational capacities, especially their low range. However, flows records on the treated water side indicate consumption in the supply zone and its daily and monthly variations.

Recorded consumption patterns are presented in the following figure. The patterns show characteristic, very emphasized seasonal variations, with rather low steady demand in the wintertime and the peak summer demand reaching 1.5 to 2 times annual average. These patterns are typical for small, dominantly tourist areas, such as Vlasina, where number of system users increases sharply during the peak tourist season. These patterns are certain to continue in the future, with alleviating monthly variations as the supply zone increases, in accordance with the planned tourist development.

Figure 5.9: WTP Vlasina - monthly water production



From the treated water tank treated water is distributed to consumers in the Vlasina Rid zone (residential and holiday houses, tourist accommodation and restaurants). The main distribution pipeline is of DN200, while there is also a distribution network consisting of smaller pipes. These consumers belong to the lowest supply zone.

With regard to water supply of other settlements in the wider area of Vlasina lake, it is normally arranged by means of a numerous micro system, relaying on small, scattered springs, usually supplying individual households, or a group consisting of a few houses.

In order to improve general level of service in potable water supply, and to ensure reliable and adequate service and water quality control, it is recommend to centralize potable water supply system and equip it with contemporary treatment and distribution facilities.

5.1.3 Potential water sources in the area

5.1.3.1 Introduction

After the first reference to geology of the Vlasina valley (Boue, 1836), the area of the trans-boundary Serbian/Bulgarian river was not prospected until the late 19th century.

In 1893, J. Žujović prepared the first geological map and wrote textual explanation for the eastern part of the area. No geological prospecting was carried out until the thirties of the last century. M. Protić and associates produced in 1936/37 a litho-strati-graphic map of a larger area. Intermittent prospecting that followed between 1939 and 1957 focused mostly on petrology and mineralogy. B. Petrović was the first to explore in detail the northern part of the Vlasina area in 1965. From 1973 to 1977 followed prospecting for the Base Geologic Map, Sheets Vlasotince (K 34-35) and Trgovište-Radomir (K 34-57). More recently, from 1986 to 1994, Z. Tasić et al. prospected the area for the thematic Geologic Map of Surdulica Massif.

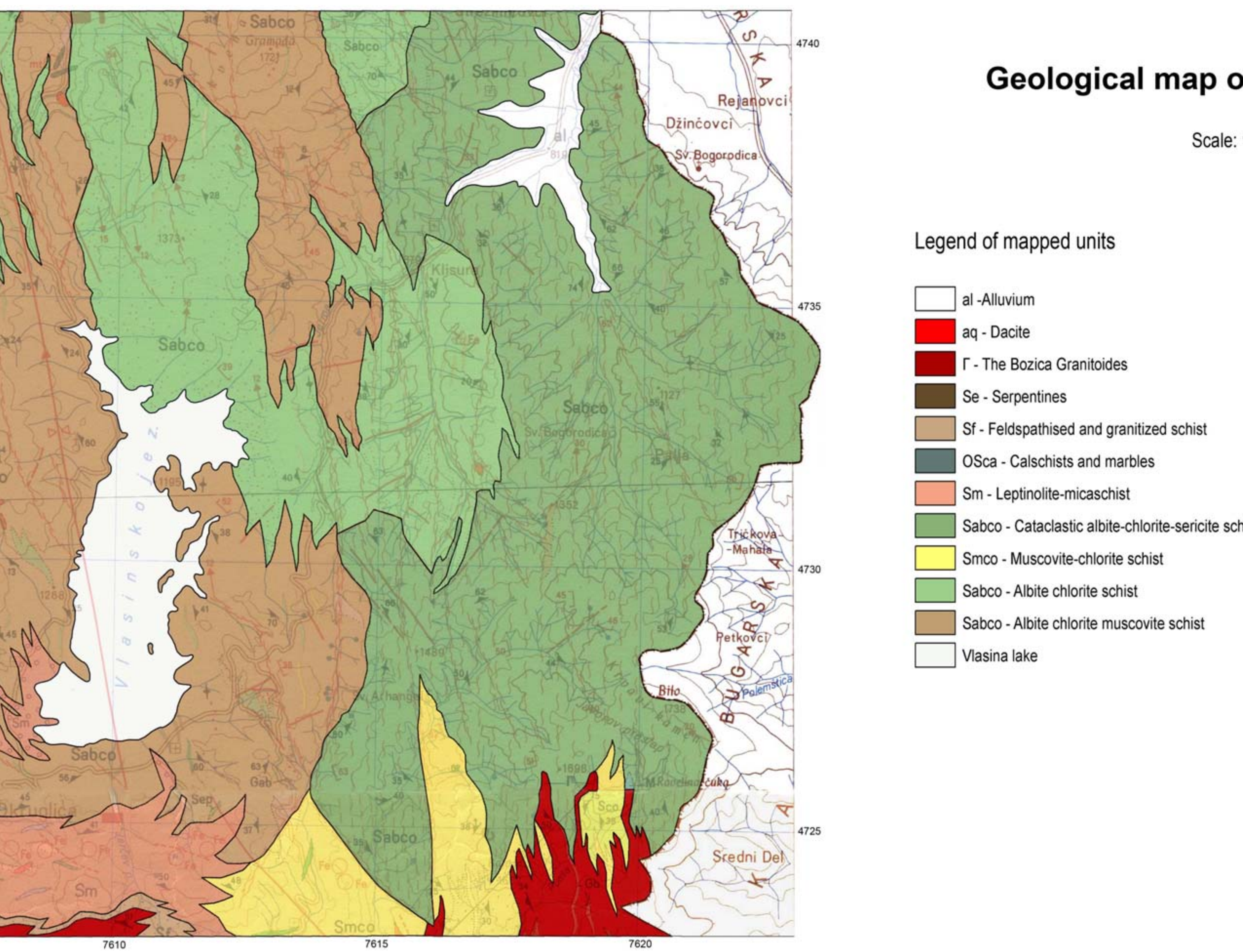
Geology of the Vlasina Lake area was well explored in the past.

5.1.3.2 Geological Setting

The Vlasina area is a part of the large geotectonic unit Serbian-Macedonian Massif, designated as the Vlasina Complex in geologic literature.

Figure 5.10 shows distribution of the mapped units in the Vlasina area.

Figure 5.10: Geological map of the Vlasina Lake area (BGM, Sheets Vlasotince K 34-45 and Trgovište-Radomir K 34-57)



Mapped Units

Paleozoic (Pz)

Oldest in the area are also dominant rocks of the Vlasina Complex. These are regionally and progressively metamorphosed schists of Cambrian age. The complex of rocks is a sedimentary-volcanic formation metamorphosed under the conditions of the greenschist facies. General characteristics of the complex are the following:

- Vertical and horizontal rock variation in a succession of primary clay, sand, marl and other deposits and basic igneous rocks;
- Intensive mineralization stable under the greenschist facies conditions; and
- Presence of albite from primary rocks.

Regional metamorphic rocks are a group of muscovite-chlorite schists and a group of 'green' rocks, including mica-chlorite (Sm), chlorite-epidote and amphibole rocks, then quartzite (q) and cataclastic albite-chlorite-muscovite schists (Sabco).

Progressively metamorphosed rocks are the most widespread unit of the metamorphic complex, which includes mica schist and leptynolite (Sm), migmatite, commonly in small masses, lenses or interbeds.

Minor masses of foliate Božica granitic rocks (G) intruded during the Paleozoic are conformable with the Vlasina Complex. These granitic rocks consist of quartzite, feldspar, mica, chlorite and less of mica group minerals, distributed SE in the area.

Cenozoic (Kz)

Strong Tertiary magmatism involved SW of the area when Surdulica granodiorite massif was intruded, described by M. Ilić (1977) as belonging to the rocks of synorogenic magmatism. Rocks of the granodiorite massif, different in the orthoclase/plagioclase ratios, are the following varieties: granodiorite, monzogranite and quartz diorite. Molten granodiorite intruded fault zones in NW-SE and NE-SW strike directions forming dykes thick between 10 and 150 metres. Vein accessory minerals are granodiorite porphyrite, quartz diorite porphyrite, aplite and lamprophyre.

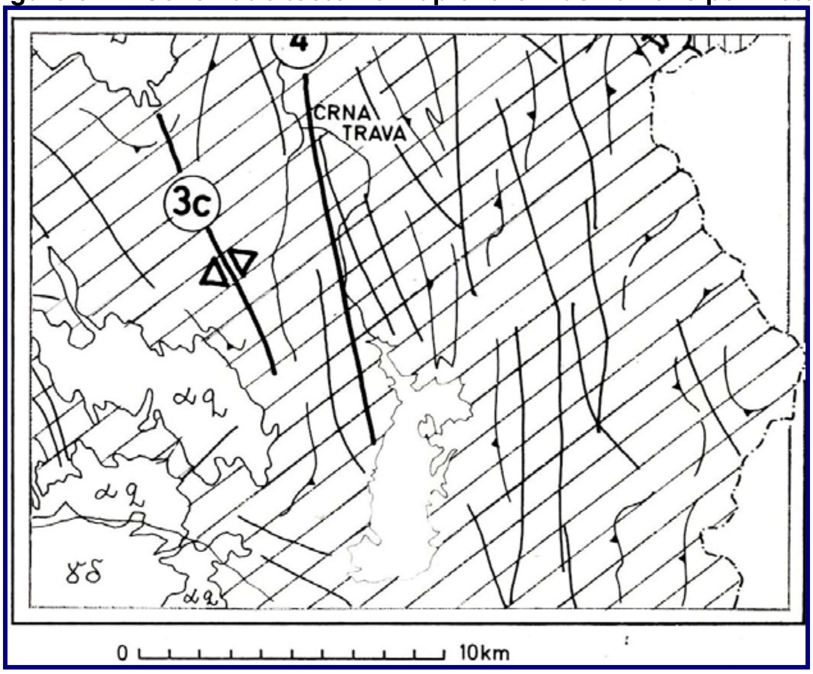
Effusive rocks produced during the Tertiary magmatism built up Čemernik composed of propilitised and less commonly zeolitised dacite (aq) and andesite. These rocks are chemically similar with granodirite rocks of the Surdulica massif.

Quaternary sedimentary rocks are genetically associated with the landsurface shaping events, or with the process of fluvial erosion. These rocks are products of alluvial (al), deluvial-proluvial and colluvial deposits along the Vlasina, Vrla, Čemernica, Jerma rivers and minor streams and around the lake.

Structure

The structure consequent upon deformational processes in this area of the Trgovište-Božice tectonic unit (Dimitrijević, 1995) controlled the formation of structural terraces in different epochs. Figure 5.11 shows a schematic tectonic map.

Figure 5.11: Schematic tectonic map of the Vlasina Lake perimeter



Crystalline schist of the Vlasina Complex forms the first, lowest, structural terrace. Tectonic features homogeneous in the deformation fabric within the terrace are the following:

- Čemernik anticlinorium extends along the central mountain ridge and is bounded by a longitudinal dislocation zone on either side. The width of the anticlinorium varies between 10 and 15 kilometres.
- Vlasina syncline east of the Čemernik anticlinorium has the synclinal bend generally coincident with the Vlasina River course. The syncline width is variable, 10 km in the south, and the length more than 40 km.
- Tumbe, east of the Vlasina River, is a thrust front. Many longitudinal dislocations in the Tumbe area make it most deformed feature in the Vlasina Lake perimeter. The symmetry of the feature is monoclinal and the folds are isoclinal high-indexed and clearly east-vergent (Petrović et al., 1973).
- Transitional Ljubata area, along the eastern margin of the Surdulica granodiorite massif, is a series made up of longitudinal beds gently dipping to the NNE.

Granite intrusions of Božice are part of the first structural terrace.

The Surdulica Massif belongs to the fourth structural terrace. Neotectonic events from the Lower Miocene to the present day destroyed the old alpine structures and left the recent pattern of the massif and the surrounding area. The new pattern includes two distinct regional fault zones of decakilometric size and NW-SE and N-S strike directions. Regional faults in NW-SE strike direction form a parallel system of faults.

Mineral Resources

There are no mineral deposits worked in the Vlasina Lake area. Mineral raw materials in the Lake perimeter for the local or possibly wider use are the following:

1) A small deposit of elevated graphite concentration at Donja Ljubata.
2) Small masses of dacite andesite associated with sulphide minerals in fault zones along the Klisura-Preslot line.
3) Dacite in the Vrla valley excavated for regional production of pavement-stone cubes.
4) Peat was dug on the Vlasina Lake margin and treated in a plant at Blizinci until the area has been nominated protected for its nature value.
5) Low-mineral ground water partly intercepted and bottled is a major resource, generally at lower elevations than the Plan area.

5.1.3.3 Hydrogeology

The Vlasina Lake area was hydro-geologically prospected only within the Vlasina River basin.

The area was investigated in detail in 1946 for the Vrla Hydroelectric Power Plant project. The study 'General Water Resources Development Plan for the Vlasina Basin' (1978) described hydrogeology, gave a list of water structures and included a hydro-geologic map at scale 1:200,000.

The Hydro-meteorological Institute of Serbia prepared for the quartermaster corps of Yugoslavia (1979) an inventory of hydro-geologic occurrences map at scale 1:50,000.

More recently, low-mineral water of Veliki Strešer was prospected, intercepted and bottled for commercial use under the name Rosa. The hydrogeology department of the Geozavod researched from 2000 the feasibility of thermal water and energy utilization for Surdulica.

Hydro-geologic Properties of Rocks and Types of Aquifers

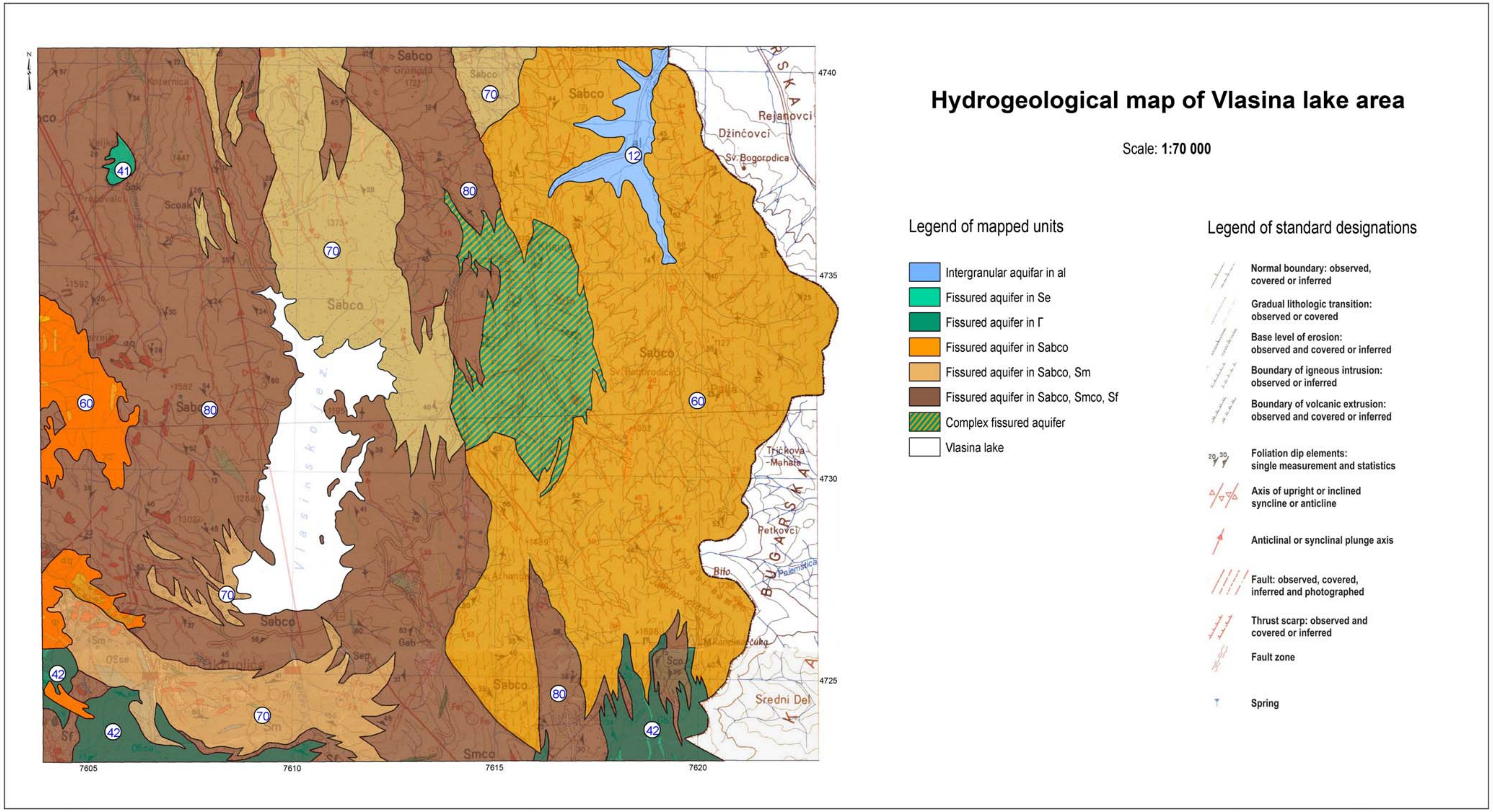
Rocks that build up the Vlasina Lake area are dominantly fractured, deformed, differentially weathered and characterized by some other properties as well. With respect to hydraulic properties and horizontal or vertical distribution, the lithologic units identified are the following:

- dense-rock aquifer of alluvial, eluvial and deluvial-proluvial deposits;
- fractured-rock aquifer of crystalline schist varieties, or igneous rocks; and
- combination of the former two aquifers in fault-zone areas.

Provisionally designated 'waterless' terrains are individual mapped units of the Paleozoic schist.

The distribution of lithologic members and respective hydro-geologic formations are shown in Figure 5.12.

Figure 5.12: Hydro-geological map of the Vlasina Lake area



Dense-rock aquifer

Alluvial, eluvial or deluvial-proluvial deposits that form dense-rock aquifers have limited distribution.

Alluvial aquifers of this type extend along minor streams (Vrla, Manćina, and Simanova Reka) and are more abundant in the Jerma valley.

Alluvial deposits form along streams of variable hydraulic gradient and thereby are heterogeneous in granulation and thickness depending on the distance from the stream source. In upper reaches of the streams, rocks are around 2 m thick and contain many clay particles and a low proportion of gravel. Average hydraulic conductivity for the whole Vlasina basin varies from 10^{-3} to 10^{-5} cm/s (Čolić, 1978).

Swift current carried and deposited rock waste in a depression where the Vlasina Lake is now. These rocks crop out in the SW, S and E parts of the Vlasina Lake, at the Murina debouchment, clay-supported about 10 m thick at Promaja and 5-6 m at the Šaovica mouth.

Atmospheric precipitation and surface streams, and groundwater from higher grounds, recharge dense-rock aquifers. Groundwater seeps from these aquifers to weak springs and surface streams.

Hydrodynamic pressure in the mentioned deposits suggests commonly unconfined groundwater.

Fractured-rock aquifer

Metamorphic rocks of the Paleozoic Complex, and subordinately igneous rocks of the Surdulica Massif and Božica granitoid, are most extensive aquifers of this type.

Schist and schist varieties of the Paleozoic Complex form a fractured-rock aquifer, though referred to as 'waterless' in some published sources. Because the fracture density decreases with the depth, rocks shallower than 15 m are baring water. Clay-sand rock waste commonly fills cracks or small fractures. Jovanović and Purić (1972) estimate the transmission capacity (T) of the rock complex to be 10^{-5} m^2/s.

Groundwater seeps from the aquifer to springs and flows out at a rate from only 0.1 to 0.5 l/s. Most springs are perennial, with variable flow rates. Total discharge of groundwater is not as small as the above may suggest. Cement channels conduct the intercepted water to the Vlasina Lake. The two channels drain 50 l/s each (Čolić, 1978).

Igneous rocks, dacite and granodiorite, of Tertiary age are also fractured-rock aquifers. Hydrogeologically, these either are the older, schistose and hydrothermally altered, or newer, exogenic rocks.

Sources of groundwater recharge in the fractured rocks above the base level of erosion are atmospheric and surface waters, and groundwater from other aquifers (alluvium and fractured schist). General directions of groundwater flow, controlled by the distribution and position of cracks and fractures, are NW-SE and N-S, the strike directions of regional faults.

Groundwater flows from the fractured-rock aquifers to springs, or seeps into bogs (where granodiorite grus prevails), or is pumped out. The springs are mostly medium to weak, commonly 0.1 to 0.5 l/s, or stronger (to 5 l/s) where fed from aquifer depth. Groundwater generally flows uncontrolled, discharged by gravity springs, contact springs, or few rising springs. Major springs are Vukanov Vir (0.5 l/s) in Jančina Mahala, a spring in Veličovi (1.5 l/s), white-water spring on Čemernik (2 l/s), and Ignjatova Česma on Cvejin Senjak (1.5 l /s).

The ground waters are odourless, tasteless, colourless and clear, with temperature between 7º and 14ºC. Hydrogen ion variation range is from 6.4 to 7.95. Mineral concentrations in groundwater are low. Classified on mega-ions, these are hydrocarbonate-calcium or hydrocarbonate-sodium waters.

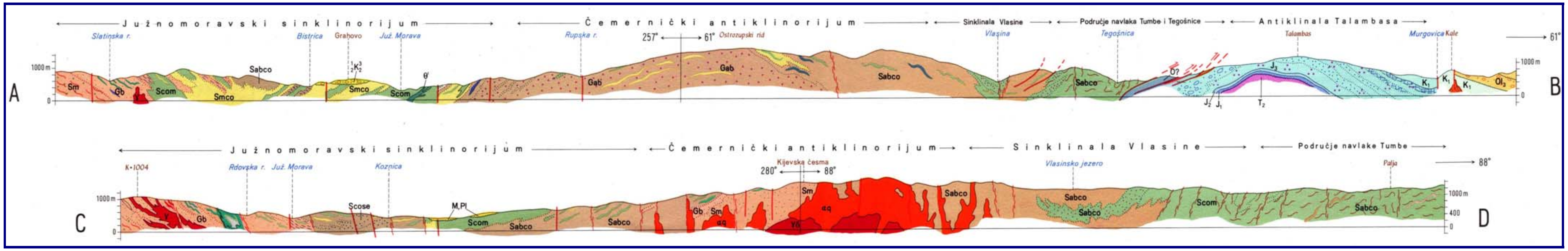
Because hydrogeological information is scarce for the area, it is cited that “Except the mentioned major resources, water bodies in the extensive fractured rocks have not been identified for quantification. However, with regard for the low degree of research on one hand, and knowledge of similar terrains on the other, opening of workings and production of 50 l/s or more water are feasible” (Filipović, Krunić, Lazić, 2005, p.180).

Combined aquifer in fault-zone area

Paleozoic rocks in which tectonic events formed a fault zone contain more groundwater in a nameless stream basin above the Gorge than in other similar terrains.

This aquifer feeds from atmospheric and surface waters and from dense-rock aquifers filling the faults.

Figure 5.13: Hydro-geologic section



Provisionally 'waterless' terrains

Designated 'waterless' are the grounds made up of Paleozoic sandstone, schist and quartzite, which are fractured but do not continuously or abundantly conduct ground water.

Water bodies intermittently form in the ground subsurface, where dense Paleozoic or Triassic fractured rocks are fed from deluvial deposits. Intermittent weak springs or seeping groundwater marks these zones of no particular significance.

5.1.3.4 Constraints and Problems

Any use of ground as an integral part of the environment, or use of the geological heritage and groundwater, is inevitably under constrain and causes problems, which are addressed through planning and economical management. For this area, the considerations are the following:

- Remediation of slope instability or soil stabilization;
- Arrangement for torrential stream regulation;
- Coordination of water supply system extension and increasing water demand; and
- Mining concessions for deposits or parts thereof, or for groundwater abstraction.

5.1.3.5 Guidelines

Priority should be given to detail hydro-geological research including:

- Planning and sampling groundwater in prospective zones;
- Building a fixed rectangular spillway in suitable place of the stream, and selection of new points for continuous gauging;
- Design, setting and location of a fixed rain-gauging station;
- Hydro-geological prospecting for a hydro-geologic plan (scale 1:1000) including punctual assay of physical and chemical parameters of any water occurrence at the source over the hydraulic cycle;
- Estimate of hydro-geological parameters for full determination of hydro-geologic regime of the given water source;
- Assessment of hydro-geological environment vulnerability that restrict the planned land use.

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5.1.4 Water Demand Projection

Water demand estimate in this study covered the Project Area including the following:

1. Tourist Centre Vlasina – as defined in the Spatial Plan of the Special Purpose Area, Spatial Plan of Surdulica Municipality and Master Plan of Tourism Development
2. The two hamlets of Klisura and Božica, respectively at the north-eastern and the south-eastern side of the lake, both along a ‘major’ road to Bulgaria, are historically the two most important settlements of the area.

Water demand estimate in this study is based on the following:

1. Number of permanent and temporary residents in the Project Area defined by Surdulica Municipality;
2. Spatial Plan of Surdulica Municipality, reference documentation 11;
3. Spatial Plan of Spatial Purpose Area Vlasina, reference documentation 12;
4. Master plan with business plan of tourism development at Vlasina Lake, reference documentation 13;
5. Chapter on Tourism Strategy for the Project Area in this Study

With regard the estimated implementation schedule of planned tourist development in the Project Area, the following milestones/phases have been defined:

1. Phase I – till 2015 – User count in accordance with the proposed Tourism Strategy – Scenario 1;

2. Phase II – till 2025 – User count in accordance with the proposed Tourism Strategy – Scenario 2;
3. Phase III – till 2035 – generally in accordance with reference documentation 11, 12 and 13

Apart from the total number of users per category it is also important to define corresponding occupancy rates in order to be able to calculate real average annual water demand. The occupancy rates are defined in chapter on planned Tourism Strategy.

Here are the major parameters incorporated in the potable water demand projection:

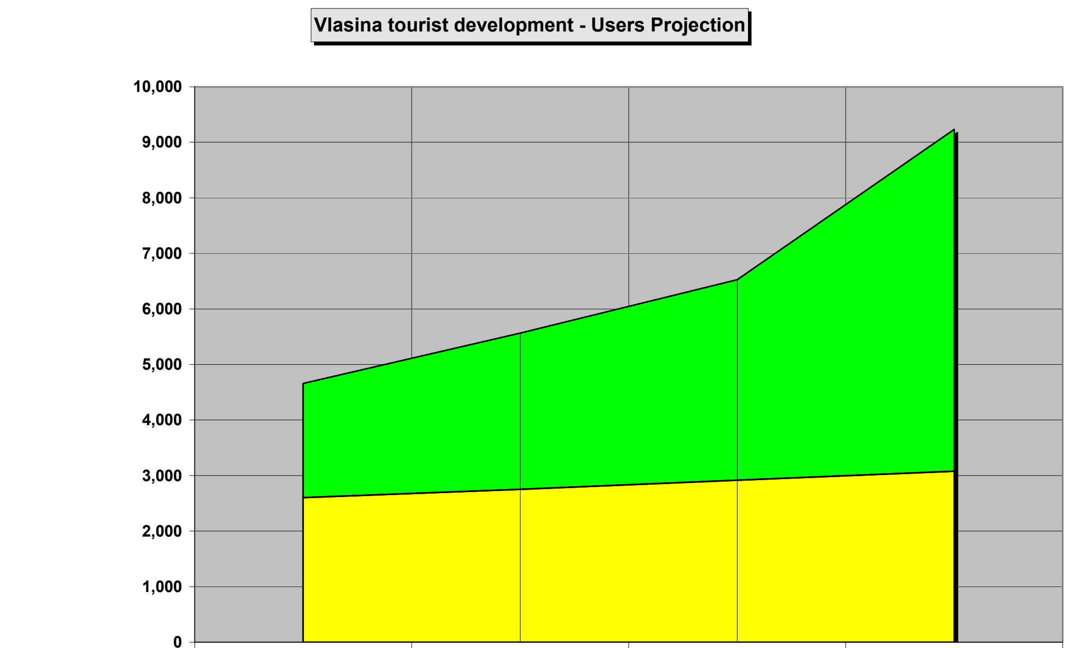
1. Average unit loading rates are adopted as follows: for households 150 l/capita/day, for users in tourist accommodation 300 l/capita/day;
2. Maximum day factor has been assumed based on the recorded consumption as well as in accordance with usual characteristics of similar systems – $K_{max\ day} = 1.5$.
3. Water losses are calculated as for new, well-maintained systems, and are assumed to be 20% of gross water production;
4. The peak hour factor has been derived based on the size of an individual supply sub-zone;
5. The peak tourist accommodation occupancy rate is assumed to be 100%, while the average annual occupancy rate is assumed to vary from 30 to 60%.

The following tables and figures provide an overview of estimated potable water demand in the Project area for different design horizons; 2015, 2025 and 2035.

Table 5.2: Potable Water Demand Projection for Vlasina tourism development

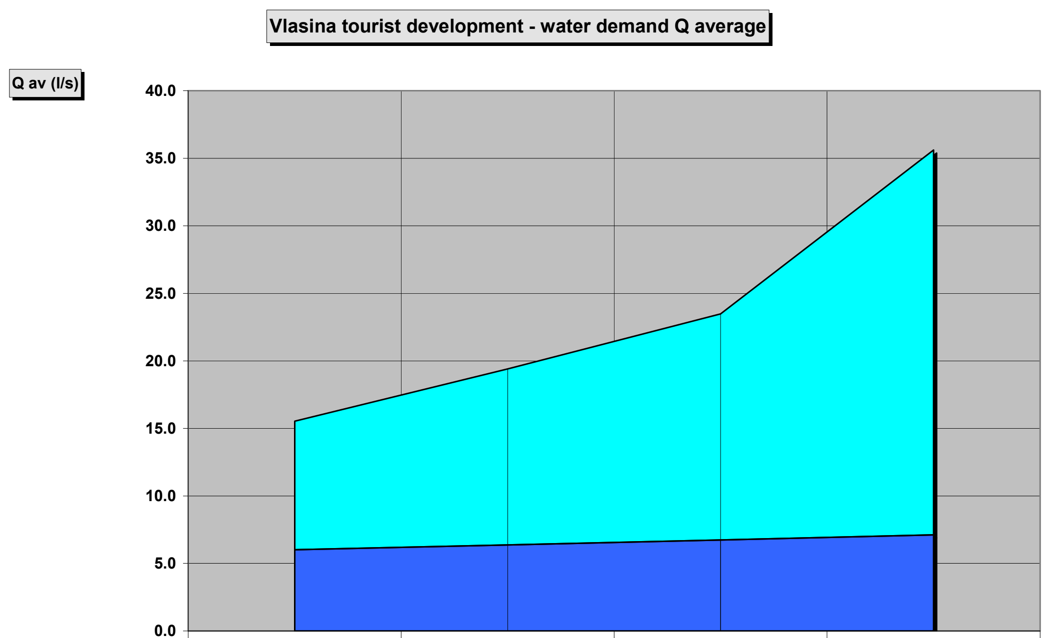
Vlasina Resorts & Villages	User count				Unit loading rate (l/cap/day)	$Q_{average\ gross}$ (l/s)				$K_{max\ day}$	$Q_{max\ day\ gross}$ (l/s)				K_h				$Q_{peak\ hour}$ (l/s)			
	2006	2015	2025	2035		2006	2015	2025	2035		2006	2015	2025	2035	2006	2015	2025	2035	2006	2015	2025	2035
Tourist Resorts																						
Vlasina Rid	1,715	2,025	2,425	4,815	300	7.9	9.4	11.2	22.3	1.5	10.9	12.9	15.4	30.7	2.1	2.0	2.0	1.8	20.5	23.7	27.9	51.4
Vlasina Okruglica	180	480	880	880	300	0.8	2.2	4.1	4.1	1.5	1.1	3.1	5.6	5.6	2.7	2.4	2.2	2.2	2.7	6.6	11.3	11.3
Vlasina Stojkoviceva	160	310	310	460	300	0.7	1.4	1.4	2.1	1.5	1.0	2.0	2.0	2.9	2.7	2.5	2.5	2.4	2.5	4.5	4.5	6.3
Tourist Resorts - total	**2,055**	**2,815**	**3,615**	**6,155**		**9.5**	**13.0**	**16.7**	**28.5**		**13.1**	**17.9**	**23.0**	**39.2**	**2.0**	**1.9**	**1.9**	**1.8**	**24.0**	**31.8**	**39.8**	**64.0**
Villages																						
Vlasina Rid	1,493	1,539	1,589	1,640	150	3.5	3.6	3.7	3.8	1.5	4.8	4.9	5.1	5.2	2.3	2.3	2.3	2.3	9.8	10.0	10.3	10.6
Vlasina Okruglica	478	531	589	648	150	1.1	1.2	1.4	1.5	1.5	1.5	1.7	1.9	2.1	2.6	2.6	2.6	2.5	3.5	3.9	4.3	4.6
Vlasina Stojkoviceva	632	681	736	790	150	1.5	1.6	1.7	1.8	1.5	2.0	2.2	2.3	2.5	2.5	2.5	2.5	2.5	4.5	4.8	5.2	5.5
Total-Villages	**2,603**	**2,750**	**2,914**	**3,078**		**6.0**	**6.4**	**6.7**	**7.1**		**8.3**	**8.8**	**9.3**	**9.8**	**2.1**	**2.1**	**2.1**	**2.1**	**16.0**	**16.8**	**17.7**	**18.6**
Total Tourist Resorts and Villages	**4,658**	**5,565**	**6,529**	**9,233**		**15.5**	**19.4**	**23.5**	**35.6**		**21.4**	**26.7**	**32.3**	**49.0**	**1.9**	**1.9**	**1.8**	**1.7**	**37.2**	**45.4**	**53.8**	**78.1**
Božica-population	350	350	350	350	150	0.8	0.8	0.8	0.8	1.5	1.1	1.1	1.1	1.1	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
Božica-visitors	0	100	100	100	300	0.0	0.5	0.5	0.5	1.5	0.0	0.6	0.6	0.6	0.0	2.9	2.9	2.9	0.0	1.6	1.6	1.6
Božica-total	**350**	**450**	**450**	**450**		**0.8**	**1.3**	**1.3**	**1.3**		**1.1**	**1.8**	**1.8**	**1.8**	**2.7**	**2.6**	**2.6**	**2.6**	**2.7**	**4.0**	**4.0**	**4.0**
Klisura-population	350	350	350	350	150	0.8	0.8	0.8	0.8	1.5	1.1	1.1	1.1	1.1	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
Klisura-visitors	0	100	100	100	300	0.0	0.5	0.5	0.5	1.5	0.0	0.6	0.6	0.6	0.0	2.9	2.9	2.9	0.0	1.6	1.6	1.6
Klisura-total	**350**	**450**	**450**	**450**		**0.8**	**1.3**	**1.3**	**1.3**	1.5	**1.1**	**1.8**	**1.8**	**1.8**	**2.7**	**2.6**	**2.6**	**2.6**	**2.7**	**4.0**	**4.0**	**4.0**

Figure 5.14: Vlasina Tourism Development – Projected Number of Users



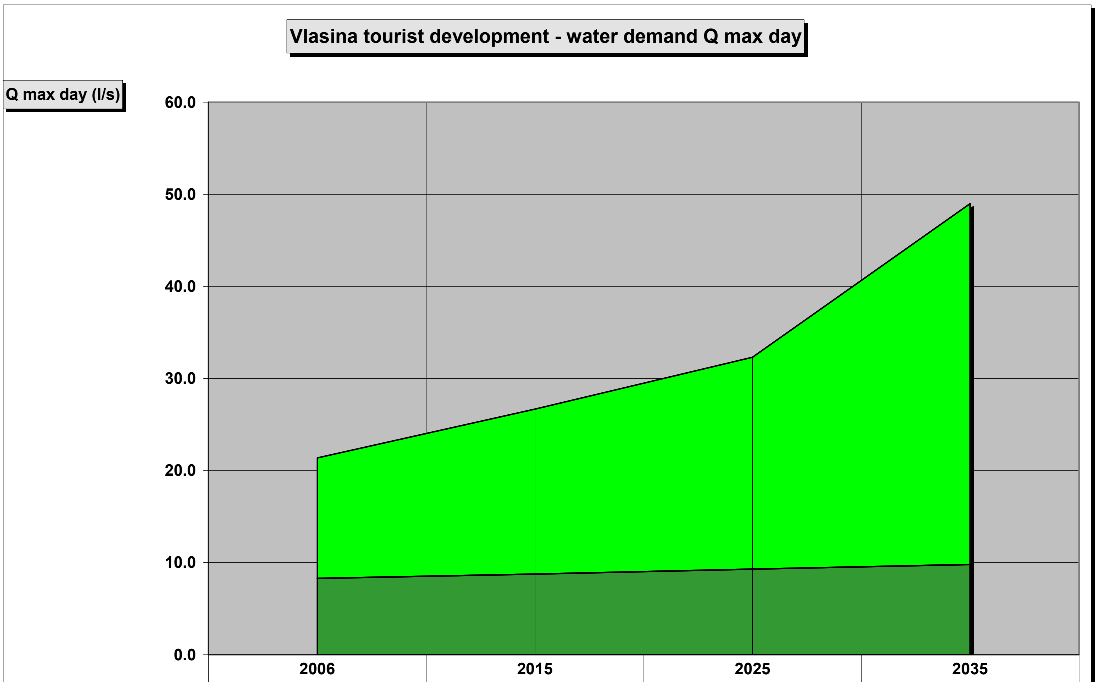
	2006	2015	2025	2035
Tourist Resorts - total	2,055	2,815	3,615	6,155
Total-Villages	2,603	2,750	2,914	3,078

Figure 5.15: Vlasina Tourism Development – Average Water Demand



	2006	2015	2025	2035
Tourist Resorts - total	9.5	13.0	16.7	28.5
Total-Villages	6.0	6.4	6.7	7.1

Figure 5.16: Vlasina Tourism Development – Maximum Daily Water Demand



	2006	2015	2025	2035
Tourist Resorts - total	13.1	17.9	23.0	39.2
Total-Villages	8.3	8.8	9.3	9.8

The maximum daily water demand that is normally used as a key design parameter for a number of water supply elements is defined as follows:

- For settlements and tourism developments around Vlasina lake (Vlasina Rid, Okruglica and Stojkoviceva) – Q $_{max\ day}$ = 49 l/s at the end of the design horizon;
- For rural centres, Klisura and Bozica – Q $_{max\ day}$ = 1.8 l/s for each settlement;

The aforesaid flows are used for sizing of raw water intakes, transport, water treatment and storage facilities, while the peak hourly flows are representative for sizing of the water distribution network.

5.1.5 Raw water transport and treatment

This section presents options that were considered in this study in relation to provision of sufficient quantities of raw water, raw water transport and treatment. Presented options of raw water transport and treatment include raw water intakes, transportation up to water treatment facilities, and water treatment facilities. Corresponding water distribution network is considered common for all options considered and it is elaborated in the following sections.

The following main options were considered:

1. **Option 1:**
    a. Connecting additional water sources west of the existing WTP Vlasina – towards Cemernik
    b. Continual use of the existing WTP Vlasina, with extended capacity up to 30l/s;
    c. Activation of a new raw water intake at the Grubina River, next to Klisura settlement of 20 l/s capacity, transportation to the future WTP Jerma, located some 1.400m west of the main road M 1.13 (Vladicin Han – Surdulica – Klisura – Strezimirovci);
    d. Therefore, supply of consumers would be based on two groups of sources (Cemernik, Grubina reka), i.e. two corresponding treatment plants (Vlasina and Jerma);
2. **Option 2:**
    a. Connecting additional water sources west of the existing WTP Vlasina – towards Cemernik
    b. Continual use of the existing WTP Vlasina, with extended capacity up to 30l/s;
    c. Activation of a new raw water intake from Jerma canal with a capacity of 20l/s, transportation and treatment at the WTP Jerma as described in option 1;
    d. Similarly to alternative 1, consumers would be supplied from two main sources (Cemernik, Jerma canal) and two treatment plants.
3. **Option 3;**
    a. Usage of the existing and additional springs at the outskirts of Cemernik, upgrade and extension of the existing WTP for the capacity of 50l/s;
    b. If required, supplementary raw water supply from the Vlasina lake to supplement raw water capacity up to 50l/s.

c. Raw water would be provided from two water sources (Cemernik, supplementary from Vlasina lake), while there would be a single water treatment plant at the location of the existing WTP.

5.1.5.1 Option 1

This option is based on the findings presented in the hydro-geological report. Namely, as shown in the hydro-geological map of the project area, a complex fissured aquifer south of Klisura settlement (basically it includes the catchment of Grubina River with all belonging springs) has been identified as a water source of considerable capacity, which can provide potable water supply for the project area.

It has to be noted that this potential water source has been neither investigated in terms of representative flows, nor from the prospective of water quality. Additional hydro-geological and hydrological investigations of this catchment area are therefore compulsory in the following phases of project preparation in order to establish suitability of this water source for its planned purpose.

However, for the purpose of preparation of this study, it was considered necessary to arrange for at least a single water quality sampling and analysis that would indicate water quality at the source. This water quality analysis has been carried out by the representatives of the Institute for Public Health in Vranje, and supervised by the MISP program and local PUC representatives. The results of the analysis are presented in Annex 5.7.

Basic concept of the water treatment and transportation option No. 1 includes the following:

- **Construction of a water intake at the Grubina river, just upstream of Klisura settlement;**
- **Provision of a raw water pumping station next to the intake**
- **Raw water transmission system from the raw water pumping station to a future water treatment plant located in Vlasina Stojkovica zone**
- **Potable water treatment plant that would ensure water quality, as required in accordance with the potable water regulations;**

Water Intake

Basic characteristics of the water intake:

- Type - directly from the water course, Tyrolian type
- Altitude - 950 masl
- Estimated capacity - 20 l/s (to be verified by additional hydro-geological investigations)

This type of intake belongs to bottom intake types. It includes three basic elements:

- Intake block
- Spillway
- Settling basin

The Tyrolian weir is suited for streams carrying coarse sediments with particles smaller than 5mm amounting to less of 25% of the total sediment.The inlet block and the spillway for a massive barrier across the stream.The intake block is in particular subject to abrasion by sediment movement and its design and construction must allow for adequate protection. The spillway crest elevation should be determined to allow for downstream requirements, including guaranteed minimal flow, water demand, etc. The collection gallery below the screen diverts abstracted water into the settling basin. The settling basin is located on the stream bank and should containg all solid matter that passed the screen. Sediment in the settling basin is periodically or contionously flushed back into the stream.

Raw water pumping station

The raw water pumping station would be located just off the intake and has got the following main characteristics:

- Nominal capacity – total - 12 l/s
- No. of pumps - 2+1 (2 duty, 1 stand-by)
- Type - multi-stage centrifugal
- Nominal capacity of pump - 12 l/s
- Nominal head - 105 m
- Motor power - 18.5 kW
- Type of regulation - frequency-drive controlled
- Mode of operation - flow-controlled

Operation of the raw water pumping station would be flow-controlled in relation to actual demand, but also should take into account downstream requirements and guaranteed minimal flow.

Transmission system

Transmission system provides raw water transfer from the intake to the WTP. Main features of the transmission system are shown in the following table:

Table 5.3: Main Features of raw water transmission system - Klisura

Pumping stations				
Name	**No. of pumps**	**Q (l/s)**	**H (m)**	**P (kW)**
Raw water PS “Klisura”	2+1	12	105	18.5
Transfer PS “Klisura-01”	2+1	12	145	30
Tanks/Break pressure tanks				
Name	**Volume**	**Minimum level (masl)**	**Maximum level (masl)**	**Note**
BPT “Klisura-01”	100	1150	1153	Suction basin for transfer PS “Klisura-01”
Pipelines				
Start section	**End section**	**DN (m)**	**L (m)**	**Recommended material**
Raw water PS “Klisura”	BPT “Klisura-01”	150	2100	PN16/25 HDPE/ST/DI
Transfer PS “Klisura-01”	WTP “Jerma”	150	2300	PN16/25 HDPE/ST/DI

It is envisaged that the aforesaid pumping station operate on a constant-flow basis, preferably contolled by frequancy drives. Also a remote control and regulation system should be implemented in order to ensure optimal usage of this sub-system elements and good energy efficiency.

Water Treatment Plans

Option 1 proposes production of drinking water at two different locations and from two different raw water sources including:

- The upgraded existing WTP Vlasina that will make use of raw water from the Plavilo Creek (existing intake) and Cvejina Dolina (new) with maximal capacity of 30 l/s
- The new WTP Jerma that will treat raw water from the Grubina Reka with maximal capacity of 20 l/s.

WTP Jerma

Potable water treatment plant (WTP) called Jerma would be located some 1400m west of the of the main road M 1.13 (Vladicin Han – Surdulica – Klisura – Strezimirovci). It is also located conveniantly close to the canal called Jerma, which normally feeds Vlasina lake.

In the option 1, the WTP is to be supplied from the water intake Klisura, as described eariler. The abovementioned raw water transport system should ensure stable, nearly constant feeding of raw water to the WTP in order to provide optimal conditions for its operation.

The WTP concept is based on limited data on raw water quality related to the Klisura intake. As mentioned earlier, for the purpose of preparation of this study, comprehensive water quality analysis (physical, chemical and micro-biological parameters) was conducted in April 2009. The analysis results are shown in Annex 5.7.

Raw Water Quality

Water quality assessment of the Grubina Reka is based on a single extended water quality analysis of the sample taken by the Institute for Public Health in Vranje on April 10, 2009. In addition to water quality parameters measure in the laboratory a number of parameters including temperature, dissolved oxygen and pH were measured at the site (Figure 5.17).

Figure 5.17: Grubina Reka at the proposed water intake site

Results of the available water quality analysis (Annex 5.7.) suggest that raw water from the Grubina Reka at the site suggested for the future raw water intake (a few hundreds meters upstream the village Klisure) is of high quality. The river water is saturated with oxygen and has neutral pH (7.3), low levels on nitrate and ammonia and nitrite bellow the detection limit. Water was odourless and had slightly elevated turbidity (2.3 NTU), colour (5 unit Pt-Co scale) and organic matter (8 mg $KMnO_4$/l), likely due to strong rains preceding the sampling. Concentrations of iron, manganese and all other metals measured (copper, lead, zinc, chromium, cadmium, nickel, arsenic - unclear, mercury) were found to be bellow the levels of detection. Similarly presence of pesticides and other selected organic micro pollutants could not be detected. Like other surface water streams in the area water from the Grubina Reka was found to have very low hardness (low calcium and magnesium concentrations) and bicarbonate, and is consequently potentially aggressive towards several materials used in water supply system.

Results of 8 standard microbiological parameters analysed showed presence of some coliform (88/100ml) and mezofilic bacteria (400/ml), but also absence of bacteria of faecal origin. In general results of analysed sample suggest that raw water from the Grubina Reka has no serious bacteriological contamination.

Available results strongly suggest that the catchment of the Grubina Reka is still free from human activities and consequently free from pollution.

Based on available water quality analysis and the site visit it can be concluded that the Grubina Reka is an attractive raw water source for the water supply of the Vlasina area. Production of drinking water from this raw water source will require reduction of turbidity (very high turbidity levels cannot be excluded after strong rains) and some colour and organic matter. It is further assumed that elevated colour and organic matter are partially in the form of particulate matter that can be relatively easily removed through conventional treatment. Further on proper disinfection is required to protect consumers from possible microbiological pollution that could take place in the catchment.

It should be, however stressed that this assessment is made on a single water quality analysis. Further monitoring of water quality that will cover different seasons is consequently strongly recommended. Further on additional hydrological investigations are required to establish reliable project of available water quantities specifically during periods of maximal water demand that could coincide with minimal flow of the hydrological cycle (e.g. August-September).

WTP Jerma – Proposed Process Scheme

WTP Jerma will have to face the following water quality challenges:

- variable turbidity that is predominantly low but can rapidly increase and become very high specifically after heavy rains,
- variable and occasionally high colour and organics
- microbiological contamination can not be excluded
- soft and aggressive nature of the raw water.

Further on WTP Jerma will most likely operate discontinuously and will be in function only to cover demand peaks (e.g. during summer months) given the higher operational costs (e.g. pumping) in comparison to WTP Vlasina.

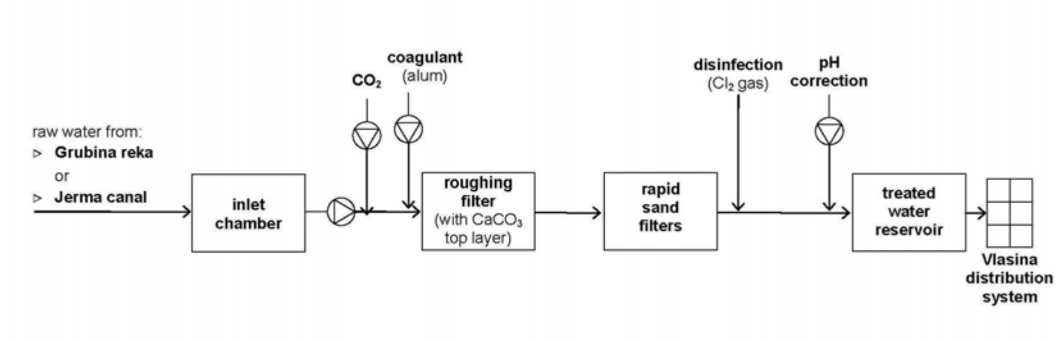
To address such challenges the following treatment scheme is proposed (Figure 5.18):

- inlet chamber
- addition of CO_2 ,
- (optional) addition of coagulant,
- up flow roughing filtration with $CaCO_3$ layer at the top,
- rapid sand filtration,
- disinfection (chlorination)
- treated water storage.

Up flow roughing filtration is proposed as a simple and robust technology that could effectively reduce occasional turbidity peaks. This technology is easy to operate and intermittent operation of the plant will not cause problems. Optional addition of coagulant (e.g. alum or iron salts) is suggested upstream the roughing filters to reduce colour and organics bellow the maximal acceptable values set by the standards. Coagulant dosing should be, however applied only if physical (pre)treatment can not guarantee organics and colour concentration in treated water bellow the maximal acceptable level.

Addition of carbon dioxide upstream the roughing filtration unit and provision of the top layer of calcium carbonate will expectedly remove water aggresivity. Assuming filtration rate of 1.5 m/h total required surface area of the roughing filters will be approximately 50 m^2. Taking into account operation and maintenance aspects two roughing filter units, each with surface area of 25 m^2 are proposed. More detailed description of the roughing filter unit is given under description of WTP Vlasina.

Figure 5.18: Proposed treatment scheme for WTP Jerma (option 1 and 2)



Filtrate from the roughing filters will be transported by gravity to 3 steel rapid sand filters, filled with quartz sand (size fraction 0.8-1.2 mm) as a filter media. Assuming conservative filtration rate of up to 8 m/h, three rapid sand filters of 2.0 m in diameter and 1.5 m filter media depth are recommended. Filters will be suitable for automatic operation including automatic backwashing on time and/or water quality basis. Filter backwashing will be done by water only at backwash rates that will allow some filter bed expansion. Filters will be placed in a building to prevent water freezing during winter period.

A chlorination system based on use of chlorine gas is proposed for disinfection of treated water. The chlorine dosing system will be automatic and will allow that required chlorine dosage is adjusted based on residual chlorine levels. It is assumed that the chlorine dosing system with a capacity of 0.20 kg/h will be sufficient. Chlorine gas will be introduced in the treated water reservoir.

Chlorinated water will be stored in the treated water reservoir with capacity of 300m^3. This reservoir will also cover hourly fluctuation of the water demand in the Vlasina distribution system together with the treated water reservoir at the Vlasina plant.

A sedimentation tank of 50 m^3 with a conical bottom will be provided to improve quality of filter backwash and roughing filters drain water before its discharge to the Jerma Creek. For the treatment of accumulated sludge at the bottom of settling tank the following process steps are recommended:

- Sludge thickening,
- Sludge storage (could be integrated with the sludge thickener),

- Sludge transport to a dewatering plant and subsequently to its final disposal.

A gravity sludge thickener is recommended. It is expected that up to approx. 10% of dry sludge content could be produced in a gravity thickener. Either a separate sludge storage tank or a storage tank integrated with the sludge thickener should be provided for the storage of thickened sludge. The total volume of sludge that should be dewatered will be very low and it is, consequently not reasonable to provide a dedicated sludge dewatering unit for the Vlasina area. Consequently transport of ticked sludge, together with the sludge from WTP Vlasina, to another central sludge dewatering plant in the region is recommended.

The plant will have continuous on-line monitoring of feed water, pre-treated and treated water turbidity. In addition pH of raw and treated water will be continuously monitored.

The plant will have no own laboratory and basic quality parameters (e.g. chlorine, turbidity, pH) should be analysed at the small laboratory that will be established at the upgrade WTP Vlasina, while some other parameters could be analysed at the central laboratory of PUC Surdulica that will own and operate WTPs Jerma and Vlasina.

Further on flow and water meters will be provided on the feed line and transport line to the distribution system. Monitoring of these instruments from the control room will be provided.

All the main valves at the plant will be equipped with electro magnetic drives that could be operated from the control room.

To minimize operational (staff) costs feasibility of to full automation of the WTP Jerma that will allow operation and control from the WTP Vlasina control room should be considered in the next design stage.

WTP Jerma – Overview of Investment and Operational Costs

It is estimated that the total investment costs associated with construction of WTP Jerma will be € 510.000. It is further estimated that contribution of mechanical and electrical equipment will be 60% and that remaining 40% will be civil works.

The water treatment process proposed in option 1 will have limited operational costs in terms of energy and chemicals.

Chemicals that will be used will most likely be limited to use of chlorine gas for disinfection and carbon dioxide dosing and refiling of calcium carbonate layer placed at the top of roughing filters. Occasional dosing of low coagulant (e.g. alum) dosages can not be excluded. Related operational costs will, however, be marginal taking into account low required dosages, only occassional need for coagulant dosing, and low costs of coagulants. Based on an indicative calculations the total chemical costs will be limited to approximately 0.04 €/m^3.

It is assumed that the major use of electricity will be for backwashing of filters, possibly transport of pre-treated water to rapid sand filters (only in the case if the site topography will not allow gravity transport) and the heating of the building during winter months. It is estimated that total energy costs will not exceed 0.02 €/m^3 produced water. Energy costs for transport of raw water from the Grubina Reka to WTP Jerma are however, not included. It is further estimated that in total 2 operators will be employed part time to take care about the plant together with staff working at the WTP Vlasina.

Operation and maintenance costs of the WTP Jerma under option 1 will consequently include:

- chemical costs (e.g. chlorine, calcium carbonate filter media, carbon dioxide and possibly a coagulant) - 0.04 €/$m^{3,}$
- electricity (excluding transport of raw water to the plant) - 0.02 €/m^3,
- operators estimated at 2 persons available 6 months per year (e.g. during high season),
- the plant maintenance including civil works and equipment estimated at 0.5% and 3% of investment value, respectively.

WTP Vlasina

In option 1 the existing WTP Vlasina will remain in use and will be the major drinking water production plant for the Vlasina water supply system.

Raw Water Quality

<u>Plavilo and Cvejina Dolina Creeks / Springs</u>
At present WTP Vlasina receives raw water from the Plavilo Creek. Water quality of this source was assessed based on water quality analysis received from the PUC that included samples taken between January 2008 and March 2009. These analysis, however covered only very limited number of a number of basic water quality parameters including pH, colour, turbidity, organics (measured as $KMnO_4$ consumption), ammonia, nitrate, nitrite and conductivity. In addition results from single more extended water quality analysis conducted as a part of Hidrosanitas detailed design in 1983 were considered.

Based on available water quality data taken at the WTP Vlasina inlet raw water from the Plavilo creek is characterised by:

- slightly elevated turbidity (0.9 – 3. 2 NTU)*,
- occasionally elevated colour level (5-7 Pt-Co units),
- elevated and variable organic matter content as measured by $KMnO_4$ demand (6.0-26.2),
- low levels of ammonia that are only occasionally exceeding the maximal acceptable levels (<0.03-0.16 mg/l),
- low levels of nitrate (≤10.8 mg/l),

- occasional presence of nitrites at concentrations above the maximal acceptable levels (<0.006-0.06),
- very low concentrations of all metals measured,
- very low conductivity (27-37 μS/cm) and
- very low hardness (0.85-1.38 ˚dH).

*It should be mentioned that the raw water supply to the plant is typically closed when turbidity levels are high and that consequently turbidity picks were most likely not recorded.

Variation of basic water quality parameters of the Plavilo Creek at the WTP Vlasina entrance and treated water is given in Figures 5.19 to 5.23.

Figure 5.19: Turbidity of raw water from Plavilo Creek at the WTP Vlasina inlet and treated water

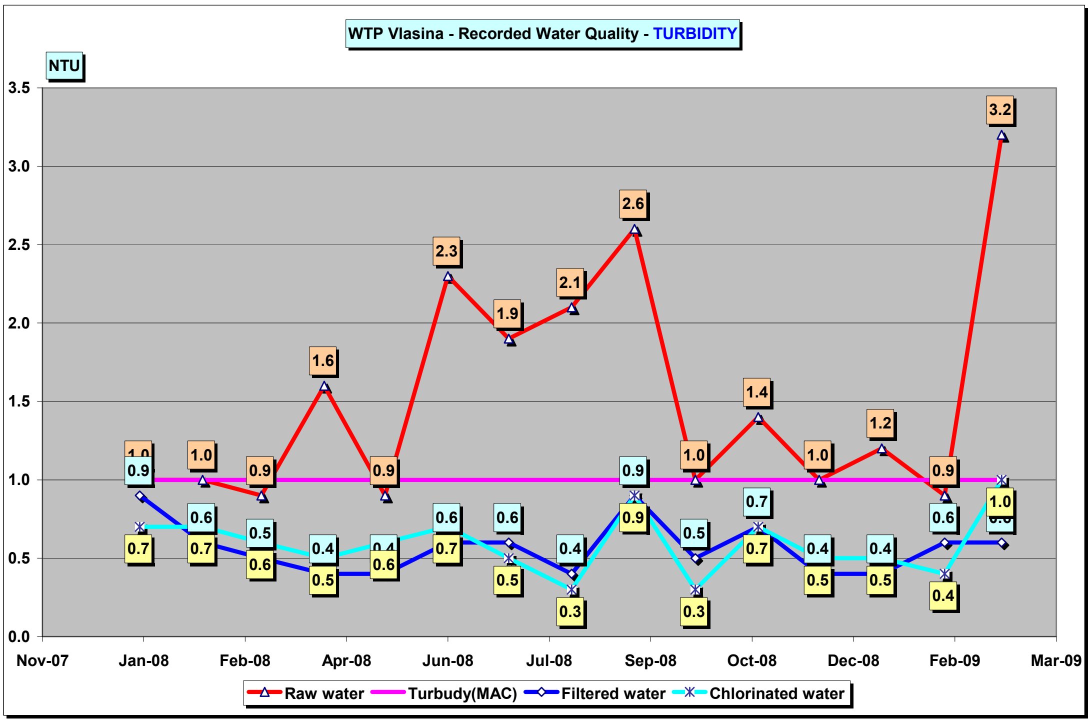


Figure 5.20: Concentration of organic matter measured as KMnO4 consumption of Plavilo Creek water at the WTP Vlasina inlet and treated water.

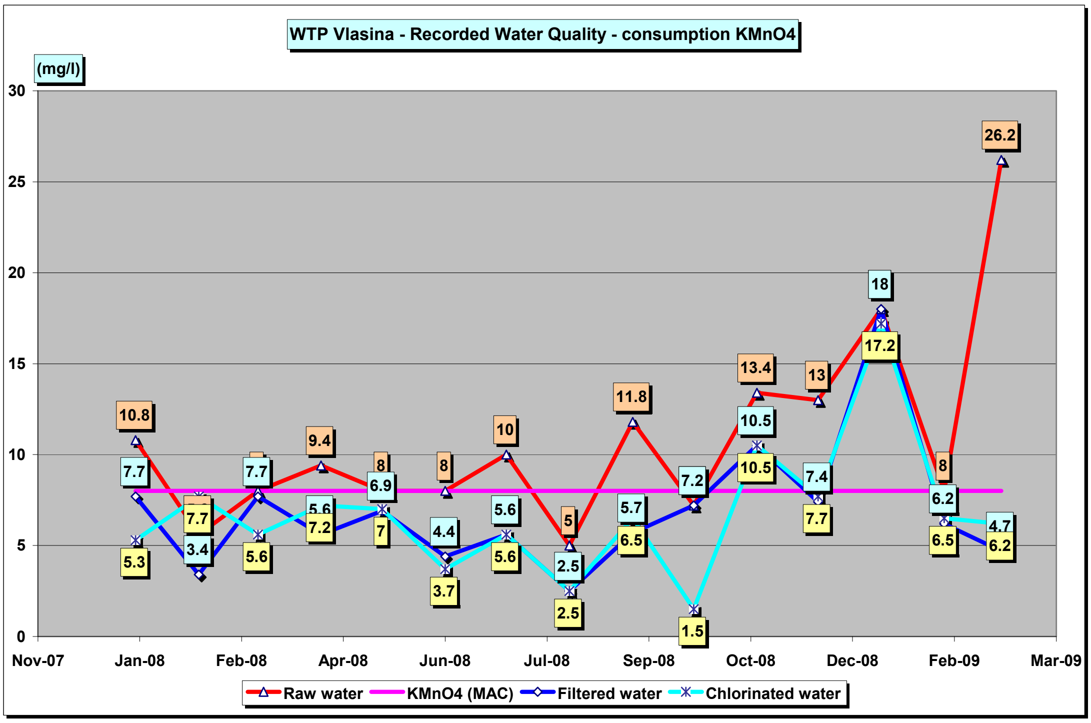


Figure 5.21: Fluctuations of conductivity of raw water and treated water from the Plavilo Creek

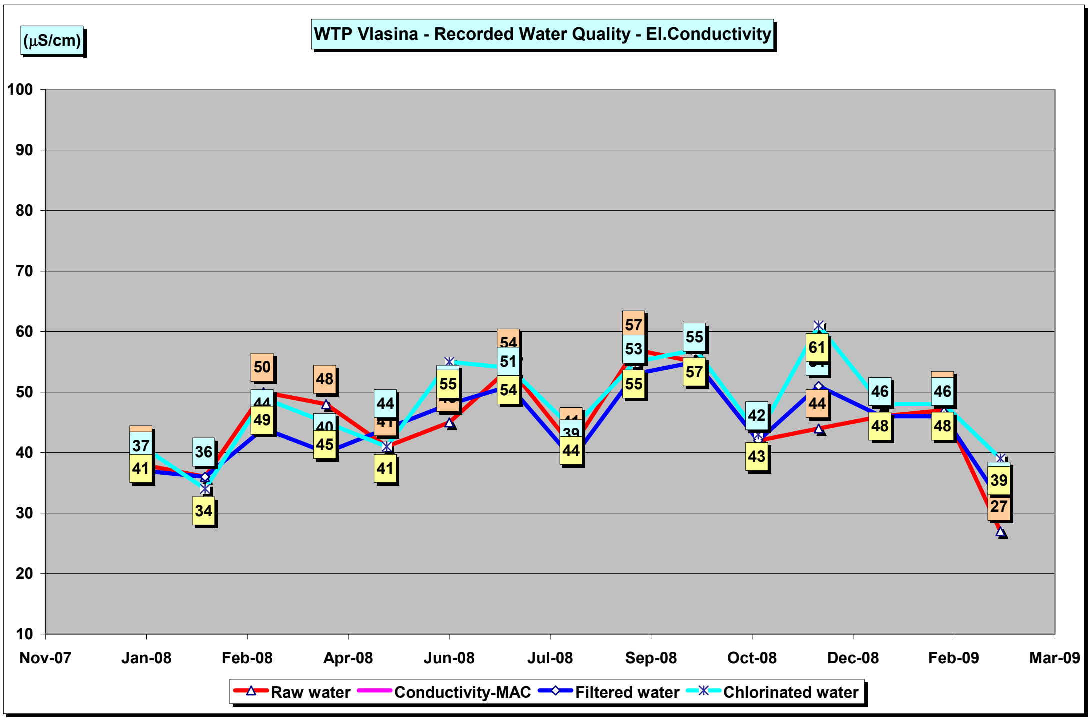


Figure 5.22: Fluctuations of ammonia in raw water and treated water from the Plavilo Creek

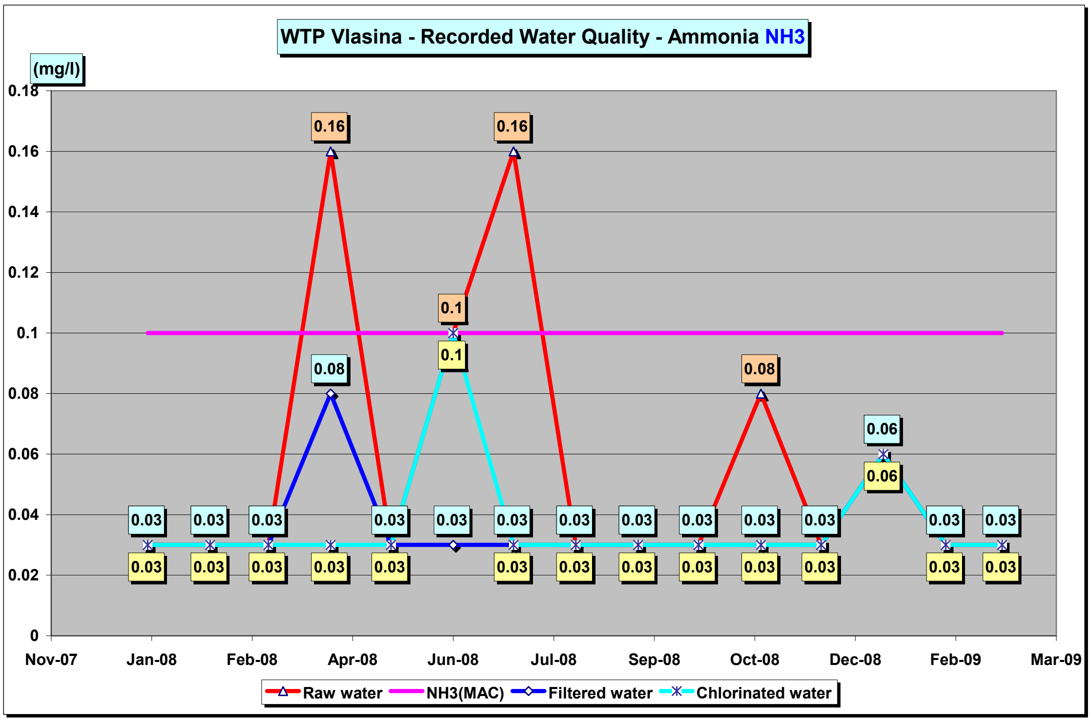
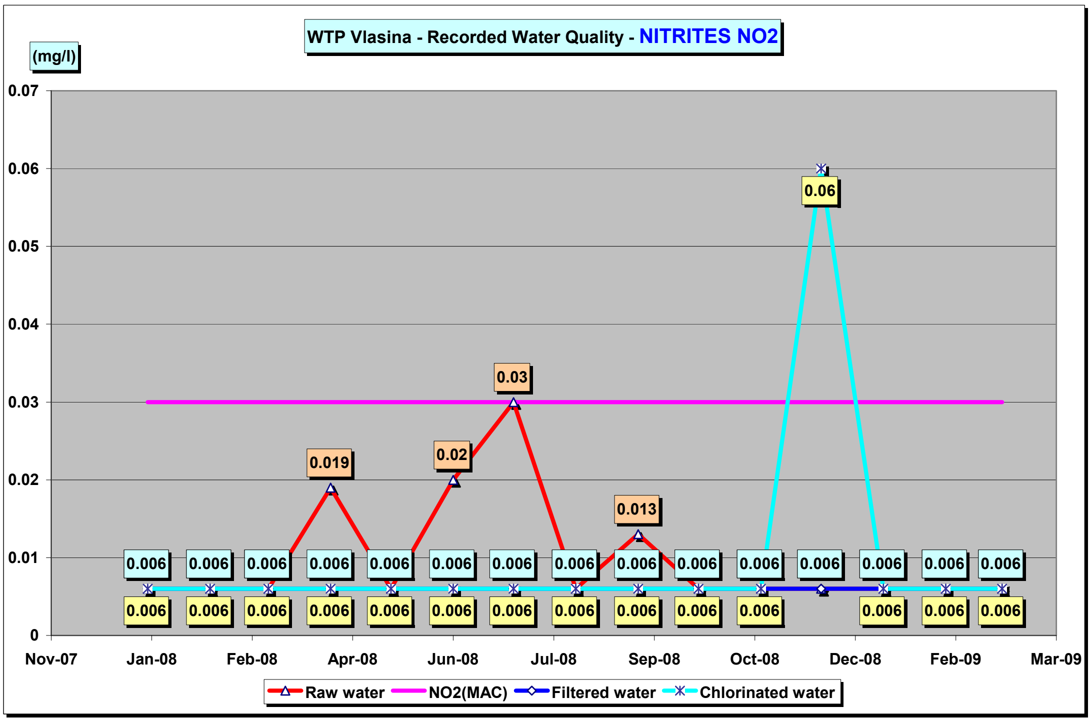


Figure 5.23: Fluctuations of nitrites in raw at treated water from the Plavilo Creek



Based on the information received from the operators it could be concluded that much higher turbidity levels are occasionally present in the raw water, specifically after heavy rain events. It is common operational practice at present to stop operation of the plant under such conditions.

Results of regularly conducted basic micro biological water quality analysis show high concentration of coliform bacteria including occasional presence of high concentration of coliforms of faecal origin (>24.000 per 1000 ml). Presence of coliform bacteria of faecal origin suggests that there is uncontrolled discharge of domestic waste water in the creek catchment. Consistent presence of microbiological contamination of faecal origin strongly suggests that proper disinfection is essential to protect the safety of water consumers. It is not unlikely that microbiological quality will strongly improve if the current water intake is moved further upstream and is located at the Plavilo Spring.

Available water quality data as well as applied treatment process are suggesting that both raw and treated water are very aggressive towards different materials present in the water supply system including concrete, asbestos cement, etc.

Very limited available hydrological data suggest that the Plavilo spring could provide up to 30 l/s during summer months characterized by highest water demand. This capacity could be sufficient to cover the maximal daily demand of the Vlasina area till approximately the year 2020. Additional 10 l/s could be provided by connecting the Cvejina Dolina Spring to the raw water supply system transporting water to the Vlasina WTP. It is assumed that joint capacity of Plavilo and Cvejina Dolina springs will be sufficent to cover the maximal daily water demand of the Vlasina area till approximately the year 2030.

Water quality of the Cvejina Dolina spring was not closely monitored since 1983. The only available water quality data for this spring were found in the Hidrosanitas (1983) detailed design (Table 5.4). Table 5.4 also gives water quality of the Čemernica River that collects water from Plavilo, Cvejina Dolina and other springs on the Čemerno Mountain. These results together with the similar location and geological formations are suggesting that raw water quality of Plavilo and Cvejina dolina springs is most likely very similar.

Table 5.4: Water quality of Plavilo and Cvejina Dolina creeks and River Čemernica (Hidrosanitas 1983)

Water Quality Parameter	Plavilo	Cvejina Dolina	Čemernica River
Turbidity (NTU)	2.0	2.5	3.0
pH	6.85	7.0	6.7
Colour (Pt-Co scale)	0.0	0.0	0.0
Conductivity at 20˚C (µS/cm)	27.8	39.5	35
$KMnO_4$ consumption (mg/l)	6.0	5.37	6.3
Hardness (˚dH)	0.85	1.38	1.37
NH_4 (mg/l)	0.026	0.028	0.03
Fe (mg/l)	0.00	0.00	0.00
Mn (mg/l)	0.00	0.00	0.00
Cu (mg/l)	0.005	0.003	0.0035
Cr –total (mg/l)	0.004	0.002	0.005

Water Quality Parameter	Plavilo	Cvejina Dolina	Čemernica River
Cd (mg/l)	0.0001	0.0001	0.0001
Zn (mg/l)	0.0004	0.001	0.0005
Pb (mg/l)	0.001	0.001	0.001
Ni (mg/l)	0.001	0.001	0.001
Al (mg/l)	0.005	0.004	0.03
NO_3 (mg/l)	0.010	0.014	Traces
NO_2 (mg/l)	0.000	0.001	0.000
Cl (mg/l)	2.05	2.05	2.05
SO_4 (mg/l)	0.4	0.5	1.5
PO_4 – ortho (mg/l)	0.01	0.015	0.005

WTP Vlasina – Proposed Process Scheme

Option 1 proposes that the existing WTP Vlasina will remain the major drinking water production facility in the Vlasina area. In order to fulfil this task the existing plant will have to be expanded to an operational capacity of 30 l/s. Together with the WTP Jerma with capacity of 20 l/s upgraded WTP Vlasina will have sufficient capacity to cover the maximal projected water demand till the year 2035, the end of project period of this study.

In addition applied treatment process should be upgraded in order to guarantee continuous and consistent production of high quality drinking water from raw water from Plavilo and Cvejina Dolina creeks.

The plant will initially continue to treat raw water from the Plavilo Creek only. Once capacity of the Plavilo Creek is insufficient the plant will be followed by treatment of mixture of raw water from the Plavilo and Cvejina Dolina creeks. These two springs has very similar water quality and consequently treatment of mixed raw water from thee two springs will not require alteration of the treatment strategy.

Based on the available water quality data are suggesting that the WTP Vlasina should achieve the following treatment objectives:

- turbidity of raw water that is variable occasionally very high should be consistently reduced; intermittent operation as done at present with stop of water production when turbidity levels are high is unacceptable,
- concentrations of organics and colour that are also occasionally high and exceeding the maximal acceptable concentration should be consistently reduced,
- produce drinking water should be bacteriologically safe even when occasionally raw water has high bacteriological contamination
- high water aggresivity should be neutralized.

When establishing appropriate treatment strategy the following criteria were considered:

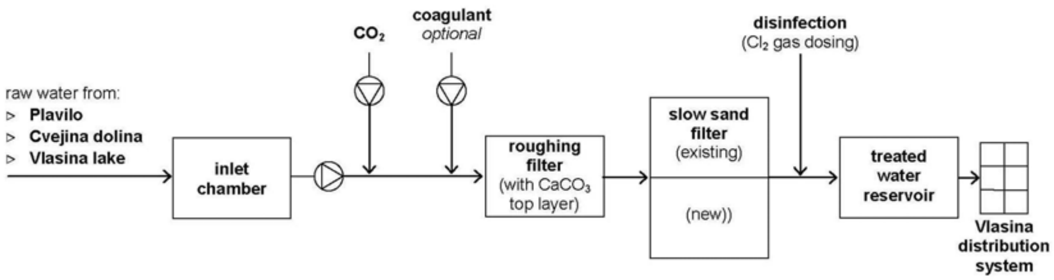
- upgraded plant should be simple and if possible based on principles of the existing treatment process

- existing treatment plant should be incorporated in the treatment process of the upgraded plant as much as possible,
- operational and maintenance costs should be minimal,
- the proposed treatment process should be suitable for strong variation of production capacity,
- the plant should be designed for a maximal production capacity of 30 l/s.

Based on available water quality data and established criteria the following treatment scheme is proposed (Figure 5.24):

- inlet chamber
- addition of CO_2,
- optional addition of coagulant,
- up flow roughing filtration
- slow sand filtration (existing and extended),
- disinfection (chlorination)
- treated water storage.

Figure 5.24: Proposed Treatment scheme for upgrading of WTP Vlasina



The inlet chamber will provide stable hydraulic conditions at the plant entrance. An electromagnetic flow regulating valve should be placed at the chamber inlet together with a flow meter. Operation of the flow meter and reading of flow should be possible from the control room. Feed water flow should be hand adjusted by the operator from the control room based on expected water demand and water level in the treated water reservoir.

In order to allow smooth and continuous operation of the plant and specifically slow sand filters an up flow roughing filter is suggested as pre-treatment upstream the slow and filters. Roughing filter is very simple and reliable pre-treatment technology for removal of (fine) particulate matter. Based on results from practice it could be assumed that roughing filter will reduced ≥ 90% of particulate matter and also improve the bacteriological water quality by achieving 1 to 2 log reduction of faecal coliforms. It has also been reported that roughing filters could reduce the colour and organics. Additional coagulant dosing could be, however required when colour and organics are in dissolved forms.

For the upgraded WTP Vlasina two up flow roughing filter units are proposed, each consisting of several layers of coarse sand and gravel (4-20 mm) with coarser material at the filter bottom, with total filter media depth of approximately 2.5 m. Total surface area of both roughing filters of 72 m^2 is suggested corresponding to maximal filtration rate of 1.5 m/h. Such filtration velocity is rather high for roughing filters but it the case of WTP Vlasina this loading rate will be reached only in the day of maximal water demand at the end of the project period. Most of the time this unit will operate at much lower loading. Cleaning of the roughing filters will be done under gravity, by simple opening of drain valve(s) at the bottom of the filters.

A separate plain sedimentation tank with volume of approximately 50m^3 with conical bottom will be provided to settle water from cleaning of roughing filters and discharge of settled water to the sewer system. Similar strategy for further handling of accumulated sludge as proposed for the WTP Jerma will be followed for the WTP Vlasina.

At the top of the roughing filter a layer (e.g. 0.5 m) of calcium carbonate filter media (e.g. 1.8-2.5 mm) will be placed. In addition to polishing effect on water quality filtration through calcium carbonate will increase calcium and bicarbonate concentration of water and reduce its agressivity. Placing the calcium carbonate at the to of the roughing filter will allow simple and easy refilling that should be done occasionally, to replace calcium carbonate dissolved (e.g. when approximately 1/3 of the layer is dissolved). To allow proper chemical conditions for this unit addition of CO_2 upstream the filter will most likely be required. It is expected that pH of the roughing filter filtrate will have pH of approximately 8.0.

A separate storage room should be provided for storage of calcium carbonate filter media that should re refilled several time per year.

Optional and occasional addition of coagulant (e.g. an iron or aluminium based salt) could be considered if levels of dissolved organics and colour increase in the future and the proposed treatment consisting of roughing and slow sand filtration cannot remove it with required efficiency. Based on the available water quality data such possibility is likely remote.

The roughing filtration unit should be placed in a building to avoid the risks of freezing during winter months. The roughing filtration unit should be if possible located such to allow gravity transport of water to slow sand filtration units. Consequently no water pumping will be required at the plant.

Existing slow sand filtration units are at present not operated as real slow sand filters (e.g. intermittent operation, inappropriate filter cleaning procedure, short filter run cycle, etc.). Nevertheless based on available data on treated water quality it looks that drinking water of good quality is produced. Further on the plant operators have experience with this process. Slow sand filtration does not require use of any chemicals, use of energy and produced waste are is minimal. It is consequently recommended to continue use of slow sand filtration also in the upgraded plant.

Total surface area of slow sand filters will be increased to 220 m^2 by constructing one additional filter with surface area of 70 m^2. Slow sand filters will consequently operate at the maximal filtration rate of 0.5 m/h. This is rather high filtration rate for slow sand filters, however filters will operate at such high rate only during the days of maximal water demand at the end of the project period (2035). It is also assumed that feed water will have low turbidity due to positive effect of pre-treatment with roughing filters.

To allow better maintenance of the slow sand filters sand washing area and sand storage room will be provided. This will allow washing and re-use of the dirty sand removed after the filters cleaning.

As suggested under the assessment of the existing plant complete reconstruction of the entrance chamber, three slow sand filter units including replacement of all pipes and valves in the filter gallery is also required. To allow continuous production of drinking water also during the plant reconstruction it is suggested that the reconstruction of the existing filter should be initiated during low season and only after completion of the new the plant components consisting of the roughing filter and the extension of slow sand filter units.

In agreement with the existing plant filtrate from the slow sand filter filtrate will be disinfected with chlorine. The existing chlorination is based on use of sodium hypochlorite and is in very poor condition. Consequently, completely new chlorination system is proposed based on use of chlorine gas. The chlorine dosing system should be automatic and should allow that required chlorine dosage is adjusted based on residual chlorine levels. It is expected that proposed treatment scheme together with improved operation of slow sand filters will reduce required chlorine dosages. It is assumed that the capacity of chlorination system of 0.3 kg/h will be sufficient.

The existing treated water reservoir of 500 m^3 will be sufficient for the smooth operation of the plant and consequently no additional reservoir volume is proposed on the treatment plant site.

The existing plant has at present no water quality monitoring system. The upgraded plant should have continuous on-line monitoring of feed water, pre-treated and slow sand filtrate turbidity. In addition pH of raw and treated water should be continuously monitored.

It is further proposed to provide a very simple laboratory that can measure a few basic water quality parameters (e.g. chlorine, turbidity, and pH) as a control of systems for continuous automatic water quality monitoring. It is further assumed that all other water quality parameters could be monitored in the central laboratory of PUC Surdulica that will own and operate the WTP Vlasina.

All the main valves at the plant will be equipped with electro magnetic drives that could be operated from the control room. Further on flow and water meters will be provided on the feed line and transport line to distribution area. Monitoring of these instruments from the control room will be provided.

WTP Vlasina – Overview of Investment and Operational Costs

It is estimated that the total investment costs associated with the upgrading of existing water treatment plant, that will include rehabilitation of the existing plant and provision of new treatment units including one new slow sand filter unit will be € 540.000. It is further estimated that majority of works (75%) will consist of civil works and that remaining 25% will be mechanical and electrical works.

Very similar to current situation the upgrade water treatment process will have very limited operational costs in terms of energy and chemicals.

Chemicals that will be used will most likely be limited to use of chlorine gas for disinfection and carbon dioxide dosing and refilling of calcium carbonate layer placed at the top of roughing filters and possible occasional dosing of coagulant. Based on an indicative calculations the total chemical costs will be limited to approximately 0.04 €/m^3.

The Vlasina WTP will operate by gravity and consequently will make only marginal use of electricity for the plant operation. It is assumed that the major use of electricity will be for the heating of the building during winter months. It is estimated that total energy costs will not exceed 0.01 €/m3 produced water.

It is assumed that continuous (24 h/day) presence of an operator will be provided. It is estimated that in total 5 staff should be employed to take care about the plant in addition to 3 operators that will mainly take care about the WTP Jerma.

Operation and maintenance costs of the WTP Vlasina under option 1 will consequently include:

- chemical costs (e.g. chlorine, calcium carbonate filter media, carbon dioxide and possibly a coagulant) - 0.04 €/$m^{3,}$
- electricity - 0.01 €/m^3,
- operators estimated at 5 full time staff members,
- the plant maintenance including civil works and equipment estimated at 0.5% and 3% of investment value, respectively.

Raw water intakes and transport to WTP Vlasina

As a pre-requisite for planned development of the proposed option 1, it is necessary to improve and extend raw water supply to the WTP Vlasina. This would enable stable, safe and efficient operation of the WTP and extension of its capacity, as required by the proposed option 1. The works related to upgrade and extension of the raw water intakes and transport system include the following:

- Adequate/sanitary protection and upgarde of the existing water intake/spring Plavilo;

- Construction of an intake and connecting pipeline for a nearby spring called Cvejina Dolina;
- Additional hydrological, hydro-geological and water quality investigations and construction of new water intakes at the outskirts of Cemernik: Bele vode and Ignjatova cesma. Connection of these spring to the WTP Vlasina;

It should be noted that the abovementioned works related to sanitary protection and construction of new water intakes would be strogly affected by planned hydrological, hydro-geological and water quality investigations that are deemed to be necessary before the abovementioned works commenece.

However, options 1 makes use of the existing water supply system components (raw water intakes and pipelins, water treatment facilities, water storage facilities, distribution network) with necessary qualitative and quantitative upgrade. The following table provides the main technical measures and features of the proposed raw water intake and transportation in accordance with option 1.

Table 5.5: Main Features of raw water transmission system – Cemernik-Vlasina

Water intakes				
Name	**Estimated capacity (l/s) ***	**Elevation (masl)**	**Notes**	
Water intake Cvejina dolina	7	1.450	* Capacity of spring/intake should be established by means of hydrological and hydro-geological investigations	
Water spring/intake Bela voda	10	1.560	* Capacity of spring/intake should be established by means of hydrological and hydro-geological investigations	
Water spring/intake Ignjatova cesma	10	1.560	* Capacity of spring/intake should be established by means of hydrological and hydro-geological investigations	
Tanks / Break pressure tanks				
Name	**Volume**	**Minimum level (masl)**	**Maximum level (masl)**	**Note**
BPT "Cemernik"	10	1450	1452	Pressure control on line from spring Bela voda
Pipelines				
Start section	**End section**	**DN (m)**	**L (m)**	**Recommended material**
Intake Cvejina dolina	Connection to line to WTP Vlasina	100	400	PN10 HDPE/ST/DI
Spring Bela voda	Connection to line to WTP Vlasina	100	2300	PN10 HDPE/ST/DI
Spring Ignjatova cesma	Intake Plavilo	100	1900	PN10/16 HDPE/ST/DI

Table 5.6: Option 1 - Overview of investment costs

1 Raw water abstraction, transmission Cemernik - Vlasina Rid

	Description	Unit of measure	Qty.	Civil works (€)	Equipment (€)	Piping (€)	TA (€)	Total costs (€)
1.1	Sanitary protection and upgrade of water intake Plavilo	lump sum	1	15,000				**15,000**
1.2	Water intake Cvejina dolina	l/s	7	15,000				**15,000**
1.3	Water spring/intake Bela voda	l/s	10	20,000				**20,000**
1.4	Water spring/intake Ignjatova cesma	l/s	10	20,000				**20,000**
1.5	Raw water pipeline from intake Cvejina dolina DN100	m	400	5,200		4,800		**10,000**
1.6	Raw water pipeline from intake Bela voda DN100	m	2300	29,900		27,600		**57,500**
1.7	Raw water pipeline from intake Ignjatova cesma DN100	m	1900	24,700		22,800		**47,500**
1.8	BPT Cemernik	m^3	10	4,800	600	600		**6,000**
1	**Raw water abstraction, transmission Cemernik - Vlasina Rid**			**134,600**	**600**	**55,800**	**0**	**191,000**

2 WTP Vlasina - upgrade and extension to 30 l/s

	Description	Unit of measure	Qty.	Civil works (€)	Equipment (€)	Piping (€)	TA (€)	Total costs (€)
2.1	Upgrade of the existing WTP Vlasina	l/s	15	45,000	105,000			**150,000**
2.2	Extension of the WTP Vlasina for additional capacity of 15 l/s net	l/s	15	90,000	135,000			**225,000**
2	**WTP Vlasina - Rehabilitation and extension to 30 l/s**			**135,000**	**240,000**			**375,000**

3 WTP Jerma - (design capacity 20 l/s)

	Description	Unit of measure	Qty.	Civil works (€)	Equipment (€)	Piping (€)	TA (€)	Total costs (€)
3.1	Construction of WTP Jerma	l/s	15	94,500	220,500			**315,000**
3	**WTP Jerma**			**94,500**	**220,500**			**315,000**

4 Water storage tanks and break-pressure tanks

	Description	Unit of measure	Qty.	Civil works (€)	Equipment (€)	Piping (€)	TA (€)	Total costs (€)
4.1	**BPT Klisura**	m^3	100	36,000	4,500	4,500		**45,000**
4	**Water storage tanks**		**100**	**36,000**	**4,500**	**4,500**	**0**	**45,000**

	Description	Unit of measure	Qty.	Civil works (€)	Equipment (€)	Piping (€)	TA (€)	Total costs (€)
5 Pumping stations								
5.1	**PS Klisura 1**	kW	1.65	36,000	3,506	619		**40,125**
5.2	**PS Klisura 2**	kW	1.65	36,000	3,506	619		**40,125**
5	**Pumping stations**			**72,000**	**7,013**	**1,238**		**80,250**
6 Pipelines								
	Description	**Unit of measure**	**Qty.**	**Civil works (€)**	**Equipment (€)**	**Piping (€)**	**TA (€)**	**Total costs (€)**
6.1	Pipeline Klisura - BPT Klisura 1	m	2050	26,650		61,500		**88,150**
6.2	Pipeline BPT Klisura 1 - WTP Jerma	m	2300	29,900		69,000		**98,900**
6	**Pipelines**			**56,550**	**0**	**130,500**		**187,050**
7 Raw water intake Klisura								
	Description	**Unit of measure**	**Qty.**	**Civil works (€)**	**Equipment (€)**	**Piping (€)**	**TA (€)**	**Total costs (€)**
7.1	Raw water intake - Klisura	lump sum	1	30,000				**30,000**
7	**Raw water intake Klisura**			**30,000**	**0**	**0**		**30,000**
				Civil works (€)	**Equipment (€)**	**Piping (€)**	**TA (€)**	**Total costs (€)**
	Grand total per type of works			**558,650**	**472,613**	**192,038**	**0**	**1,223,300**

INVESTMENT COST SUMMARY

1	**Raw water abstraction, transmision Cemernik - Vlasina Rid**	**191,000**
2	**WTP Vlasina - upgrade and extension to 30 l/s**	**375,000**
3	**WTP Jerma**	**315,000**
4	**Water storage and break-pressure tanks**	**45,000**
5	**Pumping stations**	**80,250**
6	**Pipelines**	**187,050**
7	**Water intake Klisura**	**30,000**
	Gross total - no VAT, no supervision	**1,223,300**

	Construction supervision - 3%						**36,699**
	Gross total - no VAT, construction supervision included						**1,259,999**

Notes:

Lifetime for individual project elements has been adopted as follows:

- *Civil engineering works and structures - 50 years*
- *Equipment (mechanical, electrical, process) - 15 years*
- *Piping (underground, ductile iron pipes) - 50 years*

5.1.5.2 Option 2

This option is based on the optimal operation of the existing facilities (with necessary upgrade and extension) with supplementary supply using Jerma canal that normally feeds Vlasina Lake.

Jerma canal would be used as a supplementary source instead of the Grubina River as outlined in option 1.

Jerma canal is normally used for feeding of Vlasina Lake with water collected from a number of smaller springs and creeks north of Vlasina Lake. Jerma canal ends in a creek called Pojiste which further inflows into Vlasina Lake.

Jerma canal catchment was also indicated as a potential potable water source in the Spatial Plan of Special Purpose Area Vlasina. The canal is currently managed by the Power Supply Authority of Serbia (EPS). Discharges from the canal to the lake are recorded on a monthly basis. This information has been sought from EPS, but it is still not available. Based on preliminary information indicated by an EPS representative, it appears that the minimal recorded discharge in the canal was app. 70 l/s. Although this information still has not been verified, it could prove to represent a significant potable water potential, suitable for supply of the project area. In the Spatial Plan of the Special Purpose Area Vlasina it was indicated that it would be possible to abstract as much as 60 l/s for potable water supply of the Project Area.

Suitability of this potential potable water source should be verified by means of hydrological and water quality investigations.

It is proposed to abstract water directly from the canal and to divert to the WTP Jerma by gravity.

Water Intake

Basic characteristics of the water intake:

- Type - directly from the canal with inlet screen and settling chamber
- Altitude - 1280 masl
- Estimated capacity - 20 l/s (to be verified by additional hydrological investigations)

WTP Jerma

Concept and capacity of WTP Jerma would be rather similar to one described in option 1. Specifics of WTP Jerma in accordance with the option 2 include the following:

Raw Water Quality

Jerma Canal
No data on raw water quality from the Jerma canal were available at the time this report was compiled. Given the characteristic of the Jerma canal catchment it is not unlikely that raw water from the canal will show similarity to composition of raw water from the Grubina Reka and / or water from creeks on the Čemerno Mountain. As a consequence it can be assumed that raw water from the Jerma canal is in general of a high quality. The following water quality problems cannot be, however excluded:

- occasionally elevated turbidity with a realistic chance for very high turbidity levels specifically after heavy rains,
- bacteriological contamination due to some settlements and natural pollution (e.g. wild animals),
- variable and occasionally (e.g. after heavy rains) high levels of organic matter and colour,
- very soft and potentially aggressive water.

In order to verify these assumption water qualities monitoring of the Jerma canal is strongly recommended that should preferably cover different seasons.

Plavilo and Cvejina Dolina Creeks
In this option the existing water supply system based on use of raw water from the Plavilo Creek and WTP Vlasina will remain in place. Water quality problems associated with the use of the Plavilo Creek include variable and occasionally high turbidity, colour and organic matter, occasional microbiological contamination and very soft and aggressive nature of the water.

In addition to Plavilo Creek raw water from Cvejina dolina will also be used as raw water source for the WTP Vlasina. An assumption was made in this study that raw water from this source has very similar quality as raw water from the Plavilo Creek.

More detailed review of the quality of the raw water from the Plavilo and Cvejina Dolina was already provided under the option 1.

WTP Jerma – Proposed Process Scheme

Given the very similar expected raw water quality of the Jerma canal and the Grubina Dolina River it is assumed that WTP Jerma in option 2 will be identical to the plant described in the option1. The following treatment scheme is consequently proposed (Figure 5.18):

- inlet chamber
- addition of CO_2 ,
- (optional) addition of coagulant,
- up flow roughing filtration with $CaCO_3$ layer at the top,
- rapid sand filtration,
- disinfection (chlorination)
- treated water storage.

More detailed process descrition of the WTP Jerma can be found in the option 1.

WTP Jerma – Overview of Investment and Operational Costs

WTP Jerma

Given the expected similarity in raw water composition of the Grubina Reka and the Jerma canal, identical treatment process proposed and the same plant site it is assumed that investment and operational costs of the WTP Jerma in option 2 will be identical with investment and operational costs established for option 1.

Total investment costs associated with construction of WTP Jerma in option 2 are consequently estimated at € 510.000 with contribution of equipment (mechanical and electrical) and civil works estimated at 60% and 40%, respectively

Identical to option 1 operation and maintenance costs will include:

- chemical costs (e.g. chlorine, calcium carbonate filter media, and possibly a coagulant) of approximately 0.04 €/$m^{3,}$
- electricity estimated at 0.02 €/m^3,
- operators estimated at 2 operators available 6 months per year (e.g. during high season),
- the plant maintenance including civil works and equipment estimated at 0.5% and 3% of investment value, respectively.

WTP Vlasina Investment and Operational costs

Investment and operational costs of WTP Vlasina in option 2 will be identical to costs specified in option1.

The total investment costs associated with the upgrading of existing water treatment plant, that will include rehabilitation of the existing plant and provision of new treatment units including one new slow sand filter unit will be € 540.000. It is further estimated that majority of works (75%) will consist of civil works and that remaining 25% will be mechanical and electrical works.

Operation and maintenance costs of the WTP Vlasina under option 2 will consequently include:

- chemical costs (e.g. chlorine, calcium carbonate filter media, carbon dioxide and possibly a coagulant) - 0.04 €/$m^{3,}$
- electricity - 0.01 €/m^3,
- operators estimated at 5 full time staff members,
- the plant maintenance including civil works and equipment estimated at 0.5% and 3% of investment value, respectively.

WTP Vlasina and raw water intakes and transport

With regard to upgrade and extension of the WTP Vlasina and belonging water sources and raw water transport system it is planned to implement the measures and works as described in option 1.

Table 5.7: Option 2 - Overview of investment costs

1 Raw water abstraction, transmision Cemernik - Vlasina Rid								
	Description	**Unit of measure**	**Qty.**	**Civil works (€)**	**Equipment (€)**	**Piping (€)**	**TA (€)**	**Total costs (€)**
1.1	Sanitary protection and upgrade of water intake Plavilo	lump sum	1	15,000				**15,000**
1.2	Water intake Cvejina dolina	l/s	7	15,000				**15,000**
1.3	Water spring/intake Bela voda	l/s	10	20,000				**20,000**
1.4	Water spring/intake Ignjatova cesma	l/s	10	20,000				**20,000**
1.5	Raw water pipeline from intake Cvejina dolina DN100	m	400	5,200		4,800		**10,000**
1.6	Raw water pipeline from intake Bela voda DN100	m	2300	29,900		27,600		**57,500**
1.7	Raw water pipeline from intake Ignjatova cesma DN100	m	1900	24,700		22,800		**47,500**
1.8	BPT Cemernik	m^3	10	4,800	600	600		**6,000**
1	**Raw water abstraction, transmission Cemernik - Vlasina Rid**			**134,600**	**600**	**55,800**	**0**	**191,000**
2 WTP Vlasina - upgrade and extension to 30 l/s								
	Description	**Unit of measure**	**Qty.**	**Civil works (€)**	**Equipment (€)**	**Piping (€)**	**TA (€)**	**Total costs (€)**
2.1	Upgrade of the existing WTP Vlasina	l/s	15	45,000	105,000			**150,000**
2.2	Extension of the WTP Vlasina for additional capacity of 35 l/s net	l/s	15	90,000	135,000			**225,000**
2	**WTP Vlasina - Rehabilitation and extension to 20 l/s**			**135,000**	**240,000**			**375,000**
3 WTP Jerma - (design capacity 20 l/s)								
	Description	**Unit of measure**	**Qty.**	**Civil works (€)**	**Equipment (€)**	**Piping (€)**	**TA (€)**	**Total costs (€)**
3.1	Construction of WTP Jerma	l/s	15	112,500	262,500			**375,000**
3	**WTP Jerma**			**112,500**	**262,500**			**375,000**
4 Raw water intake Jerma								
	Description	**Unit of measure**	**Qty.**	**Civil works (€)**	**Equipment (€)**	**Piping (€)**	**TA (€)**	**Total costs (€)**
4.1	Raw water intake	lupm sum	1	50,000				**50,000**
4	**Raw water intake Jerma**			**50,000**	**0**	**0**		**50,000**

				Civil works (€)	Equipment (€)	Piping (€)	TA (€)	Total costs (€)
	Grand total per type of works			432,100	503,100	55,800	0	991,000

	INVESTMENT COST SUMMARY						
1	Raw water abstraction, transmision Cemernik - Vlasina Rid						191,000
2	WTP Vlasina - upgrade and extension to 30 l/s						375,000
3	WTP Jerma						375,000
4	Water intake Jerma						50,000
	Gross total - no VAT, no supervision						991,000
	Construction supervision - 3%						29,730
	Gross total - no VAT, construction supervision included						1,020,730

Notes:
* *Lifetime for individual project elements has been adopted as follows:*

- *Civil engineering works and structures - 50 years*
- *Equipment (mechanical, electrical, process) - 15 years*
- *Piping (underground, ductile iron pipes) - 50 years*

5.1.5.3 Option 3

Raw water transportation and treatment option 3 is based on maximal utilization of the existing structures (existing water treatment plant Vlasina, storage tank, distribution network) and water sources identified on the outskirts of Cemernik (Plavilo, Cvijina dolina, Bela voda, Ignjatova cesma) as described in option 2.

However, the available information is not sufficent to unumbiguoulsy verify available capacities and suitability of the aforesaid water sources for potable water supply in terms of water quality and capacities.

It is therefore envisaged to introduce a supplementary water source which would guarantee stable and sufficent water quanitities and continuous supply.

As a supplementary water source for potable water supply of the planned tourism development and population in the project area it is proposed to construct an intake directly from the Vlasina Lake.

This intake would be located in the zone of the existing Vlasina dam. The intake would be accompanied by a raw water pumping station that would transport abstracted raw water towards the WTP Vlasina.

Table 5.8: Main Features of raw water transmission system – Vlasina lake – WTP Vlasina

Pumping stations				
Name	**No. of pumps**	**Pump capacity (l/s)**	**Head (m)**	**Motor power (kW)**
Raw water PS "Vlasina"	2+1	12	90	15
Water intakes				
Name	**Nominal capacity (l/s)**	**Elevation (masl)**	**Notes**	
Water intake Vlasina lake	20	1.210	Intake directly from lake	
Pipelines				
Start section	**End section**	**DN (m)**	**L (m)**	**Recommended material**
Raw water PS "Vlasina"	WTP "Vlasina" –inlet chamber	150	750	PN10 HDPE/ST/DI

At this stage that new pressure main from PS Vlasina to WTP Vlasina can follow the easement of the existing main distribution pipeline DN200 in order to avoid legal issues related to the pipeline construction.

This concept would significantly improve flexibility of operation and security of supply by reducing reliance and dependance on raw water supply from the springs located west of the WTP.

The raw water PS Vlasina operation would be basically flow-controlled by means of frequency drives or other regulating elements. Inflow to the WTP should be nearly constant on a daily bases, depending on expected daily demand.

This technical concept would be very much affected by raw water quality at the source, as elaborated in the section on raw water quality and water treatment.

WTP Vlasina

In accordance with the proposed technical option 3, the overall demand would be catered for by a single water treatment facility that would be fed from two groups of water sources; springs at the outskirts of Cemernik and from Vlasina lake.

Therefore, nominal net output capacity of the WTP Vlasina would correspond to the total maximum daily demend in the project area – 50 l/s.

Raw Water Quality

Assessment of water quality of raw water from the Plavilo and Cvejina Dolina creeks is given under Option1.

Very limited available hidrological data (Hidrosanitas 1983) suggest that the Plavilo spring could provide up to 30 l/s during summer months characterized by highest water demand. This capacity could be sufficent to cover the maximal daily demand of the Vlasina area till approximately the year 2020. Additional 10 l/s could be provided by connecting the Cvejina Dolina spring with the raw water supply system transportng water to the Vlasina WTP. Based on available hydrological data and water demand projections joint capacity of Plavilo and Cvejina Dolina springs can consequently cover maximal daily water demand of the Vlasina area till approximately the year 2030.

Additional water quanities that will be required beyond 2035 could be provided either from some other springs (e.g. Bele Vode, Utrina Jevina, Bistri, Ignjatova Česma, etc.) on Čemernik mountin or from the Vlasina lake.

Other springs on Čemernik mountin
Use of springs on Čemernik mountain is prefred options given the expected better raw water quality. No data are, however available on the water quality and capacity of these springs. Given the location of these springs it could be assumed that water quality is very likely comparable with the quality of Plavilo and Cvejina Dolina springs. Additional investigations are however, necessary in the future that should provide more insignt and both water qualiaty and capacities of these springs.

Vlasina Lake
Vlasina like is an alternative raw source that can guarantee sufficent quantities for the Vlasina region even beyond the project period covered by this study. Water qulaity of this source is however expected to be inferior to water quality of springs on Čemernik mountine. In this study water quality of the Vlasina lake was assessed based on water quality data of regular yearly water quality monitoring conducted from 2004 to 2007 by Republic Hydrometerological Service of Serbia. Water samples from the Vlasina lake are taken once per year, typically during summer months at three different points (close to the dam, at the oposite side of the lake and at the lake mid point), and at three difftrent deapths (surface, mid and bottom).

Given the layout of the water supply system and specifically location of the WTP Vlasina it is expected that the future water intake will be located close to the dam. Consequently raw water qulaity assessment was focused on available data of samples taken close to the dam. In addition water the lake water quaity of samples taken at the surface and mid depth were considered as more relevant for the water quality assessment.

In general the lake water quality was found to deteriorate with water depth. As expected dissolved oxygen level was found to reduce strongly with from the lake surface to the bottom with oxygen levels at the bottom as low as 1.4-2.0 mg/l.

Turbidity of the lake water is low (1.0 - 4.5 NTU) with some small increase with an in increase of water deapth.

Raw water pH was found to fluctuate between 6.6 and 8.1, with clear increase in acidity close to the water bottom.

Raw water from the Vlasina lake has somewhat higher conductivity in comparison to springs on Čemernik mountin, but still conductivity levels are rather low (79-96 µS/cm), with water hardness typically between 60 and 80 mg $CaCO_3$.

Ammonium levels are very low at the surface, but strong increase was observed with water deapth with levels as high as 0.5 mg/l, close to lake bottom, significantly above the maximal acceptable levels set by the Serbian drinking water regulations. Nitrate levels are wery low with some small increse with the water depath. Nitrite level were aldso found to increase with the water deapth with recorded levels of 0.05 mg/l, significatly above the maximal acceptable levels.

Iron and managnese levels were found to be variable (0.02 to 1.81 mg/l and 0.01 to 1.23 mg/l, respectievly) with an increase in concentrations with water depth. These results are in line with reduced dissolved oxygen concentration with water depath. Measured iron and manganese levels in large number of samples were significantly exceeding the maximal acceptable levels set by both national and Europen drinking water qualiity regulations.

Concentration of monitored heavy metals was low found to be low with an exception of a few samples that showed nickel level exceding the maximal acceptable concentration of 20 µg/l set by the national drinking water quality standards. High nickel levels were typically found in samples taken close to the lake bottom.

Concentration of organic matter measured as Total Organic Carbon was found to be relatievly low (1.0-2.2 mg/l) with clear tendency of higher concentrations with n increase of water deapth. This increase could be explain by accumulation and dissolution of settled particulate organic matter on the lake bottom.

Measured pesticides and other organic micro pollutans were found to be bellow the level of detection suggesting that at present there is no significant pollution of the lake water caused by agricultural or industrila activities.

All measured samples show that the total β radioactivity is significantly bellow the maximal acepatble level set by drinking water regulations.

An overview of the Vlasina lake water quality is given in table 5.9.

Table 5.9: Water quality of Vlasina Lake based on samples taken from 2004 to 2007 close to the dam

Water Quality Parameter	Range
Temperature (˚C)	9.6 - 19.4
Turbidity (NTU)	1.0 - 4.5
pH	6.6 - 8.1
Colour (Pt-Co scale)	without
Conductivity at 20˚C (µS/cm)	79 - 96
TOC (mg/l)	1.0 - 2.2
Dissolved oxygen (mg/l)	1.4 - 10.3
Hardness (mg $CaCO_3$/l)	59 - 109
NH_4 (mg/l)	<0.01 - 0.48
Fe (mg/l)	0.02 - 1.81
Mn (mg/l)	0.01 - 1.23
Cu (µg/l)	<1 - 29
Cr –total (µg/l)	<1 - 6
Cr^{6+} (µg/l)	<5 / <1
Cd (µg/l)	<1.0 - 1.2
Zn (mg/l)	-
Pb (µg/l)	<1 - 1
Ni (µg/l)	<1 - 49
As (µg/l)	<1 – 2.0
NO_3 (mg/l)	0.07-0.60
NO_2 (mg/l)	<0.003-0.053
Cl (mg/l)	2.0 – 7.0
SO_4 (mg/l)	5.0 - 9.0
PO_4 – ortho (mg/l)	<0.005 – 0.03

(Source: Republic Hydrometerological Service of Serbia)

Microbiological quality

Available data on microbiological water quality show elevated concentration of total coliforms (<2.400/100ml) indicating pollution of the lake likely caused by discharge of untreated domestic water. Concentration of coliform bacteria was found to be typically highest at the mid lake depth and concentration of bacteria typically decreased at the lake surface due to disinfection effect of sun light. It can be assumed that microbiological quality of the raw water from Vlasina Lake will improve after construction of sewer system proposed in this study.

The Vlasina Lake can be seen as an attractive raw water source for drinking water production for the Vlasina area because of guaranteed quantities and its vicinity to the consumption area. At the same time available water quality data are suggesting that several water quality parameters of raw water from the Vlasina Lake include: turbidity, ammonia, nitrite, iron, manganese, microbiological quality, etc. should be improved in order to make this water suitable for public water supply.

WTP Vlasina – Proposed Process Scheme

Option 3 recommends that the existing WTP Vlasina will remain the major drinking water production facility in the Vlasina area. In order to fulfil this task the existing plant should be expanded to an operational capacity of 50 l/s in order to cover expected increase in water demand till the end of design period, the year 2035. In addition applied treatment process should be upgraded to guarantee continuous and consistent production of high quality drinking water from in the future possibly different raw water sources.

Assuming that available hydrological data (Hidrosanitas 1983) are reliable and water demand projections made in this study will materialize additional water quantities will be required. Use of additional springs on the Čemernik Mountain will imply that raw water quality will mostly likely remain the same, while use of raw water from the Vlasina Lake will require somewhat higher level of treatment due to presence of iron and specifically manganese at elevated concentrations. When establishing the treatment scheme for the upgraded WTP Vlasina an assumption was made that additional quantities from the Vlasina Lake will be required from approximately the year 2030.

The plant will initially continue to treat raw water from the Plavilo Creek. Once capacity of the Plavilo Creek is insufficient (based on very limited hydrological data expected to take place around the year 2020) the plant will receive mixture of raw water from the Plavilo and Cvejina Dolina creeks. To make use of raw water from Cvejina Dolina Creek construction of an intake and raw water pipeline connecting this spring (creek) with the existing pipeline from the Plavilo Creek will be required. These two springs have very similar water quality and consequently treatment of mixed raw water from these two springs will not require alteration of treatment process. Assuming that available hydrological data (Hidrosanitas 1983) are reliable and water demand projections made in this study will materialize additional water quantities will be required only from the year 2030. Additional raw water quantities required beyond the year 2030 (or possibly earlier if capacity of Plavilo and Cvejina Dolina springs are lower than expected or water demand will grow faster than expected) could be provided either from other springs on Čemernik mountain or from the Vlasina lake.

Use of additional springs on the Čemernik Mountain will imply that raw water quality will mostly likely remain the same, while use of raw water from the Vlasina Lake will require somewhat higher level of treatment due to presence of dissolved iron and specifically manganese at elevated concentrations.

When establishing the treatment scheme for the WTP Vlasina upgrading a conservative assumption was made that additional quantities required beyond the year 2020 will be provide from the Vlasina Lake. Use of raw Water from the Vlasina Lake will introduce the following additional challenges:

- raw water from the lake could occasioanlly have high bacteriological contamination,
- iron and manganese could be present in the lake at elevated concentrations. Even occasional presence of very high concentrations of dissolved managnese can not be excluded. It is known that consistent and very efficent removal of dissolved mnaganese could be difficult.

Design concept proposed for the upgrading of the WTP Vlasina introduced under option 1 is also applicable for option 3. The following treatment scheme is consequently proposed (Figure 5.24) for the upgraded WTP Vlasina:

- inlet chamber
- addition of CO_2,
- optional addition of coagulant,
- up flow roughing filtration
- slow sand filtration (existing and extended),
- disinfection (chlorination)
- treated water storage.

It is assumed that the proposed treatment scheme will provide very efficent removal and inactivation of bacteria in roughing and slow filtration units and through disinfection with completely replaced chlorination system. Importance of very efficent disinfection will be of high relevance when / if water from the Vlasina Lake is used as an additioanl raw water source with higher microbilogical contamination.

Efficent removal of occasionally high manganese level present in the lake water could be more challenging. It is expected that the filtrate from the roughing filters will have pH of approximately 8.0. It can be consequently assumed that high pH will be very benefical for effective manganese removal that will mainly take place in slow sand filters. Ripening period of several weeks could however be required to achieve very effective manganese removal spefiically after introducing raw water from the lake or after replacing filter media. In order to improve managanese removal also during the rippening period a layer of managnese adsorben can be introducted in the slow sand filtration filter bed. Real need for such measure should be assessed in the later project stages based on real need to use the lake water as well as the manganese concentration in it.

There is strong prefrence to provided additioanl raw water quanities from the Čemerno mountain springs given the expected higher water quality and possibility to provide gravity tansport of raw water to the WTP Vlasina.

In comparison to alternative 1 larger roughing filtration units will be required given the required plant capacity of 50 l/s. For the upgraded WTP Vlasina in option 1 two up flow roughing filter units are proposed, each consisting of several layers of coarse sand and gravel (4-20 mm) with coarser material at the filter bottom, with total filter media depth of approximately 2.5 m. Total surface area of both roughing filters of 120 m^2 (two units of $60m^2$) is suggested corresponding to maximal filtration rate of 1.5 m/h. Such filtration velocity is rather high for roughing filters but it the case of WTP Vlasina this loading rate will be reached only in the day of maximal water demand at the end of the project period. Under such extreme conditions more frequent filters cleaning (draining) could be required. Most of the time roughing filters, however, operate at much lower filtration rate. Cleaning of the roughing filters will be done under gravity, by simple opening of drain valve(s) at the bottom of the filters.

In addition to 3 existing slow sand filters, that will be rehabilitated, 2 additional slow filter units each with area of 100m^2 will be provided, resulting in total surface area of all slow sand filtration units of 350m^2. Slow sand filters will consequently operate at maximal filtration rate of approximately 0.5 m/h. This is rather high rate for slow sand filters, however such high rate will only be reached during the days of maximal water demand at the end of the project period (2035). It is also assumed that feed water will have very low turbidity due to positive effect of pre-treatment with roughing filters. To allow better maintenance of the slow sand filters sand washing area and sand storage room will be provided. This will allow washing and re-use of the dirty sand removed after a filter cleaning.

A separate plain sedimentation tank with volume of approximately 75 m^3 with a conical bottom will be provided to settle water from cleaning of roughing filters before discharge of settled water
to the sewer system. For the treatment of accumulated sludge at the bottom of settling tank the following process steps are recommended:
- Sludge thickening,
- Sludge storage (could be integrated with the sludge thickener),
- Sludge transport to a dewatering plant and its final disposal.

A gravity sludge thickener is recommended. It is expected that up to approx. 10% of dry sludge content could be produced in a gravity thickener. Either a separate sludge storage tank or a storage tank integrated with the sludge thickener should be provided for the storage of thickened sludge.

The total volume of sludge that should be dewatered will be very low and it is, consequently not reasonable to to provide a dedicated sludge dewatering unit for the Vlasina area. Consequently transport of ticked sludge to another central sludge dewatering plant in the region is recommended.

As indicated under the assessment of the existing plant complete reconstruction of the existing entrance chamber, three slow sand filter units including replacement of all pipes and valves in the filter gallery is required. To allow continuous production of drinking water also during the plant reconstruction it is suggested that the reconstruction of the existing filter should be initiated only after contraction of the new the plant components consisting of the roughing filter and extension of slow sand filter units.

WTP Vlasina – Overview of Investment and Operational Costs

It is estimated that the total investment costs associated with the upgrading of existing water treatment plant, that will include rehabilitaion of the existing plant and provision of new treatment units including new slow sand filter units will be € 750.000. It is further estimated that majority of works (75%) will consist of civil works and that remaining 25% will be mechanical and electrical works.

The water treatment process proposed in option 3 will have very limited operatioanl costs in terms of energy and chemicals.

Chemicals that will be used will most likely be limited to use of chlorine gas for disinfection and carbon dioxide dosing and refilling of calcium carbonate layer placed at the top of roughing filters. Further on occasional application of low coagulant dosages can not be excluded. Finally some use of a manganese adsorbent when/if the lake water is used as an additional raw water source could be required. Based on an indicative calculations the total chemical costs will be limited to approximately 0.04 €/m^3.

The Vlasina WTP will operate by gravity and consequently will make only marginal use of electricity for the plant operation. It is assumed that the major use of electricity will be for the heating of the buliding during winter months. It is estimated that total energy costs will not exceed 0.01 €/m3 produced water. It is assumed that continous (24 h/day) presence of an operator wil be provided. It is estimated that in total 5 staff should be employed to take care about the plant.

Similar to options 1 and 2 operation and maintenance costs will include:
- chemical costs (e.g. chlorine, calcium carbonate filter media, and possibly a coagulant and manganes adsorbent) of approximately 0.04 €/m^{3,}
- electricity estimated at 0.01 €/m^3,
- operators estimated at 5 persons available full time,
- the plant maintenance including civil works and equipment estimated at 0.5% and 3% of investment value, respectively.

Option 3 – Overview of Investment and Operational Costs

An overview of investment costs assocaited with option 3 is given in table 5.8. Costs assocaited with the upgrading and reconstrucing the Vlasina distribution system are not included.

Table 5.10: Option 3 - Overview of investment costs

1 Raw water abstraction, transmision Cemernik - Vlasina Rid								
	Description	**Unit of measure**	**Qty.**	**Civil works (€)**	**Equipment (€)**	**Piping (€)**	**TA (€)**	**Total costs (€)**
1.1	Sanitary protection and upgrade of water intake Plavilo	lump sum	1	15,000				**15,000**
1.2	Water intake Cvejina dolina	l/s	7	15,000				**15,000**
1.3	Water spring/intake Bela voda	l/s	10	20,000				**20,000**
1.4	Water spring/intake Ignjatova cesma	l/s	10	20,000				**20,000**
1.5	Raw water pipeline from intake Cvejina dolina DN100	m	400	5,200		4,800		**10,000**
1.6	Raw water pipeline from intake Bela voda DN100	m	2300	29,900		27,600		**57,500**

1.7	Raw water pipeline from intake Ignjatova cesma DN100	m	1900	24,700		22,800		**47,500**
1.8	BPT Cemernik	m^3	10	4,800	600	600		**6,000**
1	**Raw water abstraction, transmission Cemernik - Vlasina Rid**			**134,600**	**600**	**55,800**	**0**	**191,000**
2 WTP Vlasina - upgrade and extension to 50 l/s								
	Description	**Unit of measure**	**Qty.**	**Civil works (€)**	**Equipment (€)**	**Piping (€)**	**TA (€)**	**Total costs (€)**
2.1	Upgrade of the existing WTP Vlasina	l/s	15	45,000	105,000			**150,000**
2.2	Extension of the WTP Vlasina for additional capacity of 35 l/s net	l/s	35	210,000	315,000			**525,000**
2	**WTP Vlasina - Rehabilitation and extension to 50 l/s**			**255,000**	**420,000**			**675,000**
3 Raw water intake and transmission from Vlasina lake								
	Description	**Unit of measure**	**Qty.**	**Civil works (€)**	**Equipment (€)**	**Piping (€)**	**TA (€)**	**Total costs (€)**
3.1	Raw water intake - Vlasina lake	lupm sum	1	25,000				**25,000**
3.2	Raw water PS Vlasina	kW	30	36,000	63,750	11,250		**111,000**
3.3	Forcemain to WTP Vlasina DN150	m	750	9,750	22,500			**32,250**
3	**Raw water intake and transmission from Vlasina lake**			**70,750**	**86,250**	**11,250**		**168,250**
				Civil works (€)	**Equipment (€)**	**Piping (€)**	**TA (€)**	**Total costs (€)**
	Grand total per type of works			**460,350**	**506,850**	**67,050**	**0**	**1,034,250**

INVESTMENT COST SUMMARY - PHASE 1							
1	**Raw water abstraction, transmision Cemernik - Vlasina Rid**						**191,000**
2	**WTP Vlasina - upgrade and extension to 50 l/s**						**675,000**
3	**Raw water intake and transmission from Vlasina lake**						**168,250**
	Gross total - no VAT, no supervision						**1,034,250**

	Construction supervision - 3%						**31,028**
	Gross total - no VAT, construction supervision included						**1,065,278**

Notes:

* *Lifetime for individual project elements has been adopted as follows:*

- *Civil engineering works and structures - 50 years*
- *Equipment (mechanical, electrical, process) - 15 years*
- *Piping (underground, ductile iron pipes) - 50 years*

5.1.6 Water distribution system

5.1.6.1 Approach and methodology

This chapter outlines general approach and methodology applied for technical analysis of the considered water distribution system.

The following major groups of activities were carried out:

- Data collection, systematization and analysis;
- Site reconnaissance;
- Analysis and assessment of the existing water utilities systems;
- Identification of the major deficiencies of the existing system;
- Reconsideration and verification of the development projections (population, tourism, etc.);
- Development of the future technical concepts of the water supply system;
- Preparation of the proposal for phased Project implementation;

Data collection, systematization and analysis

The data collection, systematization and analysis were essential during the project initiation in order to get thorough understanding of the existing water utilities systems, their features, current operational status, and deficiencies, potentials for their future use and/or rehabilitation and upgrade.

During this phase of the design preparation, all technical documentation related to the existing systems that was provided by the local authorities and water utilities companies was reviewed and the information considered of importance for this project design recorded and systematized. The documentation reviewed is listed in the list of references and documentation.

The data collected were processed and systematized to be used in this project design, and include the following:

- **Topographical data** – topographical maps of the Project Area, including the locations of potential new water sources, in appropriate digital format where available. If available the topographic data were in their original electronic format and imported in a corresponding CAD program used for preparation of the design. Topographic maps obtained as a hard copy were transformed in the appropriate electronic format in several steps: scanning, cropping, rectifying, geo-orienting, importing in the CAD program used for the design preparation.
- **Project area limits and urban master plan** – include Tourist Centre Vlasina as well as two local rural centres; settlements Klisura and Bozica.
- **Data on the existing utilities systems** include the following:
  - ⇒ **Systems general layout;**
  - ⇒ **Water balances**, water production;
  - ⇒ **Hydro-geological data** (water sources locations and characteristics, capacities, water quality, etc.)
  - ⇒ **System component features**: geometry, hydraulic, structural, equipment (mechanical process, electrical, etc.);
  - ⇒ **Operational features**: operational status, operational problems, control and instrumentation;
- **Technical documentation** on the existing systems showing historic evolution of the system, planning that has been respected so far, etc.;

Site reconnaissance

The phase of site reconnaissance was very important for the project design to verify on site the information originally included in the technical documentation, to supplement said information with additional data recorded on site, to check exact status of the structures and equipment, to verify operational status of the components, to record current operational problems, to prepare and systematize photo-documentation.

The comprehensive site reconnaissance for the purpose of this design was carried out in November 2008 – March 2009, and included all major components of the water supply and sanitary sewerage system.

Analysis and assessment of the existing water utilities systems

The analysis and assessment of the existing water utilities systems were carried out after all relevant data mentioned hereinbefore were collected, verified and systematized.

The adequacy and ability of the systems to provide required level of service to the population within the Project Area was evaluated based on the system layout and coverage of the project area, characteristics of the system components, recorded operational status, operational parameters (hydraulic parameters, water quality parameters, hydro-geological parameters).

Depending on the system nature, available data and where applicable, the assessment of the existing water supply and sanitary sewerage utilities was carried out by using appropriate software packages for mathematical modelling, analysis and design of these systems. Detail description of the software packages used and corresponding methodology are described hereinafter in the report.

Identification of the major deficiencies of the existing system

Based on the performed analysis and assessment of the existing water utilities systems the key system deficiencies in terms of the Project Area coverage, provision of required level of service, system safety and reliability, major water quality health-related issues, major sanitary and environmental protection issues were individually addressed, followed by the corresponding rehabilitation/improvement measures.

Development of the future technical concepts of the water supply and sanitary sewerage systems

Future technical concepts of the water supply and sanitary sewerage systems have been devised based on the assessment of the existing systems adequacy, potentials and deficiencies versus the requirements for adequate level of service (water supply, sanitary sewerage) in the areas already served, or in the areas yet to be connected to these systems.

The proposed future concept of the sanitary sewerage system was affected by the existing system layout, natural – topographic features and planned urban development of the Project Area.

Computer-aided design was used in development of the future system concepts, both for hydraulic analysis and design of the water utilities systems and also for graphical presentation of the analysis results and of the proposed system.

Preparation of the proposal for phased project implementation

In order to assist phased implementation of the Project technical measures proposed were prioritized, i.e. grouped in major development phases up until the design horizon – year 2035. The most important, priority measures intended to significantly improve current situation are set at the first priority. Further system development was then further devised in relation to estimated increase of the demand, connection of new areas, inclusion of new water sources, etc.

5.1.6.2 Design criteria

This section presents basic design criteria applicable for water distribution system.

Table 5.11: Design Criteria

WATER DEMAND			
Component	**Design Criteria**		
Population and tourism projections	**Population and tourism**		
	Based on the Spatial Plan of Special Purpose Area Vlasina, Master plan with business plan of tourism development at Vlasina Lake, Tourism development proposal in this study		
Average net water demand	**Average water demand**		
	User category	**Unit rate (l/capita/day)**	
	Permanent population, weekend houses	150	
	Tourists in planned tourist accommodation	300	
Maximum daily demand $Q_{max\ day} = k_{max\ day} \times Q_{average}$	**Maximum daily demand Factor – $k_{max\ day}$**		
	Users category	**Factor $k_{max\ day}$**	
	Population/tourist accommodation	1.50	
Fire fighting demand Q_{ff} [l/s]	**Depends on the size of urban agglomeration**		
	Population	**No of concurrent fires**	**Q (l/s) per fire**
	Up to 5.000	1	10
	5.000 – 10.000	1	15
	10.000 – 25.000	2	20
	25.000 – 50.000	2	25

WATER TRANSPORT AND DISTRIBUTION SYSTEM	
Component	**Design Criteria**
	STORAGE TANKS
Ground water tanks (GWT)	Volume should be sufficient to balance, on a daily basis, water inflow (from water source, treatment plant) and further transfer – to transmission mains, distribution, plus fire-fighting volume.
Elevated water tanks (EWT)	Volume to balance demand fluctuations in the network plus fire-fighting volume – generally 30-40% of the maximum day demand.
	TRANSPORT
Suction pipes	Diameter sizing according to the pump NPSH. Net positive suction head. Recommended V ≤1.0 m/s
Pumps	Capacity (Q, H) Q=required water demand H= $h_{st} + H_f$ + (3 – 5m). Variable running speed module for distribution without EWT One stand by pump 25% to 100% not less than 1.
Utilities (fittings) downstream the pump	Minimum selection as following: Gate valve Non-return valve Flow meter Pressure gage

	Sampling tap Air valve
Roughness coefficient	According to the pipe specification and formula
Transport line	Max velocity (V_{max}) of water V_{max}<2.5 m/s, V_{min} ≥ 0.6 m/s. Materials: DI or plastics. Consideration must be taken for aggressive soil and economic comparison. Double air ventilation valves (DAV) to the peak points and/or according to the water hammer study. Single Air ventilation valves located on the rising distance of 500 – 1000 m if there are no DAV. Wash-outs (WO) at the low points. Gate valves (isolation valves) distance shall be less than 5.0 km. Minimum pressure 5 meters above the peak point. Parallel lines if the future need is more than the existing can carry on. Pipeline corridor must be wide enough for the future construction. Construction of two lines can be divided to two stages.
	DISTRIBUTION
	Pressure head
Minimum head	15 mwc, 1,5 bar for defined duration during the day (0,5 h)
Maximum head	50 mwc, 5,0 bar
Minimum head – fire-fighting requirement for both internal and external hydrants	25 mwc, 2,5 bar (external network, or internal on the top floor)
	Network pipelines and utilities
Distribution lines DN > 100mm	Max velocity (V_{max}) of flow <2.5 m/s (recommended) Min velocity (V_{min}) of flow>0.6 m/s (recommended) Pipe materials DN150–400mm – plastics (recommended) Pipe materials DN>400mm – DI, ST (recommended) Consideration must be taken for aggressive soil and underground water, economic comparison. No house connections on lines DN≥300mm (recommended) Trust blocks according to the design pressure and soil characteristics.
Secondary distribution lines DN <100mm	Max velocity (V_{max}) of flow <1.5 m/s. Min velocity (V_{min}) of flow >0.1 m/s. Pipe materials – plastics (recommended) Trust blocks according to the design pressure and soil characteristics.
Fire-fighting requirements	DN to satisfy pressure and flow requirements, but for external hydrant network not less than 100mm.
Water meters	Must be installed on all service connections
Cover depth	Under traffic area ≥ 1.0 m Outside traffic area ≥ 0.8 m The cover depth must follow the manufactures recommendations and protection requirements (external forces, frost protection).
Gradient	Not less than 0.1 %

5.1.6.3

Technical Concept

This section outlines basic concept of the proposed water distribution system. Considered water distribution system defines primary system components including storage tanks, main distribution pipelines, pumping stations and control and regulation elements. Secondary distribution network in Project Area is out of the scope of this study.

Technical concept of the considered water distribution system has been created by means of mathematical/hydraulic modelling of system operation for representative hydraulic loadings.

Corresponding hydraulic model was built including the following considerations:

- Maximum day water demand allocation per users in the Project Area
- Network geometry data including physical data for existing and planned pipelines, junctions/nodes, water tanks, pumping stations, flow control valves etc.
- Diurnal demand patterns for different consumer categories, i.e. villages, tourists.
- Formation of pressure zones

Detailed population data analysis, consumer categories identification, specific demand and maximum day factors assessment, water losses prediction have been described earlier. Calculated maximum daily demand has been allocated to network model nodes based on the geographical location of consumers.

Physical data; such as length, pipe size, hydraulic roughness, pipe material were prepared for **pipelines**, elevations, demands and diurnal patterns for **nodes**; volumes, minimum and top water levels for **water storage tanks**; pump curves and controls for **pumping stations;** valve throttling rate or required flow for **control valves**.

The water distribution system has been designed so that it could be implemented in two phases (Vlasina Rid and Vlasina Okruglica in the first, and Vlasina Stojkoviceva in the second phase), the hydraulic analysis has been performed for the whole system, including network components in all three zones.

The hydraulic model presents only the system components that are common for all three alternatives of raw water transport and treatment, and the boundary hydraulic conditions such as water inflow or levels have been set at connecting points – tanks and pumping stations.

Due to topographical and hydraulic reasons, consumers have been allocated to five pressure zones, which is characteristic for mountain type water supply system.

The hydraulic analysis has been carried out for the target year, based on water demand forecast, maximum connection and tourist occupancy rates, as the most demanding water supply scenario for which satisfactory level of service has to be ensured.

The consumers have been grouped into five pressure zones, defined below:

1. Pressure zone 1 – elevations below 1260 masl
2. Pressure zone 2 – elevations between 1260 - 1310 masl
3. Pressure zone 3 – elevations between 1310 - 1360 masl
4. Pressure zone 4 – elevations between 1360 - 1410 masl
5. Pressure zone 5 – elevations between 1410 - 1460 masl

The abovementioned pressure zones are by definition provided by adequate elevated water storage tanks, except for small, isolated groups of consumers which are to be supplied by local pumping stations with pneumatic pressure vessels. These elevated tanks provide stable working pressures in the pressure zones, cover demand variations and ensure supply reserve for normal and irregular operational conditions.

Proposed layout of the water distribution system and its components are shown on the attached general layout drawing. Water distribution system components are elaborated hereinafter.

5.1.6.4 Mains and distribution network

The inventory of main transport and distribution water mains is presented in the following tables:

Table 5.12: Overview of main distribution pipelines – Phase 1

No.	DN (mm)	L (m)	Share (%)
1	80	60.700	70,5
2	150	830	1,0
3	200	24.525	28,5
Total		**86.055**	**100,0**

Table 5.13: Overview of main distribution pipelines – Phase 2

No.	DN (mm)	L (m)	Share (%)
1	80	18.506	60,8
2	150	11.910	39,2
Total		30.416	100,0

5.1.6.5 Water storage tanks

Water distribution tanks have been designed to cater for variations of daily demand, to provide adequate hydraulic head and sufficient pressures in the system, and enable transfer of water to higher pressure zones.

The following table provides an overview of the water distribution tanks in the system, their main characteristics and planned construction phase.

Table 5.14: Characteristics of water storage tanks

No.	Label	Pressure Zone	Total Active Volume V (m³)	Bottom Elevation (masl)	Top Water Level (masl)	Remark/Phase
1	WT 2 - II	2	250	1,331.0	1,334.0	1
2	WT 3 - III	3	100	1,385.0	1,388.0	1
3	WTP Vlasina	2	500	1,292.5	1,295.0	Existing
4	WT 6 - III	3	100	1,385.0	1,388.0	1
5	WT 4 - II	2	500	1,335.0	1,339.0	1
6	WT 6 - III	3	100	1,385.0	1,388.0	1
7	WT 7 - IV	4	250	1,435.0	1,438.0	1
8	WT 8 - I	1	100	1,285.0	1,288.0	1
9	WT 9 - II	2	250	1,327.0	1,330.0	1
10	WT 10 - III	3	100	1,385.0	1,388.0	1
11	WT 11 - V	5	50	1,485.0	1,488.0	1
Total Phase 1:			2300 (500+1800)			
12	WT 13 - IV	4	50	1,435.0	1,438.0	2
13	WT 12 - III	3	50	1,385.0	1,388.0	2
14	WT 14 - I	1	50	1,285.0	1,288.0	2
Total Phase 2:			150			

5.1.6.6

Pumping facilities

Pumping stations provide extra head in the water supply system and transfer water from lower to higher network zones. The capacity of the pumping stations equals to maximum daily demand of the consumption area, whereas the difference between the peak hour and maximum daily demand is supplied from water distribution tanks. In almost all cases the consumers are located between the pumping station and water storage tank on the opposite side. Each pumping station has one duty and one standby pump. The hydraulic characteristics of the pumping stations are given below:

Table 5.15: Overview of pumping station characteristics

No.	Label	Pressure Zone	Design flow rate (l/s)	Design Head (m)	Motor Power (kW)	Remark/ Phase
1	**PS3 - III**	3	1.65	58	2.2	1
2	**PS4 - III**	3	1.65	101	4.0	1
3	**PS WTP Vlasina**	2	43.0	58	37.0	1
4	**PS5 - III**	3	2.5	54	3.0	1
5	**PS6 - IV**	4	10.5	141	30.0	1
6	**PS7 - V**	5	8.0	80	11.0	1
7	**PS8 - III**	3	3.2	25	1.5	1
8	**PS9 - III**	3	2.9	61	4.0	1
9	**PS10 - IV**	4	0.6	41	0.55	1
10	**PS11 - V**	5	1.2	105	3.0	1
11	**PS12 - III**	3	1.2	95	3.0	1
12	**PS15 - III**	3	3.2	45	3.0	1
13	**PS13 - III**	3	2.1	64	3.0	2
14	**PS14 - IV**	4	0.5	53	0.9	2

5.1.6.7

Hydraulic analysis

Hydraulic analysis and assessment of the existing water supply system

Approach and methodology – hydraulic/mathematical modelling of the system

This chapter outlines basic features of the software package EPANET used for the analysis of the existing water supply system and planning and design of the future water supply system. EPANET has become a world-wide standard engineering tool for analysis and planning of water supply distribution system.

WHAT IS EPANET

EPANET is a computer program that performs extended period simulation of hydraulic and water quality behaviour within pressurized pipe networks. A network consists of pipes, nodes (pipe junctions), pumps, valves and storage tanks or reservoirs. EPANET tracks the flow of water in each pipe, the pressure at each node, the height of water in each tank, and the concentration of a chemical species throughout the network during a simulation period comprised of multiple time steps. In addition to chemical species, water age and source tracing can also be simulated.

EPANET is designed to be a research tool for improving our understanding of the movement and fate of drinking water constituents within distribution systems. It can be used for many different kinds of applications in distribution systems analysis.

Sampling program design, hydraulic model calibration, chlorine residual analysis, and consumer exposure assessment are some examples. EPANET can help assess alternative management strategies for improving water quality throughout a system.

These can include:

- altering source utilization within multiple source systems,
- altering pumping and tank filling/emptying schedules,
- use of satellite treatment, such as re-chlorination at storage tanks,
- targeted pipe cleaning and replacement.

Running under Windows, EPANET provides an integrated environment for editing network input data, running hydraulic and water quality simulations, and viewing the results in a variety of formats. These include colour-coded network maps, data tables, time series graphs, and contour plots.

HYDRAULIC MODELING CAPABILITIES

Full-featured and accurate hydraulic modelling is a prerequisite for doing effective water quality modelling. EPANET contains a state-of-the-art hydraulic analysis engine that includes the following capabilities:

- places no limit on the size of the network that can be analyzed
- computes friction headloss using the Hazen-Williams, Darcy-Weisbach, or Chezy-Manning formulas
- includes minor head losses for bends, fittings, etc.
- models constant or variable speed pumps
- computes pumping energy and cost
- models various types of valves including shutoff, check, pressure regulating, and flow control valves
- allows storage tanks to have any shape (i.e., diameter can vary with height)
- considers multiple demand categories at nodes, each with its own pattern of time variation
- models pressure-dependent flow issuing from emitters (sprinkler heads)
- can base system operation on both simple tank level or timer controls and on complex rule-based controls.

WATER QUALITY MODELING CAPABILITIES

In addition to hydraulic modelling, EPANET provides the following water quality modelling capabilities:

- models the movement of a non-reactive tracer material through the network over time
- models the movement and fate of a reactive material as it grows (e.g., a disinfection by-product) or decays (e.g., chlorine residual) with time
- models the age of water throughout a network
- tracks the percent of flow from a given node reaching all other nodes over time
- models reactions both in the bulk flow and at the pipe wall
- uses n-th order kinetics to model reactions in the bulk flow
- uses zero or first order kinetics to model reactions at the pipe wall
- accounts for mass transfer limitations when modelling pipe wall reactions
- allows growth or decay reactions to proceed up to a limiting concentration
- employs global reaction rate coefficients that can be modified on a pipe-by-pipe basis
- allows wall reaction rate coefficients to be correlated to pipe roughness

- allows for time-varying concentration or mass inputs at any location in the network
- models storage tanks as being either complete mix, plug flow, or two-compartment reactors.

By employing these features, EPANET can study such water quality phenomena as:

- blending water from different sources
- age of water throughout a system
- loss of chlorine residuals
- growth of disinfection by-products
- tracking contaminant propagation events.

STEPS IN USING EPANET

One typically carries out the following steps when using EPANET to model a water distribution system:

- **Draw a network representation of your distribution system or import a basic description of the network placed in a text file.**
- **Edit the properties of the objects that make up the system.**
- **Describe how the system is operated.**
- **Select a set of analysis options.**
- **Run a hydraulic/water quality analysis.**
- **View the results of the analysis.**

Basic assumptions for the hydraulic analysis

Technical evaluation, design and sizing of the water supply system have been carried out taking into account positive engineering principles given below. Also, as the planned system is very scattered and divided into five pressure zones, it was very important to properly set all boundary conditions, hydraulic parameters and controls. The following assumptions have been made:

- Assuming that the pipe material will be PVC, HDPE or ductile iron which is usual for projects of this kind, hydraulic roughness in Colebrook-White equation has been adopted as k=0.1 mm.

- Minimum service pressure has been set to 15 m, this is considered satisfactory for consumers living in villages and small towns.

- Pipe diameter of main transport pipelines has been designed taking into consideration long lengths of pipelines i. e. low head losses, pressure fluctuations within 1 bar, successful filling of service tanks.

- Minimum distribution pipe size has been adopted as 80 mm.

- Peak hour factor for demand variation in Vlasina Rid has been set to K_{max}=1.8, and K_{min}=0.24, whereas in Vlasina Okruglica and Vlasina Stojkoviceva the values for K_{max}=2.0, and K_{min}=0.25.

- The balancing volume/capacity of service water tanks has to be around 20% of the maximum day consumption. They also have to provide an extra emergency storage for 2-8 hours of maximum daily demand. Together the two volumes equal to 30-40% of the maximum daily consumption. However, as many tanks provide balancing and water storage of isolated small consumption areas greater values have been adopted, resulting in total tank volume of around 50% of maximum day demand. This will provide greater emergency storage capacity.

- The capacity of existing and proposed water storage tanks should allow for balancing of diurnal variation of consumption.

- Water storage tanks have common inlet and outlet pipe, as they are located on the opposite side of the consumption area and provide the difference in flow during peak hour demand. All water storage tanks should have bottom inlets, to provide extra head difference i.e. flow capacity or lower pumping station head bearing in mind long pipeline lengths and friction losses in the water supply system.

- Flow velocities should not allow sedimentation in pipes.

- Pumping stations are designed with capacity equal to maximum daily demand or peak hour demand based on their location/role in the water supply system.

Results of the hydraulic analysis

The hydraulic analysis of the water supply distribution system was carried out by means of continual 24-hour simulation of the system operation for the maximum daily (on average during 24 hours) plus the maximum hourly water demand, as defined hereinbefore.

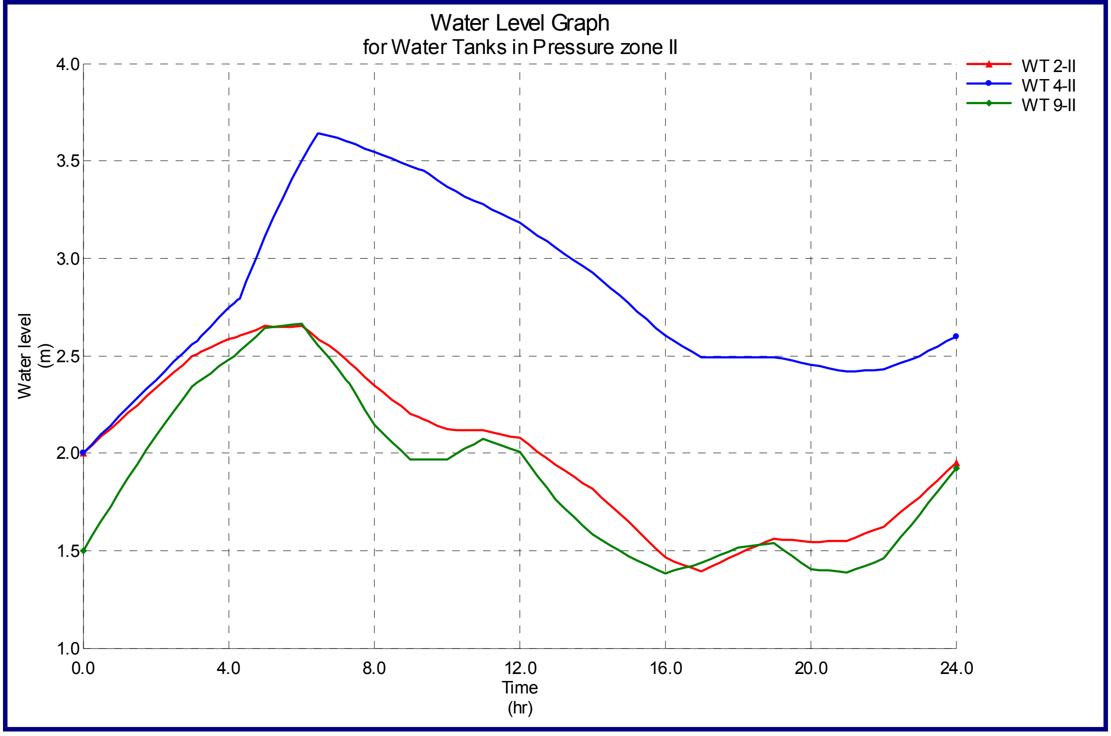
The main objective of the performed hydraulic calculations of the Vlasina Water Supply System was setting the general location of storage tanks, sizing of transport pipelines, pumping stations and water tanks, as well as providing sufficient pressures in the network. In terms of hydraulic parameters and layout, the system is very complex with five pressure zones and many hydraulic structures and facilities – pumping stations, pressure reducing valves, flow control valves.

Results of the hydraulic calculation are presented in the form of graphs, tables and charts. As the hydraulic model was used for performing extended period simulation (24h), the results of hydraulic parameters are presented as graphs. In case of a steady-state calculation, results for pipes and nodes are presented in the form of tables. The main hydraulic parameters that are subject to analysis and used in decision making are levels in water tanks, flow rates in pipes and pressures / hydraulic grade at nodes.

In terms of hydraulic capacity and bulk water supply the backbone of the system are the facilities in the second pressure zone (elevations from 1260 to 1310 masl). Because of the large network coverage, a need for three water storage tanks was identified. The first one is in the western part of the lake in Vlasina Rid (WT 4-II – 500m^3), the second tank is in the northern part of Vlasina Rid (WT 2-II – 250 m^3), and the third tank is in Vlasina Okruglica in the south (WT 9-II – 250 m^3). These water tanks serve as transfer points for higher pressure zones, and provide steady hydraulic conditions in the second (biggest) pressure zone. The WT 4 – II tank is filled from the water treatment plant Vlasina via a pumping station PS WTP Vlasina. Water is further distributed to consumers and tanks in the north and south.

The following graph presents water level variations for three water tanks in the main pressure zone. The graph shows characteristic diurnal variation of water level, i.e. filling during the night and emptying during afternoon. The design volumes have also been correctly determined.

Figure 5.25: Water Level Graph for Water Tanks in Pressure zone II – horizon 2035



The pipelines connecting the abovementioned three main system tanks, supply potable water to consumers in the pressure zone II, having a twofold role of a distribution main and a transport main.

The following graph presents flow rates in the main pipeline ring around the lake, in the pressure zone II. In general, an average daily flow of around 15 l/s is transported to the northern and southern water storage tanks. The two bottom lines indicate flow rate in tank's feed pipelines, with positive flows (tank filling) during low demands and negative flows during peak water demand (tank emptying). The flow velocities in pipelines are somewhat lower than optimal, but this is normal trade-off for long pipeline runs and the need to reduce friction head losses, and provide sufficient capacity of the transport system.

The pressures in the main pressure zone II are also within optimal limits, not exceeding 7 bar in the consumption area. It can be noticed also, that the there are no significant pressure fluctuations during a 24-hour simulation, justifying the location and volume of the water storage tanks.

The hydraulic calculation showed that pressure reducing valves may be needed in the Vlasina Okruglica area, as indicated on the layout map.

Figure 5.26: Flow Rate Graph for Pipeline DN225 in Pressure zone II – horizon 2035

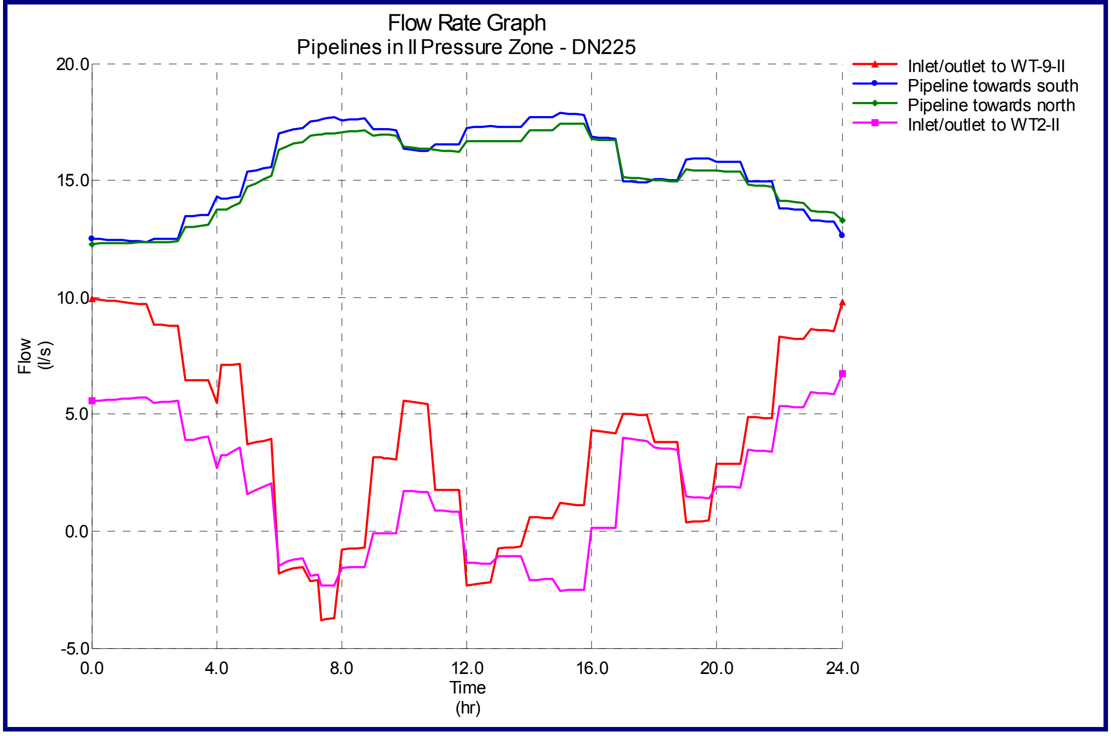
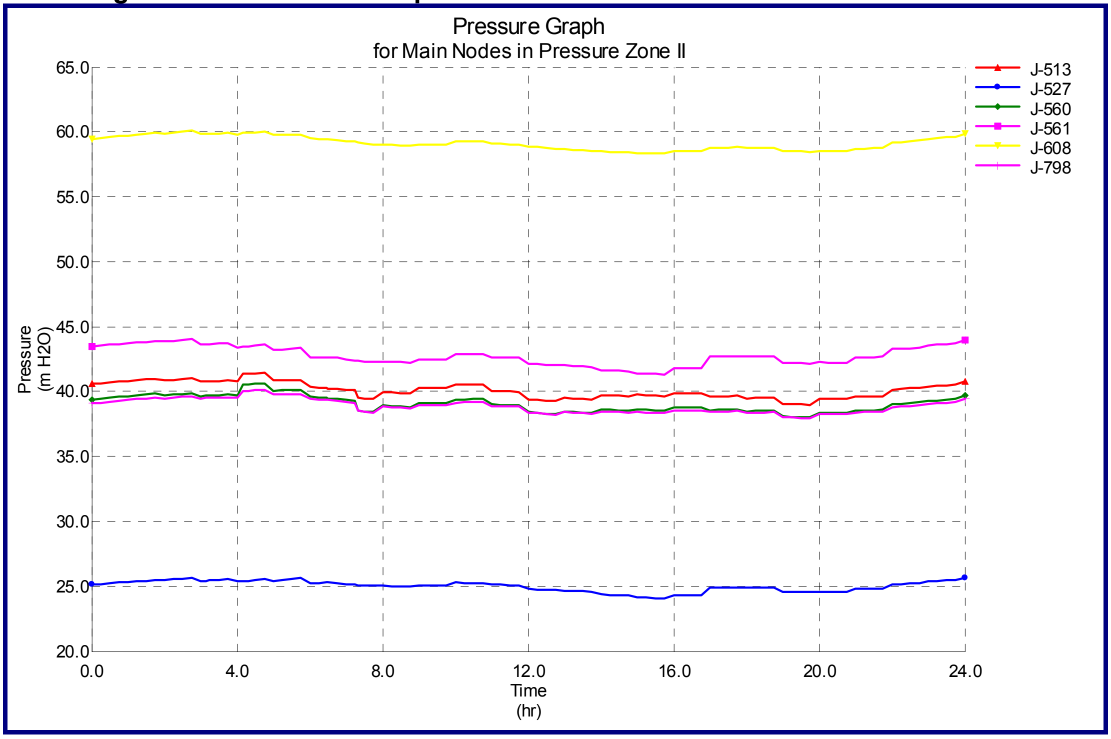


Figure 5.27: Pressure Graph for Nodes in Pressure zone II – horizon 2035



The pumping stations are designed for the maximum day flow capacity in the target year, and operate based on water level in the control water tank or follow the water demand pattern by means of variable frequency drive motors. The hydraulic model results have shown that all pumping stations have been properly sized for the horizon 2035. The operation of pumping stations in the water supply system will have more frequent on/off cycles in the period before the system reaches maximum demand, and this is another reason why control by frequency drives is recommended.

Figure 5.28: Main PS Operation in Vlasina Rid – horizon 2035

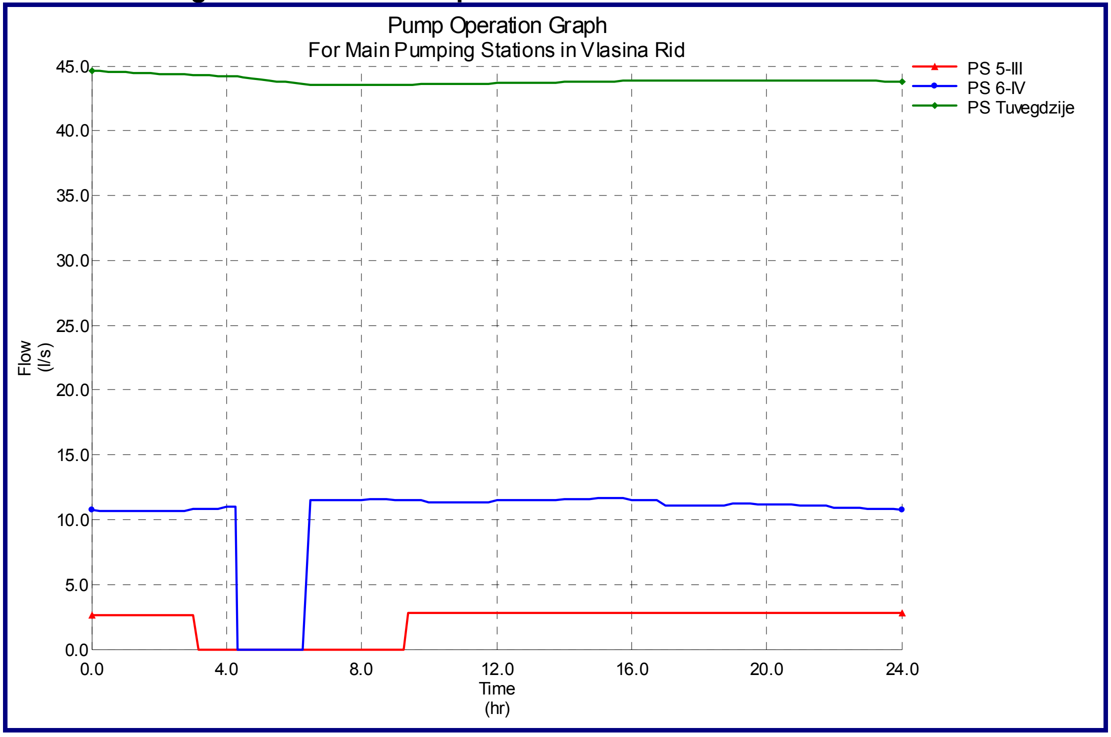
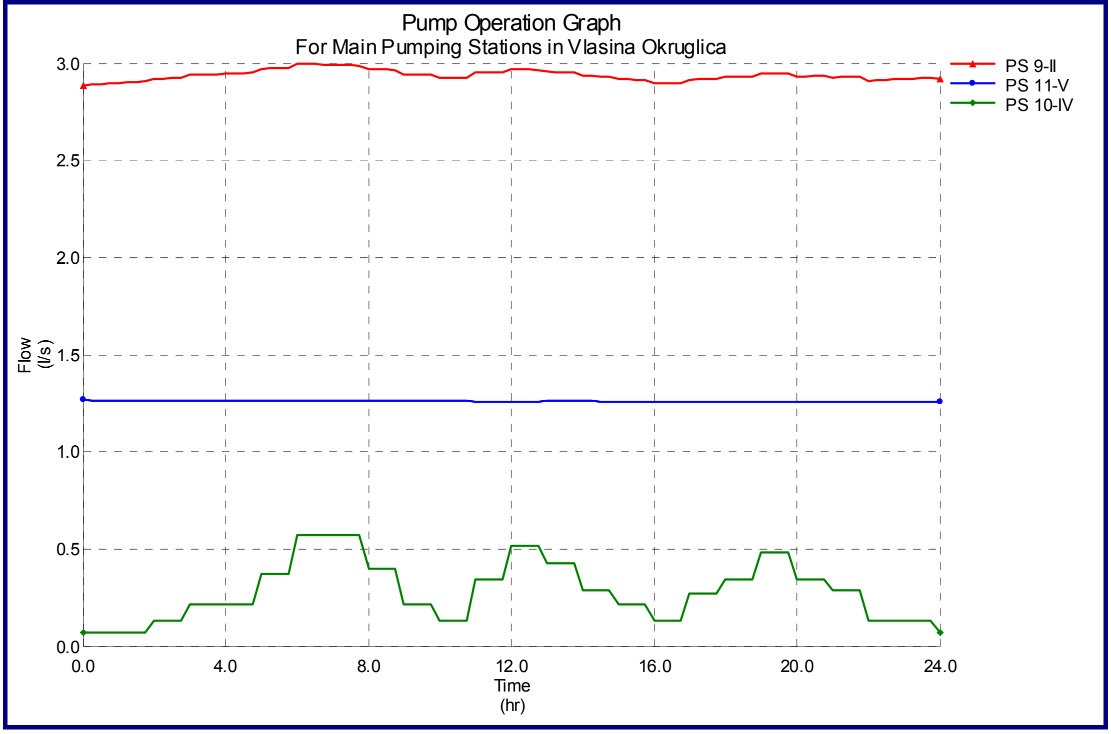


Figure 5.29 - Main PS Operation in Vlasina Okruglica – horizon 2035



5.1.6.8

Water distribution system – assessment of investment costs

Table 5.16: Water Distribution System – Overview of Investment Costs

	Description	Unit of measure	Qty.	Civil works (€)	Equipment (€)	Piping (€)	TA (€)	Total costs (€)
1 Investigation works and design								
1.1	Vlasina Tourist Centre - Development of Detail Urban Plans						250,000	**250,000**
1.2	Water supply of Vlasina Tourist Centre - Preliminary Design						90,000	**90,000**
1.3	Water supply of Vlasina Tourist Centre - Detail Design						190,000	**190,000**
1.4	Investigation works hydrology, hydro-geology, water quality of water sources						70,000	**70,000**
1.5	Sanitary protection, upgrade of water sources - Detail Design						25,000	**25,000**
1	**Investigation works & design - Sub-total**						**625,000**	**625,000**

	Description	Unit of measure	Qty.	Civil works (€)	Equipment (€)	Piping (€)	TA (€)	Total costs (€)
4 Water storage tanks - Phase 1								
4.1	**WT 2 - II**	m^3	250	90,000	11,250	11,250		**112,500**
4.2	**WT 3 - III**	m^3	100	36,000	4,500	4,500		**45,000**
4.3	**WT 6 - III**	m^3	100	36,000	4,500	4,500		**45,000**
4.4	**WT 4 - II**	m^3	500	180,000	22,500	22,500		**225,000**
4.5	**WT 6 - III**	m^3	100	36,000	4,500	4,500		**45,000**
4.6	**WT 7 - IV**	m^3	250	90,000	11,250	11,250		**112,500**
4.7	**WT 8 - I**	m^3	100	36,000	4,500	4,500		**45,000**
4.8	**WT 9 - II**	m^3	250	90,000	11,250	11,250		**112,500**
4.9	**WT 10 - III**	m^3	100	36,000	4,500	4,500		**45,000**
4.1	**WT 11 - V**	m^3	50	18,000	2,250	2,250		**22,500**
4	**Water storage tanks - Phase 1 - Total**		**1,800**	**648,000**	**81,000**	**81,000**	**0**	**810,000**

	Description	Unit of measure	Qty.	Civil works (€)	Equipment (€)	Piping (€)	TA (€)	Total costs (€)
5 Pumping stations - Phase 1								
5.1	**PS3 - III**	kW	1.65	18,000	7,013	1,238		**26,250**
5.2	**PS4 - III**	kW	1.65	18,000	7,013	1,238		**26,250**
5.3	**PS WTP Vlasina**	kW	43	36,000	91,375	16,125		**143,500**
5.4	**PS5 - III**	kW	2.5	18,000	10,625	1,875		**30,500**

5.5	**PS6 - IV**	kW	10.5	36,000	26,775	4,725		**67,500**
5.6	**PS7 - V**	kW	8	36,000	20,400	3,600		**60,000**
5.7	**PS8 - III**	kW	3.2	18,000	13,600	2,400		**34,000**
5.8	**PS9 - III**	kW	2.9	18,000	12,325	2,175		**32,500**
5.9	**PS10 - IV**	kW	0.6	18,000	2,550	450		**21,000**
5.10	**PS11 - V**	kW	1.2	18,000	5,100	900		**24,000**
5.11	**PS12 - III**	kW	1.2	18,000	5,100	900		**24,000**
5	**Pumping stations - Phase 1**			**252,000**	**201,875**	**35,625**		**489,500**

6 Main distribution pipelines - Phase 1

	Description	Unit of measure	Qty.	Civil works (€)	Equipment (€)	Piping (€)	TA (€)	Total costs (€)
6.1	Main distribution pipelines - DN200	m	24,525	318,825		1,348,875		**1,667,700**
6.2	Main distribution pipelines - DN150	m	830	10,790		24,900		**35,690**
6.3	Main distribution pipelines - DN80	m	60,700	789,100		485,600		**1,274,700**
6	**Main distribution pipelines Phase 1 - Total**			**1,118,715**	**0**	**1,859,375**		**2,978,090**

8 Water storage tanks - Phase 2

	Description	Unit of measure	Qty.	Civil works (€)	Equipment (€)	Piping (€)	TA (€)	Total costs (€)
8.1	**WT 13 - IV**	m^3	50	18,000	2,250	2,250		**22,500**
8.2	**WT 12 - III**	m^3	50	18,000	2,250	2,250		**22,500**
8.3	**WT 14 - I**	m^3	50	18,000	2,250	2,250		**22,500**
8	**Water storage tanks - Phase 2 - Total**		**150**	**54,000**	**6,750**	**6,750**	**0**	**67,500**

9 Pumping stations - Phase 2

	Description	Unit of measure	Qty.	Civil works (€)	Equipment (€)	Piping (€)	TA (€)	Total costs (€)
9.1	**PS11 - V**	kW	2.1	18,000	8,925	1,575		**28,500**
9.2	**PS12 - III**	kW	0.5	18,000	2,125	375		**20,500**
9	**Pumping stations - Phase 2**			**36,000**	**11,050**	**1,575**		**49,000**

10 Main distribution pipelines - Phase 2

	Description	Unit of measure	Qty.	Civil works (€)	Equipment (€)	Piping (€)	TA (€)	Total costs (€)
10.1	Main distribution pipelines - DN150	m	11,910	154,830		357,300		**512,130**
10.2	Main distribution pipelines - DN80	m	18,506	240,578		148,048		**388,626**
10	**Main distribution pipelines Phase 2 - Total**			**395,408**	**0**	**505,348**		**900,756**

				Civil works (€)	Equipment (€)	Piping (€)	TA (€)	Total costs (€)
	Grand total per type of works - Phase 1			2,018,715	282,875	1,976,000	625,000	4,902,590
	Grand total per type of works - Phase 2			485,408	17,800	513,673	0	1,017,256
	Grand total per type of works			2,504,123	300,675	2,489,673	625,000	5,919,471
				INVESTMENT COST SUMMARY - PHASE 1				
1	Investigation works and design							625,000
4	Water storage tanks - Phase 1							810,000
5	Pumping stations - Phase 1							489,500
6	Main distribution pipelines - Phase 1							2,978,090
	Gross total - no VAT, no supervision							4,902,590
	Construction supervision - 3%							147,078
	Gross total - no VAT, construction supervision included							5,049,668
				INVESTMENT COST SUMMARY - PHASE 2				
8	Water storage tanks - Phase 2							67,500
9	Pumping stations - Phase 2							49,000
10	Main distribution pipelines - Phase 2							900,756
	Gross total - no VAT, no supervision							1,017,256
	Construction supervision - 3%							30,518
	Gross total - no VAT, construction supervision included							1,047,774

Notes:

* *Lifetime for individual project elements has been adopted as follows:*

- *Civil engineering works and structures - 50 years*
- *Equipment (mechanical, electrical, process) - 15 years*
- *Piping (underground, ductile iron pipes) - 50 years*

5.2 Wastewater collection, treatment and disposal

5.2.1 Wastewater collection

5.2.1.1 Current status

The Vlasina lake area and catchment have been endorsed by the national and municipal authorities as a protected zone due to their specific natural features and in particular valuable water resources, flora, fauna and other amenities. However the settlements and tourist facilities around the lake do not have technically functional and acceptable sanitary sewerage system and corresponding treatment facilities. At the moment there is only partially developed sanitary sewerage system in the area of Vlasina Rid consisting of sanitary sewers and dysfunctional water treatment plant, downstream of the Vlasina dam.

However, the aforesaid facilities including both collection system and the treatment plant are out of order, which implicates that at the moment there are no operational sanitary sewerage facilities in the zone around the lake, which is declared as the protected zone. In the absence of proper sanitary sewerage, sewage from households, weekend houses and tourist facilities is discharged into septic tanks. This practice inevitably leads to consequent pollution of surrounding areas, ground and surface waters flowing towards the lake. Current situation is certainly not compliant with the declared protected status of the Project Area and it does not allow even for current tourist activities, let alone ambitious tourist development plans in the area.

Among the several areas in the Vlasina lake sanitary protection zone (as designated by the Spatial Plan of Special Purpose Area), three areas closest to the lake are: Vlasina Rid, Vlasina Okruglica and Vlasina Stojkoviceva. In accordance with the proposed tourism development scenario (see chapter on tourism development) small rural centres Klisura and Bozica are planned to be included in the overall tourism development scheme, and are therefore also included in the project scope.

So far, only part of the system has been constructed in the Vlasina Rid area, in the north-west lake region, including the trunk sewer and treatment plant. The wastewater treatment plant is just off the Vlasina River, about 700m downstream of the dam. The treatment plant is based on the RBC technology but has never been put into service.

Sanitary sewage from most of residential houses, tourist resorts and facilities is being disposed to individual septic tanks with inadequate technical and process characteristics. Consequently, current situation bears great environmental as well as human health risks.

5.2.1.2 Review of the existing documentation

For the purpose of this Study, the following technical and planning documentation has been compiled and analysed:

- Spatial plan Surdulica (reference documentation 11)
- Spatial plan of Special Purpose Area (reference documentation 12)
- Preliminary Design of the Sanitary Sewerage System in the Vlasina Region with Feasibility Study (reference documentation 16)

On one hand, the information obtained from the mentioned material was valuable, but on the other hand the general a number of technical deficiencies and inconsistencies have been identified. Therefore the task of the Study was to balance and optimize the proposed solutions.

It also appeared that the Conceptual and Preliminary Design have not been technically appraised by the relevant authority, which needs to be done before the Project implementation. This would result in the overall improvement of the design and lead to a successful realisation of the tendering process and construction works.

5.2.1.3 Technical concept

The concept proposed by the Study generally relies on the above mentioned technical designs, questionnaires filled by the local authorities and relevant services, site visits and other information obtained from the municipality of Surdulica.

It is of utmost importance to provide a feasible and sustainable solution for the wastewater collection, treatment and disposal system for the settlements in the protected zone. Consequently, the technical concept addresses the service area including Vlasina Rid, Vlasina Okruglica, Vlasina Stojkoviceva, small scattered villages in the zone called mahalas, as well as Klisura and Bozica settlements.

The capacity of the sewerage network and treatment plants has been designed for the target year 2035, accounting for the maximum possible tourist occupancy. Staged development of the system has been considered, in line with envisaged development of the settlements and resorts.

Due to the characteristics of the Project Area, with significant number of existing tourist facilities and settlements scattered throughout in the Project Area, it would be technically viable and environmentally necessary to construct the whole system in the first instance/stage thus augmenting the overall sewerage system efficiency and environment protection. However, the study envisages a two-phased system development based, on realistic tourism development scenario, social and economic development forecast and realistic information collected on site. The first stage encompasses wastewater system development in the Vlasina Rid and Vlasina

Okruglica area. The second phase would include wastewater collection system extension in the Vlasina Stojkoviceva area.

The concept promotes a gravity-based sanitary sewerage system around the lake perimeter with reduced number of pumping stations and pressure pipes to minimise maintenance and protect the environment. Taking into account topography of the area, two alternatives have been considered. The first alternative would have the sewerage system discharging to two wastewater treatment plants situated in Vlasina Rid and Vlasina Okruglica as shown on the layout map. The second alternative considers a sewerage system with only one wastewater treatment plant located in Vlasina Rid and a long collection network for sewage collection and transport from Vlasina Stojkoviceva, Vlasina Okruglica and Vlasina Rid to the proposed location of the treatment plant.

In case of Alternative 1, the treatment plant in Vlasina Rid, WWTP Vlasina, would be sized for 2x3000 PE, and is supposed to serve users in the Vlasina Rid and partly Vlasina Stojkoviceva settlements. The WWTP Vrla, in Vlasina Okruglica would have the capacity of 2x1500 PE and serves users in Vlasina Rid, Vlasina Okruglica and most of the Vlasina Stojkoviceva settlement.

The second alternative includes a single WWTP in Vlasina Rid, with capacity of 9.000 PE. Two settlements detached from the lake, Klisura and Bozica, would have their separate collection networks and treatment plants each having capacity of 500 PE. This is considered to be technically and financially better option compared with the alternative of these two systems integrated in the main/trunk wastewater collection system around the lake.

5.2.1.4 Demand Projection

5.2.1.4.1 Population and tourist projection

The projection of the Vlasina zone tourism development has been set out in the Tourism Strategy Report and serves as the bases for calculation and prediction of design wastewater flows. An estimate of number of permanent and occasional residents in the project area is based on a comprehensive report prepared by Surdulica municipality (Annex 3.4).

Main users of the sanitary sewerage system have been identified as:

Tourist users:

- Hotels + Annexes
- Private B&B's
- Resorts
- Camps

- Rural Accommodation + lodges
- Relaxation facilities

Residential (domestic) users in villages:

- Permanent
- Temporary

The abovementioned Tourist Strategy envisages phased tourist development and user number increase through several milestone years – phases:

1. Phase I development - by year 2015
2. Phase II development - by year 2025
3. Phase III development - by year 2035

The forecast of design wastewater flows is related to domestic, tourist and recreational user number and tourist occupancy and represents main input data for planning, sizing and design of the wastewater collection, treatment, and disposal facilities. This projection is takes into account the original Tourist Development Master Plan for Vlasina Lake (reference documentation 13) and also the realistic tourism development scenario outlined in this study.

The following table provides an overview of the phased user development by category, location and year:

Table 5.17: Tourist and Population Projection

Vlasina Resorts & Villages	User count			
	2006	2015	2025	2035
Tourist Resorts				
Vlasina Rid	1,715	2,025	2,425	4,815
Vlasina Okruglica	180	480	880	880
Vlasina Stojkoviceva	160	310	310	460
Tourist Resorts – total	**2,055**	**2,815**	**3,615**	**6,155**
Villages				
Vlasina Rid	1,493	1,539	1,589	1,640
Vlasina Okruglica	478	531	589	648
Vlasina Stojkoviceva	632	681	736	790
Total-Villages	**2,603**	**2,750**	**2,914**	**3,078**
Total Tourist Resorts and Villages	**4,658**	**5,565**	**6,529**	**9,233**
Božica-population	350	350	350	350
Božica-visitors	0	100	100	100
Božica-total	**350**	**450**	**450**	**450**
Klisura-population	350	350	350	350
Klisura-visitors	0	100	100	100
Klisura-total	**350**	**450**	**450**	**450**

Further breakdown of the above figures has been carried for the sake of spatial distribution of sanitary sewerage users and their allocation to the appropriate sewer catchment.

5.2.1.4.2 Unit wastewater rates and peak factors

Unit sanitary sewage flowrates have been based on the water consumption analysis and estimates.

As quoted in the potable water demand analysis, average unit water consumption rates are set as follows:

⇒ Resident Water Demand - 150 litres/capita/day

⇒ Tourist Water Demand - 300 litres/capita/day

Resulting sewage discharges are estimated at 85% of potable water usage, and unit wastewater rates are defined as follows:

⇒ Average resident unit wastewater rate - 128 litres/capita/day

⇒ Average tourist unit wastewater rate - 255 litres/capita/day

Peak Day Factors

Peak day factors/flows have been adopted in accordance with current operational records and future system characteristics:

⇒ For population - Peak day factor $\mathbf{K_{max\ day} = 1{,}50}$

⇒ For tourists - Peak day factor $\mathbf{K_{max\ day} = 1{,}50}$

Peak Hour Coefficient

The peak hour coefficient has been calculated in relation to a baseline flow rate i.e. maximum daily flow in the sanitary sewerage system, in accordance with the Fedorov equation:

$$K_{peak\ hour} = 2.69\ x Q^{(-0.12)}{}_{max.day}$$

where Q refers to the maximum daily flow rate.

5.2.1.4.3 Infiltration/Inflow

Infiltration may be defined as the water entering a sewerage system, including sewer service connections, from the surrounding ground, through such means as, but not limited to, defective pipes, pipe joints, connections, manhole walls, and other. The rate and quantity of infiltration depend on the length of sewers, the area served, the soil and topographic conditions, and to a certain extent, the population density (which affects the number and total length of house connections).

On the other hand inflow may be described as the water discharging into a sewerage system, including service connections, from such sources as, but not limited to, individual property (yard, cellar) drains, roof leaders, foundation drains, cooling water discharges, swimming pool discharges, cross connections from storm drains and combined sewers, storm waters, surface runoff, street wash waters, etc. Proper design and consequent construction of the sanitary sewerage system are supposed to minimize the possibility of intrusion of water, other than sewage, into the system. Contemporary sewer design calls for the use of high-quality pipes with dense walls (PVC, PE, GRP, etc.), with joints sealed with impervious rubber or synthetic gaskets, or achieved by thermal fusion. Completely or partially pre-cast watertight manholes are also normally used, preventing infiltration into the sewerage system.

However, although infiltration and inflow into the sewerage system can be minimized, in most cases it can not be eliminated altogether and therefore certain allowance has to be made by means of additional hydraulic, pumping and treatment capacities of the system.

This study prescribes the use of:

⇒ Sewer pipe materials and joints of the highest standard for the installation of the main gravity sewers, collection network and service connections;

⇒ Completely or partially pre-fabricated, or high quality cast in-situ manhole structures or prefabricated polypropylene (PP) or polyethylene (PE) manholes.

⇒ Adequate, heavy-duty manhole covers and frames;

Therefore expected infiltration flow resulting from the defects on the collection network is estimated to be rather low.
For the purpose of this study infiltration rates within the project area have been estimated as follows:

⇒ $\mathbf{Q_{infiltration} = 0.2\ l/s/km}$

5.2.1.4.4 Total sewage flows

Total sewage flows for the project area have been estimated based on the population/tourist projection, estimated sewage unit (per capita) rates, estimated infiltration and corresponding peak factors. Tourist occupancy has been adopted as 100% in the peak tourist season, as this yields the highest wastewater flow rates in the sewerage system and allows for calculation of the maximum system capacity. However, for calculating average annual wastewater discharges, realistic tourist accommodation occupancy rates have been taken into account, as described in tourism development study chapter.

In accordance with the Tourist Strategy Report, estimated sewage flows for each of the settlements in the service area and user category (tourist or population) are presented in the enclosed tables indicating:

- Number of users
- Average wastewater units rates (litres/capita/day)
- Average sewage flows (l/s)
- Infiltration flow (l/s/km)
- Seasonal/day peak factor – $\mathbf{K}_{\text{max day}}$
- Maximum daily flow (l/s)
- Peak hour coefficient - $\mathbf{K}_{\text{max hour}}$
- Peak hour flow (l/s)

Said flowrates and peak factors have been organized to represent proposed phased development of the project area, and also to reflect spatial distribution of the sewage flows over the project area. Presented sewage flows have been further used as the basic input data for the design of the wastewater collection, treatment and disposal facilities.

Table 5.18: Total Wastewater Flows – WWTP Vlasina – Alternative 1

Vlasina Resorts&Villages	User count		Sanitary Unit load (l/cap/day)		Q average (l/s)		L sewers (km)		Q infilt. (l/s)		Q av.gross (l/s)		Kmax day		Q max day (l/s)		Kh		Qpeak hour (l/s)	
	2006	2035	2006	2035	2006	2035	2006	2035	2006	2035	2006	2035	2006	2035	2006	2035	2006	2035	2006	2035
Catchment area for WWTP Vlasina - Design Wastewater flows																				
Tourist Resorts																				
Vlasina Rid	1,715	4,765	255	255	5.1	14.1							1.5	1.5	7.6	21.1	2.1	1.9	16.0	39.4
Vlasina Okruglica	0	0	255	255	0.0	0.0							1.5	1.5	0.0	0.0	0.0	0.0	0.0	0.0
Vlasina Stojkoviceva	0	0	255	255	0.0	0.0							1.5	1.5	0.0	0.0	0.0	0.0	0.0	0.0
Tourist Resorts - total	**1,715**	**4,765**			**5.1**	**14.1**									**7.6**	**21.1**	2.1	1.9	**16.0**	**39.4**
Villages																				
Vlasina Rid	1,156	1,268	128	128	1.7	1.9							1.5	1.5	2.6	2.8	2.4	2.4	6.1	6.7
Vlasina Okruglica	0	0	128	128	0.0	0.0							1.5	1.5	0.0	0.0	0.0	0.0	0.0	0.0
Vlasina Stojkoviceva	48	56	128	128	0.1	0.1							1.5	1.5	0.1	0.1	3.5	3.5	0.4	0.4
Total-Villages	**1,204**	**1,324**			**1.8**	**2.0**									**2.7**	**2.9**	**2.4**	**2.4**	**6.4**	**6.9**
Total Tourist Resorts and Villages	**2,919**	**6,089**			**6.8**	**16.0**	**0.0**	**21.322**	**0.0**	**4.3**	**6.8**	**20.3**			**10.3**	**28.3**	**2.0**	**1.8**	**20.9**	**55.2**

**** wastewater to water ratio = 0,85***

Table 5.19: Total Wastewater Flows – WWTP Vrla – Alternative 1

Vlasina Resorts&Villages	User count		Sanitary Unit load (l/cap/day)		Q average (l/s)		L sewers (km)		Q infilt. (l/s)		Q av.gross (l/s)		Kmax day		Q max day (l/s)		Kh		Qpeak hour (l/s)	
	2006	2035	2006	2035	2006	2035	2006	2035	2006	2035	2006	2035	2006	2035	2006	2035	2006	2035	2006	2035
Catchment area for WWTP Vrla - Design Wastewater flows																				
Tourist Resorts																				
Vlasina Rid	0	50	255	255	0.0	0.1							1.5	1.5	0.0	0.2	0.0	3.2	0.0	0.7
Vlasina Okruglica	180	880	255	255	0.5	2.6							1.5	1.5	0.8	3.9	2.8	2.3	2.2	8.9
Vlasina Stojkoviceva	160	460	255	255	0.5	1.4							1.5	1.5	0.7	2.0	2.8	2.5	2.0	5.0
Tourist Resorts - total	**340**	**1390**			**1.0**	**4.1**									**1.5**	**6.2**	2.6	2.2	**3.9**	**13.3**
Villages																				
Vlasina Rid	337	372	128	128	0.5	0.5							1.5	1.5	0.7	0.8	2.8	2.8	2.1	2.3
Vlasina Okruglica	478	648	128	128	0.7	1.0							1.5	1.5	1.1	1.4	2.7	2.6	2.8	3.7
Vlasina Stojkoviceva	584	734	128	128	0.9	1.1							1.5	1.5	1.3	1.6	2.6	2.5	3.4	4.1
Total-Villages	**1399**	**1754**			**2.1**	**2.6**									**3.1**	**3.9**	**2.3**	**2.3**	**7.3**	**8.9**
Total Tourist Resorts and Villages	**1739**	**3144**			**3.1**	**6.7**	**0.0**	**25.935**	**0.0**	**5.2**	**3.1**	**11.9**			**4.6**	**15.2**	**2.2**	**1.9**	**10.3**	**34.7**

**** wastewater to water ratio = 0,85***

Table 5.20: Total Wastewater Flows – WWTP Vlasina – Alternative 2

Vlasina Resorts & Villages	User count				Unit loading rate (l/cap/day)	$Q_{average}$ (l/s)				L_{sewers} (km)				$Q_{infilt.}$ (l/s)				$Q_{av.gross}$ (l/s)				$K_{max\ day}$		$Q_{max\ day}$ (l/s)				K_h				$Q_{peak\ hour}$ (l/s)			
	2006	2015	2025	2035		2006	2015	2025	2035	2006	2015	2025	2035	2006	2015	2025	2035	2006	2015	2025	2035	2006	2021	2006	2015	2025	2035	2006	2015	2025	2035	2006	2015	2025	2035
Tourist Resorts																																			
Vlasina Rid	1,715	2,025	2,425	4,815	255	5.1	6.0	7.2	14.2													1.5		7.6	9.0	10.7	21.3	2.1	2.1	2.0	1.9	16.0	18.5	21.7	39.7
Vlasina Okruglica	180	480	880	880	255	0.5	1.4	2.6	2.6													1.5		0.8	2.1	3.9	3.9	2.8	2.5	2.3	2.3	2.2	5.2	8.9	8.9
Vlasina Stojkoviceva	160	310	310	460	255	0.5	0.9	0.9	1.4													1.5		0.7	1.4	1.4	2.0	2.8	2.6	2.6	2.5	2.0	3.6	3.6	5.0
Tourist Resorts - total	**2,055**	**2,815**	**3,615**	**6,155**		**6.1**	**8.3**	**10.7**	**18.2**															**9.1**	**12.5**	**16.0**	**27.2**	**2.1**	**2.0**	**1.9**	**1.8**	**18.8**	**24.8**	**30.9**	**49.3**
Villages																																			
Vlasina Rid	1,493	1,539	1,589	1,640	128	2.2	2.3	2.3	2.4													1.5		3.3	3.4	3.5	3.6	2.3	2.3	2.3	2.3	7.7	7.9	8.1	8.4
Vlasina Okruglica	478	531	589	648	128	0.7	0.8	0.9	1.0													1.5		1.1	1.2	1.3	1.4	2.7	2.6	2.6	2.6	2.8	3.1	3.4	3.7
Vlasina Stojkoviceva	632	681	736	790	128	0.9	1.0	1.1	1.2													1.5		1.4	1.5	1.6	1.7	2.6	2.6	2.5	2.5	3.6	3.9	4.1	4.4
Total-Villages	**2,603**	**2,751**	**2,914**	**3,078**		**3.8**	**4.1**	**4.3**	**4.5**															**5.8**	**6.1**	**6.5**	**6.8**	**2.2**	**2.2**	**2.2**	**2.1**	**12.6**	**13.2**	**13.9**	**14.6**
Total Tourist Resorts and Villages	**4,658**	**5,566**	**6,529**	**9,233**		**9.9**	**12.4**	**15.0**	**22.7**	**0.0**	**23.6**	**35.4**	**47.3**	**0.0**	**4.7**	**7.1**	**9.5**	**9.9**	**17.1**	**22.0**	**32.2**			**14.9**	**23.3**	**29.5**	**43.5**	**1.9**	**1.8**	**1.8**	**1.7**	**28.9**	**38.9**	**47.3**	**67.7**
Božica-population	350	350	350	350	128	0.5	0.5	0.5	0.5													1.8		0.9	0.9	0.9	0.9	2.7	2.7	2.7	2.7	2.5	2.5	2.5	2.5
Božica-visitors	0	100	100	100	255	0.0	0.3	0.3	0.3													1.8		0.0	0.5	0.5	0.5	0.0	2.9	2.9	2.9	0.0	1.5	1.5	1.5
Božica-total	**350**	**450**	**450**	**450**		**0.5**	**0.8**	**0.8**	**0.8**	**0.0**	**3.0**	**3.0**	**3.0**	**0.0**	**0.6**	**0.6**	**0.6**	**0.5**	**1.4**	**1.4**	**1.4**			**0.9**	**2.1**	**2.1**	**2.1**	**2.7**	**2.5**	**2.5**	**2.5**	**2.5**	**4.2**	**4.2**	**4.2**
Klisura-population	350	350	350	350	128	0.5	0.5	0.5	0.5													1.8		0.9	0.9	0.9	0.9	2.7	2.7	2.7	2.7	2.5	2.5	2.5	2.5
Klisura-visitors	0	100	100	100	255	0.0	0.3	0.3	0.3													1.8		0.0	0.5	0.5	0.5	0.0	2.9	2.9	2.9	0.0	1.5	1.5	1.5
Klisura-total	**350**	**450**	**450**	**450**		**0.5**	**0.8**	**0.8**	**0.8**	**0.0**	**3.0**	**3.0**	**3.0**	**0.0**	**0.6**	**0.6**	**0.6**	**0.5**	**1.4**	**1.4**	**1.4**			**0.9**	**2.1**	**2.1**	**2.1**	**2.7**	**2.5**	**2.5**	**2.5**	**2.5**	**4.2**	**4.2**	**4.2**

5.2.1.5 Hydraulic Analysis

5.2.1.5.1 Design criteria

LIMIT VELOCITIES

The limit design velocities in the main gravity sewers and forcemains have been defined based on the total design wastewater flowrates as specified earlier.

The limit design velocities **for the main gravity sewers** have been set as follows:

1. Minimum velocity of 0,75 m/s for pipes flowing just full;

2. Maximum velocity of 4,50 m/s;

For the forcemains the limit design velocities have been set as follows:

3. Minimum velocity of 0,6 m/s in order to promote scouring action.

4. Maximum velocity is set to 1,60 – 1,80 m/s at the design flow in order to maintain head losses and associated energy costs within the reasonable limits.

HYDRAULIC ROUGHNESS

Equivalent hydraulic roughness used in hydraulic calculations has been adopted in accordance with the pipe material to be installed and expected dominant hydraulic conditions in the forcemains and main gravity sewers under considerations.

High-density polyethylene pipes have been proposed for the forcemains. These pipes are considered to be suitable for aggressive sewage water and are also very suitable for handling, installation and maintenance. Given the range of nominal diameters to be used for the forcemains, these pipes can be delivered in coils and be joined either by butt welding or electro-fusion. In hydraulic calculations of the friction head losses along the forcemains equivalent hydraulic roughness is set to be K=0.3 mm, corresponding to expected normal hydraulic and operational conditions.

uPVC pipes are to be used for the construction of the main gravity sewers and the collection network. Equivalent hydraulic roughness used for hydraulic calculation of friction head losses is set to be K=1.5 mm (or n=0,013 $sm^{-1/3}$ after *Manning*) in accordance with the pipe materials adopted and expected normal hydraulic and operational conditions.

SEWER DEPTHS

The minimum depth of cover for the main gravity sewers from pipe crown to finished surface is set to be 1.6m in order to allow house connections (where applicable).

The minimum depth of cover for the forcemains from pipe crown to finished surface is set to be 1,2 m, in order to minimise the effect of traffic load and frost.

The maximum sewer depths have been determined based on the following principles:

5. Deep trench excavation has a significant impact on the construction costs, especially if the sewers are to be laid in geologically hard soils or below the groundwater table.
6. Generally, at this stage, the sewer trench depth has been limited to 4-5 meters, except locally, for the sewer route short sections in order to avoid construction of the sewage pumping stations;

LIQUID DEPTH AT PEAK FLOW (D/D)

The maximum ratio of the liquid depth (d) to the pipe diameter (D) - d/D at the peak design flow is set to be 0,70 in order to provide continuous ventilation of the main gravity sewers and a safe margin for pipe flow capacity in case of underestimation of flow rate (e.g. additional connections to the system) or for unexpected inflow into the sewer (for instance abrupt intrusion of surface runoff during the storms).

MINIMUM PIPE DIAMETERS

According to engineering publications and national best practice the pipe hydraulic diameters are to be 250 mm and larger for the main gravity sewers.

SEWER SLOPES

The following minimum and maximum sewer slopes have been proposed based on *Chezy-Manning* equation and limit velocities mentioned earlier.

Table 5.21: Minimum and maximum slopes of gravity sewers

n =	0.013	$m^{-1/3}s$					
DN (mm)	100	150	200	250	300	400	500
V_{min} (m/s)	0.75						
V_{max} (m/s)	3.00						
Q_{min} (l/s)	6	13	24	37	53	94	147
Q_{max} (l/s)	24	53	94	147	212	377	589
I_{min} (%)	1.30	0.76	0.52	0.38	0.30	0.20	0.15
I_{max} (%)	20.81	12.12	8.26	6.13	4.81	3.28	2.43

BASIC EQUATIONS

The hydraulic calculation of open channel flow in gravity sewers, resulting in sizing and capacity determination has been carried out based on “capacity flow” approach and *Chezy–Manning* equation for head loss calculation. This approach stipulates uniform flow in sewers and calculation of sewer capacity based on normal depth of water in pipes.

The well known *Chezy–Manning* equation gives the following relation:

$$V = \frac{1}{n} \cdot R^{\frac{2}{3}} \sqrt{S}$$

And the corresponding flow rate equals:

$$Q = \frac{1}{n} \cdot A \cdot R^{\frac{2}{3}} \sqrt{S}$$

Where

- V - wastewater flow velocity (m/s)
- Q - wastewater flow rate (m^3/s)
- N - Manning's roughness coefficient ($sm^{-1/3}$)
- A - wetted cross section area of flow (m^2)
- R - Hydraulic radius (m)
- S - Hydraulic grade due to friction (m/m)

Friction head loss in pressure pipes has been derived from the *Darcy-Weisbach* equation:

$$\Delta h = f \cdot \frac{L}{D} \cdot \frac{v^2}{2g}$$

Where

- Δh - friction head loss (m)
- F - friction factor
- L - pipeline length (m)
- D - pipe diameter (m)
- V - flow velocity (m/s)
- g - gravitational acceleration – 9.81 (m/s^2)

The friction factor f is calculated by the explicit *Swamee and Jain* equation:

$$f = \frac{1.325}{\left[\log_e\left(\frac{k}{D} + \frac{5.74}{Re^{0.9}}\right)\right]^2}$$

Where:

- f - friction factor
- k - equivalent hydraulic roughness (m)
- D - pipe diameter (m)
- Re - Reynolds number

5.2.1.5.2 Results of hydraulic calculation

As described in the previous section, friction head loss and flow conditions in gravity sewer pipes are calculated by using of the *Chazy - Manning equation*. This method assumes uniform free-surface flow in sewers.

The results are shown in the table below and refer to the main sewers shown on the general layout map. Corresponding design flow rates have been taken for the 2035 horizon and were determined for each subcatchment based on the Wastewater Flow Tables and spatial allocation of users.

Table 5.22: Alternative 1 - Results of Hydraulic Analysis for Gravity Sewers

No	Section/ Sewer	DN	Equivalent hydraulic diameter	Length	Average Slope	Q_{design}	Q_{full}	V_{full}	V_{design}	H_{design}	H_{design}/h_{full}
		(mm)	(mm)	(m)	(%)	(l/s)	(l/s)	(m/s)	(m/s)	(m)	(%)
1	VL-1	315	300	7 419	0.30	41.1	53.0	0.75	0.83	0.20	66.2
2	VL-2	250	238	13 902	0.38	17.7	37.0	0.75	0.74	0.12	49.0
3	VR-1	250	238	9 025	0.38	8.8	37.0	0.75	0.61	0.08	33.4
4	VR-2	250	238	8 010	0.38	30.0	37.0	0.75	0.83	0.17	68.8
5	VR-3	250	238	8 900	0.38	14.9	37.0	0.75	0.71	0.11	44.4
6	Bozica-main	250	238	670	0.38	4.2	37.0	0.75	0.50	0.06	22.9
7	Klisura-main	250	238	1 015	0.38	4.2	37.0	0.75	0.50	0.06	22.9

Table 5.23: Alternative 2 – Results of Hydraulic Analysis for Gravity Sewers

No	Section/ Sewer	DN	Equivalent hydraulic diameter	Length	Average Slope	Q_{design}	Q_{full}	V_{full}	V_{design}	H_{design}	H_{design}/h_{full}
		(mm)	(mm)	(m)	(%)	(l/s)	(l/s)	(m/s)	(m/s)	(m)	(%)
1	VL-1	400	380	7 419	0.20	75.9	93.0	0.75	0.83	0.27	68.6
2	VL-2	250	238	13 902	0.38	17.7	37.0	0.75	0.74	0.12	49.0
3	VR-1&2	315	300	6 620	0.30	34.8	53.0	0.75	0.80	0.18	59.1
4	VR-3	250	238	8 900	0.38	14.9	37.0	0.75	0.71	0.11	44.4
5	Bozica-ain	250	238	670	0.38	4.2	37.0	0.75	0.50	0.06	22.9
6	Klisura-ain	250	238	1 015	0.38	4.2	37.0	0.75	0.50	0.06	22.9

Notes:

- *The design flow rates shown in the above tables refer only to the specific most downstream section of the main sewer. The calculated flow parameters in the table show that the sewer pipes are of sufficient capacity for the estimated hydraulic loadings.*
- *Indicated average pipe gradients are in accordance with the Preliminary Design of Sanitary Sewerage (reference documentation 16). However, in the following stages of design preparation (Detail Project Design) it is necessary to verify terrain topography and detail sewerage network elevations by means of corresponding topographic survey.*

5.2.1.6 Proposed sanitary sewerage system – Alternative 1

5.2.1.6.1 Gravity sewers

Based on the above considerations, the sanitary sewerage system serving users in the Vlasina lake area will have the following main components/facilities as shown on the general layout map:

- Main gravity sewers VL-1, VL-2, VR-1, VR-2, VR-3
- Wastewater treatment plants Vlasina and Vrla
- Sewage pumping station in the southern region of the Vlasina lake, transferring flows from Vlasina Stojkoviceva to Vlasina Okruglica and to WWTP Vrla
- Forcemain DN140 from the sewage pumping station, with length app. 1600m
- Individual gravity sewerage networks in Bozica and Klisura settlements
- Small package wastewater treatment plants in Bozica and Klisura.

The size of the gravity sewers has been checked and confirmed in the hydraulic design table.

The wastewater treatment plants Vlasina and Vrla have been designed for 2x3000 and 2x1500 PE respectively, whereas treatment plants in Klisura and Bozica will each have nominal capacity of 500 PE.

The main gravity sewers VL-1 and VL2 will serve the northern part of the Project Area, from the western (VL-1) and eastern (VL-2) side towards the location of the WWTP Vlasina in the north.

On the other hand, the southern collection system will comprise main sewers VR-1, VR-2 and VR-3 from the western and eastern side of the lake towards the southern WWTP Vrla.

The sewage pumping station and forcemain will be part of the southern sewerage sub-system and will transfer sewage from VL-3 to VL-2 sewer.

It is highly recommended to apply watertight prefabricated manholes at normal distances of around 160xDN in the straight runs, and also at all changes of sewer slopes and directions. The manholes can be made of polypropylene (PP), HDPE or GRP material, with special sealing system between sections to provide a watertight structure.

The proposed system would improve to a great extent environmental protection of the designated protected zone, and in particular protection of the Vlasina Lake water resources.

Plastic uPVC pipes may be used for gravity sewerage system, offering a cost effective solution, easier installation and handling. However as the sewerage system runs in the sanitary protection zone, special care during installation is needed, and compliance with the manufacturers laying instructions. The pipes should be installed in a trench on a 10 cm compacted bedding in granular material (sand). The pipe zone material up to 30 cm above pipe crown will also be selected granular material (sand). The rest of the backfill will be with excavated material free from stones, organic material, debris, etc.

5.2.1.6.2 Sewage pumping station and forcemains

Due to the topographical characteristics of the project area a pumping station shall be constructed in order to transport sewage from the lowest point of the gravity sewer VR-3 (serving mainly Vlasina Stojkoviceva settlement) towards the main gravity sewer VL-2 that further transfers collected sewage through Vlasina Okruglica to the planned treatment facilities.

The design flow for the concerned pumping station corresponds to the peak hour sewage flow for the corresponding contributing area, i.e. catchment area of the main sewer VR-3.

The sewage is transported via a HDPE DN140, PN10 forcemain to a manhole at the upstream end of VL-2. The approximate length of the forcemain is around 1.600m. It runs along a local road and the same installation specifications apply, as for the gravity sewers.

The pumping stations structures, mechanical and electrical equipment will be sized to provide pumping capacity corresponding to the mentioned peak design flow.

It has been recommended that the sewage pumping station be of the wet-well submersible type. This type of sewage pumping stations is suitable for relatively low discharge rates, can be designed for completely underground installation that is a great aesthetical advantage for tourist areas.

Pumping stations will be designed to minimize potential risks of operation disruption. It will be equipped with one duty plus one stand-by pump of the design capacity each. In addition, an introduction of dual power supply may be considered as well as the provision of diesel-powered power generators that would start automatically upon power failure. In order to reduce nuisance to the local residents and tourists the pumping station may be equipped with odour control equipment.

It is a matter of utmost importance that the pumping stations structure provides full water-tightness eliminating leakage of sewage that may contaminate surrounding soil and underground water and represent substantial environmental hazard.

Therefore, in order to cope with this issue, at this stage it has been recommended that pumping station structure be pre-fabricated high-density polyethylene or GRP wet-wells.

Major characteristics of the sewage pumping station and corresponding forcemain are shown in the following table:

Table 5.24: Sewage pumping station and forcemain characteristics

$Q_{nominal}$ (l/s)	$H_{nominal}$ (m)	No of pumps	Rated power (kW)	Forcemain length (m)	DN (mm)	PN (bar)	SDR
14.9	**50.0**	1+1	18.5	1.600	140/10.3	10	13.6

5.2.1.7 Proposed sanitary sewerage system – Alternative 2

5.2.1.7.1 Gravity sewers

Another alternative that has been considered in the Study differs from the Alternative 1 as follows:

- The Alternative 2 envisages a single, centralised wastewater treatment option in Vlasina Rid via a WWTP of 9.000 PE capacity.
- The main sewers would run around the lake perimeter from Vlasina Stojkoviceva to the WWTP, comprising sewers of greater size (as shown in the results of hydraulic analysis) compared to Alternative 1.
- The Alternative 2 would have more sewage pumping stations and corresponding forcemains (at least 2 main SPS) compared with Alternative 1, posing greater environmental impact risk, more intensive maintenance and higher operational costs.

The size of the gravity sewers has been checked and confirmed in the hydraulic calculation table.

The Alternative 2 concept is based on connecting the sewerage system serving the eastern and southern part of the lake to the VL1 main sewer which serves the western part of the lake and conveys wastewater to the central WWTP.

The southern collection system will now comprise sewers VR-1 and VR-2 as one integral sewer, whereas the eastern sewerage system includes VR-3 sewer and a SPS for wastewater transfer to sewers VL-1 and VL-2.

5.2.1.7.2 Sewage pumping station and forcemains

The same technical description and assumptions apply as for Alternative 1. The following table denotes the main pumping station in the system for Alternative 2.

Table 5.25: Sewage pumping station and forcemain characteristics

$Q_{nominal}$ (l/s)	$H_{nominal}$ (m)	No of pumps	Rated Power (kW)	Forcemain Length (m)	DN (mm)	PN (bar)	SDR
14.9	**50.0**	1+1	18.5	1.600	140/10.3	10	13.6
35.8	**30.0**	1+1	30.0	470	225/16.6	10	13.6

5.2.1.8 Wastewater collection system – investment costs

This section presents an overview of the estimated investment costs for construction of the described main sanitary sewerage system in the zone of Vlasina Lake. Detail bill of quantities is shown in Annex 5.8.

Table 5.25: Collection System Alternative 1 – Investment costs overview

Sub-catchment	VL1	VL2	VR1	VR2	VR3	Klisura	Bozica	Total
PRELIMINARY WORKS	39.676	67.608	46.100	24.680	54.000	8.009	8.216	**248.289**
EARTHWORKS	409.297	758.260	492.253	200.178	536.926	71.872	74.158	**2.542.944**
CONCRETE WORKS	645	1.212	696	696	783	126	126	**4.284**
PIPEWORK AND MISCELLANEOUS EQUIPMENT	461.731	675.572	437.800	177.710	459.000	69.910	70.630	**2.352.353**
MISCELLANEOUS WORKS	135.322	234.805	156.721	72.521	243.373	25.285	25.895	**893.923**
Total	**1.046.670**	**1.737.457**	**1.133.570**	**475.785**	**1.294.082**	**175.203**	**179.025**	**6.041.793**

Table 5.26: Collection System Alternative 2 – Investment costs overview

Sub-catchment	VL1	VL2	VR1&2	VR3	Klisura	Bozica	Total
PRELIMINARY WORKS	39.676	67.608	39.040	54.000	8.009	8.216	216.549
EARTHWORKS	500.387	758.260	473.299	536.926	71.872	74.158	2.414.904
CONCRETE WORKS	645	1.212	654	783	126	126	3.546
PIPEWORK AND MISCELLANEOUS EQUIPMENT	602.692	675.572	560.928	459.000	69.910	70.630	2.438.732
MISCELLANEOUS WORKS	146.925	234.805	284.448	243.373	25.285	25.895	960.731
Total	**1.290.324**	**1.737.457**	**1.358.369**	**1.294.082**	**175.203**	**179.025**	**6.034.462**

5.2.2 Wastewater treatment

5.2.2.1 Review of existing documentation

This section presents a general review of the available technical documentation related to the planned wastewater treatment facilities to serve the Project Area. The following related documentation was provided by the representatives of Surdulica municipality in the course of preparation of this study:

- Preliminary Design of System for Collection, Transport and Treatment of Wastewater and Stormwater Drainage, Vlasina; Faculty of Architecture and Civil Engineering, Nis, 2006;
- Extract from Detail Design of sewerage system for collection, transfer and treatment of wastewater at Vlasina; Civil Engineering and Architecture Faculty, University of Nis, 2008;

The abovementioned documentation included the following concept of a wastewater collection and treatment system in the Project Area:

1. Sewerage system is of so called separate type with sewage being transferred to planned wastewater treatment facilities, without any interference with stormwater;

2. Project Area is divided in two catchments, northern and southern, the former being directed towards planned wastewater treatment plant some 700m north of Vlasina dam, towards Crna Trava, and the latter directed towards another wastewater treatment plant to be located south of the Vlasina Lake and to be discharged in the Vrla River;

3. Hydraulic and pollutant loadings for the considered sewerage system and the abovementioned wastewater treatment facilities is based on the Spatial Plan of the Special Purpose Area, i.e. on the number of users defined in the Spatial Plan;

4. WWTP serving the northern catchment is sized for a pollutant loading of 6,000 population equivalent, while WWTP serving the southern catchment is sized for a loading of 3,000 population equivalent;

5. Both WWTP-s, Vlasina and Vrla include the following units:

   - ⇨ Water line:
     - ⇨ Coarse screen
     - ⇨ Flowmeter
     - ⇨ Sand trap
     - ⇨ Primary settling tank
     - ⇨ Bio-aeration basin
     - ⇨ Sand filters
     - ⇨ Chlorination tank
   - ⇨ Sludge line:
     - ⇨ Sludge recirculation pumping station
     - ⇨ Sludge digesters
     - ⇨ Sludge drying beds

6. Technical proposal is presented in a very general manner and includes: brief comment of adopted design criteria, list of applied process units, process schemes, general layouts and rough cost estimate.

With regard to the abovementioned technical documentation, the following comments and observations are made:

1. In principle, proposed wastewater collection and treatment system respects normal environmental and sanitation requirements; full service coverage is planned for the Project Area, as well as treatment of collected wastewater before discharge into receiving water courses;

2. Hydraulic and pollutant loadings refer to the existing planning document (Spatial Plan of Special Purpose Area) which is in general acceptable practice;

3. Some of key parameters of influent quality have not been presented (suspended solids, COD, nitrogen, phosphorus) and it is therefore not possible to assess effectiveness of the proposed treatment;

4. Assumed unit organic loading (per capita) deviates from the standard unit organic loading, producing incorrect resulting organic loading presumably used in the design;

5. The following elements, normally included in a Preliminary Design of wastewater facilities are missing:

    - ⇨ Clearly defined influent quality
    - ⇨ Technical description of proposed process units with corresponding sizing, other appurtenant calculations, effectiveness of proposed treatment and effluent quality;
    - ⇨ Hydraulic design including corresponding calculations and presentation of structures;
    - ⇨ Structural design with corresponding calculations and sizing of the structures;
    - ⇨ Mechanical design, with corresponding calculations, sizing of the equipment, technical descriptions;
    - ⇨ Electrical design with appropriate descriptions, calculations, drawings and schemes;
    - ⇨ Process and instrumentation;
    - ⇨ Layout and sections of proposed process units and other structures;
    - ⇨ Adequate bill of quantities including breakdown of costs per type of works and equipment;

6. To summarize, the abovementioned technical documentation lacks a number of key elements of normally included in a design of wastewater treatment facilities, and needs to be substantially supplemented and detailed before it could be considered for further implementation.

5.2.2.2 Wastewater characteristics

Wastewater flows

Details of projected wastewater flows till the end of Project period have been shown earlier in the report. Here is an overview of wastewater flowrates relevant for sizing and design of required wastewater treatment facilities.

Two basic alternatives of the wastewater collection and treatment were considered; alternative 1 with two separate treatment plants serving appurtenant catchment areas and alternative 2 where all collected wastewater from the Project Area is directed to a single wastewater treatment plant. The following tables provide hydraulic loadings of the considered wastewater treatment facilities for both alternatives.

Table 5.27: Overview of wastewater flows to WWTP Vlasina (alternative 1, capacity 6.000 PE, maximal occupancy in tourist areas)

Vlasina Resorts & Villages	Q av.erage - with infiltration (l/s)		Q max day (l/s)		Q peak hour (l/s)	
	2006	2035	2006	2035	2006	2035
Gross Total	6.8	20.3	10.3	28.3	20.9	47.5

Table 5.28: Overview of wastewater flows to WWTP Vrla (alternative 1, capacity 3.000 PE, maximal occupancy in tourist areas)

Vlasina Resorts & Villages	Q av.erage - with infiltration (l/s)		Q max day (l/s)		Q peak hour (l/s)	
	2006	2035	2006	2035	2006	2035
Gross Total	3.1	11.9	4.6	15.2	10.3	24.7

Table 5.29: Overview of wastewater flows to WWTP Vlasina (alternative 2, capacity 9.000 PE, maximal occupancy in tourist areas)

Vlasina Resorts & Villages	Q av.erage – with infiltration (l/s)				Q max day (l/s)				Q peak hour (l/s)			
	2006	2015	2025	2035	2006	2015	2025	2035	2006	2015	2025	2035
Gross Total	9.9	17.1	22.1	32.2	14.9	23.3	29.5	43.5	28.9	38.9	47.3	67.7

Pollutant loading

Based on the demographic and tourism development projection number of system users has been defined earlier in the report. The total number of users for the end of project period (2035) has been projected as follows:

- Catchment WWTP Vlasina - 6,089
- Catchment WWTP Vrla - 3,144
- Whole Project Area - 9,233

The wastewater characteristics are predicted in accordance with the following typical per capita loadings:

Table 5.30: Loadings per capita

BOD	60g/cap.d
TSS	70g/cap.d
COD	120g/cap.d
Total Nitrogen	12g/cap.d
Total Phosphorus	2g/cap.d

Resulting typical quality of wastewater has been calculated based on the abovementioned flowrates (average including infiltration for the end of design period), projected number of users and assumed unit/per capita pollutant loading rates.

Table 5.30: Typical Quality of Wastewater

Contaminants	Units	WWTP Vrla (Alternative 1- 3.000 PE)	WWTP Vlasina (Alternative 1 – 6.000 PE)	WWTP Vlasina (Alternative 2 – 9.000 PE)	*Average*
Suspended solids	mg/l	214	243	233	***230***
Biological oxygen demand (BOD_5)	mg/l	183	208	199	***197***
COD	mg/l	367	417	399	***394***
Total nitrogen	mg/l	34	38	40	***37***
Total phosphorus	mg/l	6	7	7	***7***

It should be noted however that the aforesaid assumed typical quality of wastewater applies only for a single combination of wastewater flows and unit, per capita, loading rates. Actual wastewater quality may vary in a wide range depending on seasonal and daily demand variations, ground water table, etc.

5.2.2.3 Effluent criteria

Effluent criteria represent key design parameters for design of wastewater treatment facilities. With regard to effluent criteria for the concerned facilities, two sets of criteria have been considered:

- Serbian national regulations and guidelines in the area of wastewater treatment, discharge and protection of waters;
- Corresponding relevant EU regulations;

The design for the WWTP refers to the relevant local (Law on Waters and respective strategic documents, acts of law, rules and regulations, especially act 56 and act 59) and EU (EU Council Directive 91/271/EEC as amended by Directive 98/15/EC) legislation.

EU regulations

The following EU emission standards for treated water quality have been considered:

Table 5.31: Emission standards for municipal wastewater treatments in accordance with the Council Directive 91/271/EEC

Parameter	Concentration	Lowest removal efficiency (%)[1]
Biochemical oxygen demand (BOD_5 at 20°C) Without nitrification	25 mg/l O_2	70-90 %
Chemical oxygen demand (COD)	125 mg/l O_2	75 %
Total suspended matter	35 mg/l [(3)] 35 mg/l (>10.000 PE) 60 mg/l (2000-10000 PE)	90 %[(3)] 90 % (>10000 PE) 70 % (2000 - 10000 PE)

(1) Decrease relative to loading inlet water.
(2) The use of other comparable parameters is allowed: Total Organic Carbon (TOC) or Total Oxygen Demand (TOD) if a relation can be established between BOD_5 and these alternative parameters.
(3) Optional parameter.

Discharge of wastewater effluents in sensitive areas implies tertiary treatment for the removal of nitrogen and phosphorus to required levels presented in the following table.

Table 5.32: Emission requirements for discharge in sensitive areas

Parameter	Concentration	Lowest removal efficiency (%)[1]
Total phosphorus	1 mg/l P (>100.000 PE) 2 mg/l P (10.000 – 100.000 PE)	80 %
Total nitrogen [(4)]	10 mg/l N (>100.000 PE)[(5)] 15 mg/l N (10.000 – 100.000 PE)	70-80 %

(4) Total nitrogen: sum of Kjeldahl-N (organic- N + NH_4 – N), and NO_2 –N
(5) Alternative, daily average value must not exceed >20 mg/l N. This requirement relates to water with temperature of 12°C or above, during operation of the bio-reactor for waste water treatment. An alternative to temperature is the use of utilization time that considers regional climate. This alternative can be applied if it can be proven that requirement 1 of Annex ID is fulfilled.

The following table summarizes emission standards applicable for different sizes of urban agglomerations and for various classifications of areas.

Table 5.33: Summary of effluent requirements according to EU-directive 91/271/EEC

Load	PE	<2,000	2,000 -10,000	10,000-100,000	>100,000
BOD_5	mg/l	*	25	25	25
COD	mg/l	*	125	125	125
SS	mg/l	*	35 (60)	35	35
Total Nitrogen **	mg/l	-	-	15	10
Total Phosphorous **	mg/l	-	-	2	1

Notes:

* - *So called appropriate treatment*

** - *Applicable for sensitive areas*

The wastewater treatment facilities considered in this study would have to deal with the following pollutant loading:

1. **WWTP Vlasina – alternative 1 – 6,000 PE**
2. **WWTP Vrla – alternative 1 – 3,000 PE**
3. **WWTP Vlasina – alternative 2 – 9,000 PE**

All of the considered facilities pertain to the range 2,000 to 10,000 PE, and therefore main applicable effluent criteria would be:

1. **BOD_5 - 25 mg/l**
2. **COD - 125 mg/l**
3. **SS - 35 mg/l**

The aforesaid pollutant loadings do not require tertiary treatment for the removal of nitrogen and phosphorus. However, taking into account current status of the relevant Serbian national regulations (new Water Law under preparation but still not passed), and potential future use of the receiving waters (rivers Vrla and Vlasina), technical proposal in this study allows for the treatment process upgrade, including nutrients removal, at later stage.

Serbian national regulations

In general relevant national immision standards are defined in the following regulations:

- The Ordinance on Water Courses Classification, OGRS 5/68;
- The Ordinance on Classification of Waters, OGRS 5/68 (summary in Annex 5.2)
- The Ordinance on Dangerous Substances in Waters, OGRS 31/82 (summary in Annex 5.3)

Namely, the Ordinance on Dangerous Substances in Waters prescribes the maximum allowable concentrations of pollutants in wastewater for different classes of water courses.

On the other hand the Ordinance on Classification of Water Courses defines water quality classification of all major water courses in Serbia.

The Ordinance on Classification of Waters defines required basic water quality parameters for different classes of waters.

It should be noted that the following classification of local water courses applies:

1. Vrla – from source to Surdulica: Class I
2. Vlasina - from Vlasina Lake to Lužnica: Class I

The critical flow in the receiving water body that is representative for verification of the compliance with the set quality class of the receiving water body is an average monthly low flow of 95% probability.

Required treatment/removal efficiency

The compliance with the following two sets of criteria should be considered.

- **<u>National criteria (imission standard) based on a carrying capacity of the receiving water body;</u>**
- **<u>EU Directive (emission standards).</u>**

Both approaches have got their own specifics:

- **<u>Imission standards</u>** are properly oriented towards environmental protection; they take into account size, characteristics and typical flows of receiving water bodies, in such a way that a small water course requires higher treatment efficiencies than large rivers. It is important to establish a typical, representative low flow in a receiving water body that is used for challenging treatment processes. In accordance with the national standard this is the average monthly low flow of 95% probability in the receiving water body.
- **<u>Emission standards</u>** generally prescribe the required effluent standards irrespective of hydrologic conditions (with exception of sensitive areas), which can be more practical to implement, and are widely used throughout the EU.

The following table gives an overview of the required water quality parameters in different classes of water courses.

Table 5.34: Overview of required water quality for different classes of water courses

No	Parameter	Class I	Class IIa	Class IIb
1	Suspended matter - dry weather (mg/l)	10	30	40
2	Total dry residue – dry weather (mg/l)	350	1.000	1.000
3	pH	6,8-8,5	6,8-8,5	6,8-8,5
4	Dissolved oxygen (mg O_2/l), minimum	8	6	5
5	Five-day biochemical oxygen demand – BOD_5 (mg O_2/l)	2	4	6
6	Visible waste matter	without	without	without
7	Colour	without	without	without
8	Smell	without	without	without

The required treatment efficiency is calculated in accordance with the following formula:

$$TE = \left(1 - \frac{C_{REC}}{C_{WW}}\right) x100[\%]$$

Where:

TE	-	Required treatment efficiency (%);
C_{REC}	-	Maximum allowed concentration in the receiving water body (mg/l);
C_{WW}	-	Pollutant concentration in wastewater (mg/l);
C_{TWW}	-	Pollutant concentration in effluent (treated wastewater) (mg/l).

Conclusions

Future actions shall depend on whether and when the national standards will be harmonized with the EU directives, which are the basics for the WWTP design. In case the harmonisation does not take place in due course, and the compliance with the local standards is still not ensured, the WWTP will have to be upgraded accordingly, in order to enhance its treatment efficiency in a way that the national requirements will be fulfilled.

At present (2009), a new Water Law is still in preparation. This Water Law is aiming to incorporate European regulations on water quality (Water Framework Directive, EU directive 91/271/EEC on urban wastewater treatment). Therefore, the European legislation probably offers the best perspective for long term and sustainable planning, and shall be applied in this study.

5.2.2.4 Technical Description of Considered Wastewater Treatment Options

Process analysis and selection is one of the key aspects of treatment plant design. Selection of a given process depends on the following:

- Adopted design criteria (EU, National, as applicable)
- Design conditions issued by competent national authorities
- Characteristics of wastewater
- Compatibility of the various operations and processes
- Available means to dispose of the ultimate contaminants
- Environmental and economic feasibility of various systems

Wastewater treatment plants must be designed to meet a number of conditions that are influenced by flowrates, wastewater characteristics and a combination of both (mass loading). Peak process loading rates are important for sizing of process units and their support systems so that a treatment plant performance objectives can be achieved consistently and reliably. The overall objective of wastewater treatment is to provide acceptable wastewater conditions while complying with overall performance requirements.

The following key factors are considered in evaluating and selecting unit operations and processes for a WWTP:

- Process applicability
- Applicable flow range and flow variation

- Influent wastewater characteristics, inhibiting and unaffected constituents
- Climatic considerations
- Reaction kinetics and reactor selection
- Performance
- Treatment residuals
- Sludge processing
- Environmental constraints
- Chemical and energy requirements
- Personnel requirements
- Operating and maintenance requirements
- Ancillary processes
- Reliability, complexity and compatibility
- Land availability

Treatment levels achievable with various combinations of unit operations are shown in the following table.

Table 5.35: Treatment levels achievable with various combinations of treatment processes

Treatment Process	SS (mg/l)	BOD_5 (mg/l)	COD (mg/l)	Total N (mg/l)	NH_3–N (mg/l)	PO_4 as P (mg/l)	Turbidity (NTU)
Activated sludge + filtration	4–6	< 10	30–70	15–35	15–25	4–10	0.3–5
Activated sludge + filtration + GAC	< 3	< 1	5–15	15–30	15–25	4–10	0.3–3
Activated sludge / nitrification, single stage	10–25	5–15	20–45	20–30	1–5	6–10	5–15
Activated sludge / nitrification/ de-nitrification, separate stages	10–25	5–15	20–35	5–10	1–2	6–10	5–15
Metal salt addition to activated sludge	10–20	10–20	30–70	15–30	15–25	< 2	5–10
Metal salt addition to activated sludge + nitrification/ de-nitrification + filtration	< 5	< 5	20–30	3–5	1–2	< 1	0.3–3
Mainstream biological phosphorus removal	10–20	5–15	20–35	15–25	5–10	< 2	5–10
Mainstream biological nitrogen and phosphorus removal + filtration	< 10	< 5	20–30	< 5	< 2	< 1	0.3–3

Preliminary Treatment

The objective of preliminary treatment is the removal of coarse solids and other large materials often found in raw wastewater. Removal of these materials is necessary to enhance the operation and maintenance of subsequent treatment units. Preliminary treatment operations typically include coarse screening, grit removal and, in some cases, comminution of large objects. In grit chambers, the velocity of water through the chamber is maintained sufficiently high, or air is used, to prevent settling of most of organic solids. Grit removal is not included as a preliminary treatment step in most small wastewater treatment plants.

A compact unit is used where complete mechanical treatment of wastewater is needed. Several process steps are integrated within a stainless steel container, as follows:

- Screening
- Screening press
- Grit trap and grease removal
- Grit classifier
- Screenings and grit washing

Primary Treatment

The objective of primary treatment is the removal of settleable organic and inorganic solids via sedimentation, and the removal of materials that will float (scum) by skimming. Approximately 25–50% of the incoming biochemical oxygen demand (BOD_5), 50–70% of the total suspended solids (SS) and 65% of the oil and grease are removed during primary treatment. Some organic nitrogen, organic phosphorus and heavy metals associated with solids are also removed during primary sedimentation, although colloidal and dissolved constituents are not affected. The effluent from primary sedimentation units is referred to as primary effluent.

Primary sedimentation tanks or clarifiers may be circular or rectangular basins, typically 3–5 m deep, with hydraulic retention times between 2 and 3 hours. Settled solids (primary sludge) are normally removed from the bottom of the tanks by sludge rakes that scrape the sludge to a central well, from which it is pumped to sludge processing units. Scum is swept across the tank surface by water jets or mechanical means, from which it is also pumped to sludge processing units.

For smaller WWTPs of this type, primary treatment is usually not a prerequisite.

Biological Treatment

The objective of biological treatment is further treatment of the effluent from primary treatment to remove residual organics and suspended solids. In most cases, secondary treatment follows primary treatment, and involves the removal of biodegradable dissolved and colloidal organic matter using aerobic biological treatment processes. Aerobic biological treatment is performed in the presence of oxygen by aerobic microorganisms (principally bacteria) that metabolize the organic matter in the wastewater, thereby producing more microorganisms and inorganic end-products (principally CO_2, NH_3 and H_2O). Several aerobic biological processes are used, differing primarily in the manner in which oxygen is supplied to the microorganisms and in the rate at which organisms metabolize the organic matter.

High-rate biological processes are characterized by relatively small reactor volumes and high concentrations of microorganisms compared with low-rate processes. Consequently, the growth rate of new organisms is much greater in high-rate systems because of the well-controlled environment. The microorganisms must be separated from the treated wastewater via sedimentation to produce clarified secondary effluent. The sedimentation tanks used in secondary treatment, often referred to as secondary clarifiers, operate in the same basic manner as the primary clarifiers described previously. The biological solids removed during secondary sedimentation, called secondary or biological sludge, are normally combined with primary sludge for sludge processing.

Common high-rate processes include the activated sludge processes, trickling filters or bio-filters, and oxidation ditches.

In the activated sludge process, the dispersed-growth reactor is an aeration tank or basin containing a suspension of the wastewater and microorganisms, called the mixed liquor. The contents of the aeration tank are mixed vigorously by aeration devices that also supply oxygen to the biological suspension. Aeration devices commonly used include submerged diffusers that release compressed air and mechanical surface aerators that introduce air by agitating the liquid surface. Hydraulic retention time in the aeration tanks usually ranges from 3 to 8 hours, but can be higher with high BOD_5 wastewaters. Following the aeration step, the microorganisms are separated from the liquid by sedimentation and the clarified liquid is called secondary effluent. A portion of the biological sludge is recycled to the aeration basin to maintain a high mixed-liquor suspended solids (MLSS) level. The remainder is removed from the process and sent to sludge processing to maintain a relatively constant concentration of microorganisms in the system. Several variations of the basic activated sludge process, such as extended aeration and oxidation ditches, are in common use, but the principles are similar.

High-rate biological treatment processes, in combination with primary sedimentation, typically remove 85% of the BOD_5 and SS originally present in the raw wastewater and some of the heavy metals. Activated sludge generally produces an effluent of slightly higher quality, in terms of these constituents, than biofilters. When coupled with a disinfection step, these processes can provide substantial, but not complete, removal of bacteria and viruses. However, they remove very little phosphorus, nitrogen, non-biodegradable organics and dissolved minerals.

Nitrogen Removal

Nitrogen in untreated wastewater is principally in the form of ammonia or organic nitrogen, both soluble and particulate. During biological treatment, most of the particulate organic nitrogen is transformed to ammonium or other inorganic forms. Additional data regarding the effects of various treatment operations and processes on nitrogen removal are presented in the following table.

Table 5.36: Effect of various treatment operations and processes on nitrogen removal

Treatment Operation/Process	Organic Nitrogen Removal	NH_3/NH_4+ Removal	NO_3- Removal	Removal of Total N (%)
Conventional Treatment				
Primary	10–20%	No effect	No effect	0–5
Secondary	15–50%	< 10%	Slight effect	10–30
Biological Processes				
Bacterial assimilation	No effect	40–70%	Slight effect	30–70
De-nitrification	No effect	No effect	80–90%	70–95
Harvesting algae	Partial transformation to NH_3/NH_4^+	Cells	Cells	50–80
Nitrification	Limited effect	NO_3^-	No effect	5–20
Oxidation ponds	Partial transformation to NH_3/NH_4^+	Partial removal by stripping	Partial removal by nitrification/ denitrification	20–90
Chemical Processes				
Breakpoint chlorination	Uncertain	90–100%	No effect	80–95
Chemical coagulation	50–70%	slight effect	Slight effect	20–30
Carbon adsorption	30–50%	slight effect	Slight effect	10–20
Selective ion exchange for ammonium	slight, uncertain	80–97%	No effect	70–95
Selective ion exchange for nitrate	slight, uncertain	Slight effect	75–90%	70–90
Physical Operations				
Filtration	30–95% of suspended organics removed	Slight effect	Slight effect	20–40
Air stripping	No effect	60–95%	No effect	50–90
Electro dialysis	100% of suspended organics removed	30–50%	30–50%	40–50
Reverse osmosis	60–90%	60–90%	60–90%	80–90

Phosphorus Removal

With most wastewater, approximately 10% of phosphorus corresponding to the insoluble portion is normally removed by primary settling. Besides the amount removed that is incorporated into cell tissue, the additional removal achieved in conventional biological treatment is minimal, as almost all phosphorus present after primary sedimentation is soluble. The effects of conventional and other treatment on phosphorus removal are listed in the following table.

Table 5.37: Effect of various treatment operations and processes on phosphorus removal

Treatment Operation or Process	Removal of Phosphorus Entering System (%)
Conventional Treatment	
Primary	10–20
Activated sludge	10–25
Trickling filter	8–12
Rotating biological contactors	8–12
Biological Phosphorus Removal Only	
Mainstream treatment	70–90
Sidestream treatment	70–90
Combined Biological Nitrogen and Phosphorus Removal	70–90
Chemical Removal	
Precipitation with metal salts	70–90

Treatment Operation or Process	Removal of Phosphorus Entering System (%)
Precipitation with lime	70–90
Physical Removal	
Filtration	20–50
Reverse osmosis	90–100
Carbon adsorption	10–30

In recent years, a number of biological phosphorus removal processes have been analysed as alternatives to chemical treatment.

Tertiary and/or Advanced Treatment

Tertiary and/or advanced wastewater treatment is employed when specific wastewater constituents that cannot be removed by secondary treatment must be removed. Individual treatment processes are necessary to remove nitrogen, phosphorus, additional suspended solids, refractory organics, heavy metals and dissolved solids. Because advanced treatment usually follows high-rate secondary treatment, it is sometimes referred to as tertiary treatment. However, advanced treatment processes are sometimes combined with primary or secondary treatment (e.g., chemical addition to primary clarifiers or aeration basins to remove phosphorus) or used in place of secondary treatment (e.g., overland flow treatment of primary effluent).

An adaptation of the activated sludge process is often used to remove nitrogen and phosphorus. Effluent from primary clarifiers flows to the biological reactor, which is physically divided into zones by baffles and weirs. In sequence, these zones are as follows:

- Anaerobic fermentation zone (characterized by very low dissolved oxygen levels and the absence of nitrates)
- Anoxic zone (low dissolved oxygen levels, but with nitrates present)
- Aerobic zone (aerated)

The function of the first zone is to condition the group of bacteria responsible for phosphorus removal by stressing them under low oxidation-reduction conditions, which results in a release of phosphorus equilibrium in the cells of the bacteria. On subsequent exposure to an adequate supply of oxygen and phosphorus in the aerated zones, these cells rapidly accumulate phosphorus considerably in excess of their normal metabolic requirements. Phosphorus is thereby removed from the system with the waste-activated sludge.

Most of the nitrogen in the influent is in the form of ammonia, and passes through the first two zones virtually unaltered. In the third aerobic zone, the sludge age is such that almost complete nitrification takes place, and the ammonia nitrogen is converted to nitrites and then to nitrates. The nitrate-rich mixed liquor is then recycled from the aerobic zone back to the first anoxic zone. There, denitrification occurs, where the recycled nitrates, in the absence of dissolved oxygen, are reduced by facultative bacteria to nitrogen gas, using the influent organic carbon compounds as hydrogen donors. The nitrogen gas merely escapes to atmosphere. In the second anoxic zone, those nitrates that were not recycled are reduced by the endogenous respiration of bacteria. In the final re-aeration zone, dissolved oxygen levels are again raised to prevent further denitrification, which would impair settling in the secondary clarifiers to which the mixed liquor then flows.

The main objectives of wastewater filtration are to remove suspended solids, where an effluent limit of 10 mg/l or less suspended solids is required, as well as to remove particulate carbonaceous BOD_5 and residual insolubilized phosphorous. Addition of the coagulant aid is considered good practice for process efficiency improvement

5.2.2.5 Wastewater treatment – Alternative 1

Wastewater collection and treatment alternative 1 assumes wastewater collection in two separate catchment areas; southern and northern and treatment of collected wastewater at two wastewater treatment plants; - northern WWTP Vlasina and southern – WWTP Vrla.

5.2.2.5.1 WWTP Vlasina (6.000 PE)

WWTP Vlasina is to be constructed downstream of the Vlasina dam, at the location of the existing disfunctional wastewater treatment facility. The existing plant included two sets of bio-discs and sludge drying beds. Based on the information provided by the PUC Surdulica, it has never been operational.

Figure 5.30: General location of Vlasina WWTP

Figure 5.31: Existing status of Vlasina WWTP location

5.2.2.5.1.1 Process design criteria

In accordance with the demographic projections and related wastewater flows the following main design parameters have been adopted:

Table 5.38: Pollutant loading expressed in population equivalent for the end of Project period

Number of PE	Year 2006	Year 2035
WWTP Vlasina	2,919	6,089

The capacity of WWTP Vlasina is determined in accordance with the predicted quantity of the wastewater at the end of the project period, considering development of sewerage collection system and including infiltration. Typical hydraulic design loading of WWTP Vlasina is shown in the following table.

Table 5.39: Hydraulic load of the WWTP Vlasina at the end of the project period

Flow units	Average Flow	Maximum Daily Flow	Maximum Hourly Flow
(m^3/d)	1754	2445	
(m^3/h)	73	102	171
(l/s)	20.3	28.3	47.5

Table 5.40: Typical pollutant loading of the WWTP Vlasina (end of the project period, maximum daily flows)

Contaminants	Units	WWTP Vlasina
Suspended solids	kg/d	594
Biological oxygen demand (BOD_5)	kg/d	509
COD	kg/d	1019
Total nitrogen	kg/d	93
Total phosphorus	kg/d	17

5.2.2.5.1.2 Process description

Conventional wastewater treatment is a combination of physical and biological processes applied in order to remove the organic matter from solution. Wastewater is most commonly treated in a three-stage process including preliminary, primary and secondary treatment. In some cases there is a need for tertiary treatment to reduce nutrients (principally nitrogen and phosphorus, causing the eutrophication).

Preliminary treatment consists of the removal of coarse solids and grit from the influent to reduce problems associated with the operation and maintenance of downstream processes and equipment. Preliminary treatment processes are generally common to all of the treatment processes that follow and are normally required as good practice.

The purpose of screening is to remove or reduce large size solids and trash that may interfere with proper operation of a treatment plant. Screens generally may be classified as coarse and fine screens.

Hand cleaned coarse screens are used ahead of pumping station, as well as for stand-by screening in bypass channel for service during high flow-periods, power failure or when mechanically cleaned downstream screens are being repaired.

Fine screens are provided downstream pumping station, installed at the inlet of the grit and grease removal unit.

Wastewater grit material consists of sands of various size, coffee grounds, cigarette filter tips, and other large sized, relatively non-putrescible organic and inorganic substances. The reasons for removal of wastewater grit can be summarized as follows:

- to protect moving mechanical equipment in biological treatment from abrasion and accompanying abnormal wear
- to reduce formation of heavy deposits in pipelines, channels and conduits

Secondary treatment reduces BOD and SS to a level where the effluent is suitable for disposal except where tertiary treatment is required. Secondary treatment is a two stage process, comprised of activated sludge process followed by secondary sedimentation to separate micro-organisms from the final effluent.

Activated sludge is suspension of micro-organisms, both active and dead, in wastewater consisting of entrapped and suspended colloidal and dissolved organic and inorganic materials. The activated sludge process is aerobic, biological process, which uses metabolic reactions of micro-organisms to attain an acceptable effluent quality by removing substances exerting oxygen demand.

In bioaeration basins formed microbial floc particles are brought into contact with the organic components of wastewater. Contents of the reactor basin are referred to as mixed liquor suspended solids (MLSS), and consist for the most part of micro-organisms and inert and non-biodegradable matter. The overall reactions, occurring in the activated sludge process are determined by the composite metabolism of micro-organisms in the activated sludge.

Wastewater is introduced into the reactor tank, where aeration is applied. Extended aeration plants are characterised by the long term aeration, high MLSS, high sludge return ratio and low sludge wastage. This system is normally applied for plants serving less than 20.000 PE. The usage of a long detention time (16-24 hours) in the aeration basin can be particularly favourable, since the design permits the plant to operate effectively even though hydraulic and organic loads vary widely.

Since activated sludge undergoes aerobic digestion in the reactor basin, more oxygen is required in the basin compared to other activated sludge systems. Long sludge retention time (SRT) and excess oxygen during the night-time permits some nitrification to occur even with daytime deficiency, yielding nitrification/denitrification cycle.
Air is introduced into the system to satisfy the requirements of the activated sludge and to keep the activated sludge dispersed in aeration liquor. Oxygen transfer to activated sludge is accomplished by oxygen absorbed from diffused bubbles of air entrained in mixed liquor. The designed fine bubble diffused air system consists of diffusers that are submerged in wastewater, header pipes, air mains and the blowers and appurtenances through which air passes.

To achieve effective operation of an activated sludge process biological sludge of good physical quality should be maintained. The purpose of sludge return is to maintain sufficient concentration of activated sludge in the reactor basin. The SVI (sludge volume index) is used as an indication of the settling characteristics of the sludge.

For extended aeration two compact units are foreseen. Each unit consists of the final clarifier incorporated into circular bioaeration basin.

The final step in reduction of BOD_5 and suspended solids is separation of activated sludge solids from mixed liquor. A presence of large volume of flocculant solids in mixed liquor requires special consideration in the design of a final sedimentation tank. An important function of the final sedimentation tank is to maintain wastewater quality produced by the preceding unit processes and to remove and return sludge to the system.

The final sedimentation tank is designed in a conservative manner in order to handle variations of the hydraulic load and high MLSS typical for extended aeration system.

Circular tanks built inside each bioaeration tank with the central feed are foreseen. Bridge which rotates along the sides of the sedimentation tank is used for sludge collecting. The bridge serves as a support for the sludge removal system, which consists of a scraper and a suction manifold from which sludge is pumped. Due to the fact that large amount of solids may be lost in the effluent if design criteria are exceeded, effluent overflow rates are based on the peak daily flow conditions.

Owing to the continuous arrival of organic matter with the wastewater the bacteria grow and reproduce continuously. To maintain the system in equilibrium, it is necessary to remove an equivalent amount of biomass. The waste activated sludge (WAS) can be taken directly from the activated sludge tank or from the external sludge recirculation system, i.e. from the sludge settled in the bottom of the secondary clarifiers.

The following items are therefore essential in the activated sludge system:

- **aeration tank**
- **settling tank (secondary sedimentation tank, also called final clarifier)**
- **pumps for returning the sludge (external recirculation)**
- **removal of the biological excess sludge**

Treated water overflows into the collection channel, where UV disinfection is foreseen.

Disinfection with ultra violet (UV) radiation in the range off 240 to 290 nm becomes widely accepted as an alternative to conventional disinfection with chemicals. Design and performance requirements are becoming more stringent as effluent coliform limits have decreased from 1.000 to 70 MPN/100 ml in many treatment plant permits. The use of UV disinfection as part of wastewater reclamation has gained attention in many countries.

In UV disinfection systems UV light is produced by germicidal lamps, which are enclosed in a stainless steel chamber. As the wastewater flows around the UV lamps, the microorganisms are exposed to a lethal dose of UV energy. UV-rays penetrate through small particles and attack micro-organisms at their DNA/RNA-core. This mechanism causes inability to replicate.

The effectiveness of ultraviolet disinfection is directly related to the dose (quantity of energy) absorbed by the micro-organisms. The dose is the product of the rate at which energy is delivered (intensity) and the time of exposure to this intensity.
Following factors influence efficiency of UV disinfection:

- contact time
- intensity and nature of physical agent
- temperature
- number of organisms
- types of organisms
- nature of suspended solids in the water

Expected lamp life is 12.000 hours. Faecal coliform reduction is >99.99 %.

Aerobic sludge stabilisation is in principle similar to the aerated activated sludge system for wastewater treatment.

Surplus sludge is concentrated by thickening and pumped to a tank, where the thickened sludge is aerated for an extended time. Air is supplied by means of diffusers.

The main objectives by sludge treatment are:

- Reduction of volume by dewatering,
- reduction of volatile organic matter in order to reduce solids and minimise the risk of odour nuisance, and
- disinfection to avoid spreading of diseases.

The thickening process usually corresponds to an increase in the concentration of the sludge collected in the final sedimentation tanks. Thickening has following advantages:

- improved reliability along the entire water treatment line
- reduced volume of sludge for conditioning prior to dehydratation
- improved performances of the dewatering unit

The gravity thickener is designed on the basis of hydraulic surface loading and solids loading. The critical design parameter for the thickener is the loading in terms of weight of total solids per area and per day.

Sludge, introduced continuously or intermittently, is retained in the reactor for varying periods of time.
Thickened sludge will be mechanically dewatered up to a minimum of approximately 20% dry solids and disposed to the landfill. Methods of mechanical dewatering include centrifuges, belt presses and plate presses. The first two are likely to be the most appropriate. Sand drying beds could be used where there is sufficient space available.

5.2.2.5.1.3 Description of plant process units

A hand-cleaned **coarse screen** is used ahead of lifting pumping station, to protect pumps, valves and pipelines from damage or clogging by rags and large objects. A screen is composed of parallel bars, with 50 mm clear spacing between bars.

Coarse screenings consist of debris such as rocks, branches, leaves, paper, plastics or rags. The quantity and characteristics of screenings vary depending on the size of the bar screen opening and the type of sewer system. The quantity of screenings ranges between 4 – 11 l/1000 m^3 of wastewater flow.

Wastewater flows into the sump, where two working and one stand-by submersible pumps, each capacity of 25 l/s are installed.

Two compact units, each capacity of 25 l/s, comprised of fine screen followed by grit and grease removal unit are foreseen.

Wastewater passes through a **fine screen**, with clear openings of 6 mm in order to remove various types of plastic material, un-decomposed food waste, and other material. Screening material is washed, compacted and conveyed by a screw conveyor into container.

Afterwards, the wastewater flows into the **grit trap** where settling of grit takes place. Settled material from the bottom of the grit trap is lifted and dewatered by a screw conveyor to the container.

Grit trap is equipped with aeration equipment. An air rate is adjusted to create a velocity near the floor low enough to allow grit to settle.

Basic design criteria are presented in the following table:

Table 5.41: Basic Design Criteria for the Preliminary Treatment

Manually cleaned Coarse Screen	
Type	bar screen
Clear openings, mm	50
Number of units	
Quantity of screenings, m^3/day	15
Lifting Pumping station, frequency controlled	

Type	centrifugal, submersible
No of working units	2
No of stand-by units	1
Capacity of the unit, l/s	25
Mechanically cleaned Fine Screen	
Clear openings, mm	6
Screening washing and compaction	included
Number of units	2
Quantity of screenings, m^3/day	110
Sand/grease removal	
Detention time, min	4
Required volume, m^3	6.6
Number of units	2
Grit quantity, m^3/day	73

Wastewater then enters a **bioaeration tank**, where BOD removal takes place. At the inlet of the tank it is combined with sludge solids recycled from settling tanks, mixed and aerated.

Two bioaeration tanks, each of 1,160 m^3 are foreseen, providing hydraulic retention time of 23 hours, according to the maximum daily flow. Air is introduced through porous diffusers, placed at the bottom of the tank.
Process operates in a continuous flow mode. The mixed liquor from the aeration tank is transferred to a settling tank to allow gravity separation of the MLSS from the system.

Table 5.42: Basic Design Criteria for the Secondary Treatment

Extended aeration	
BOD_5 remain after preliminary treatment, mg/l	198
F/M, kgBOD5/kg MLVSS/d	0.07
MLSS (g/m^3)	4000
SRT, d	20
Number of units	2
Volume of the tank, m^3	1160
Depth of the tank, m	5
Diameter of the tank, m	19
HRT, h	23

Recycling rate, %	100
Recirculation pumps capacity, l/s	15
Air requirements	
Oxygen demand, $kgO_2/kg\ BOD_5$ removed	2
Oxygen demand, kg/d	919
Final Clarifier	
Overflow rate, $m^3/m^2/d$	32
Number of units	2
Surface per clarifier, m^2	75
Diameter, m	9.7
Side water depth, m	4
Volume of the clarifier, m^3	298
Excess sludge quantity, kg/d	184
Sludge volume, m^3/d	23
Number of excess sludge pumps	2
Capacity of excess sludge pumps, l/s	3

Clarified water passes through channel where disinfection by UV lamps takes place.

Introduction of a tertiary treatment including nutrient removal could be considered at a later stage, subject to specific requirements that may arise in the future.

5.2.2.5.1.4 Process diagram

Figure 5.32: WWTP Vlasina Wastewater Treatment Process Scheme

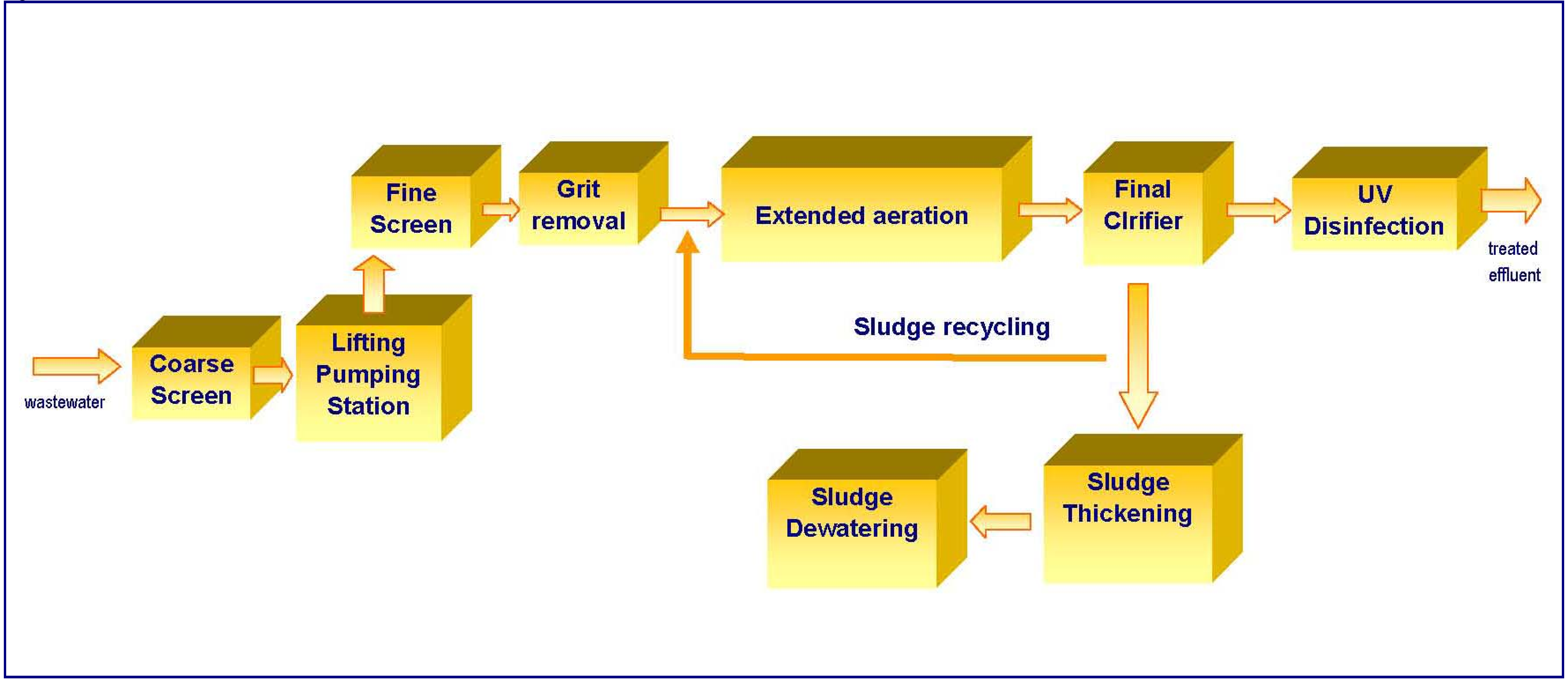
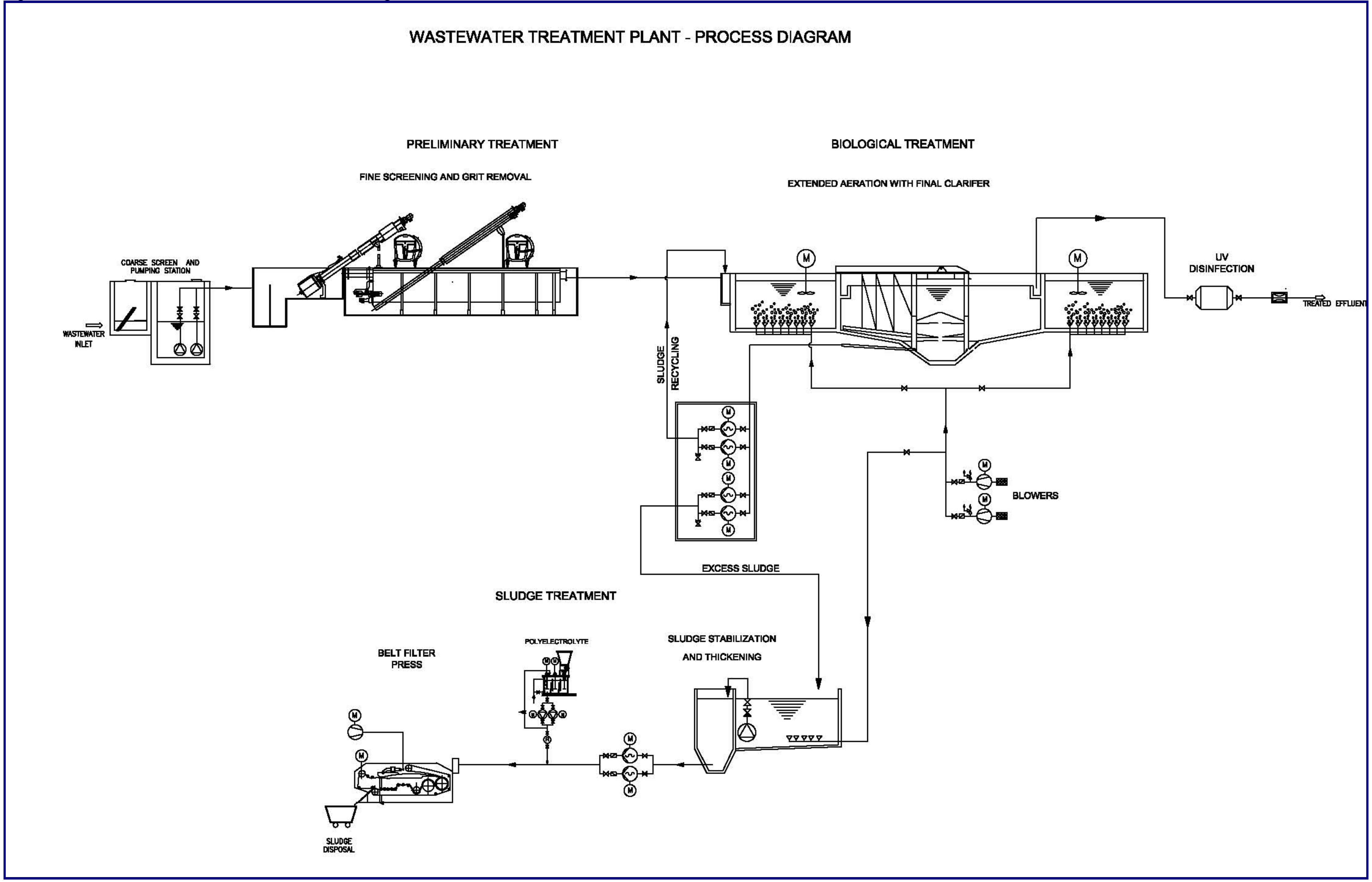


Figure 5.33: Wastewater Treatment Plant – Process Diagram



5.2.2.5.1.5 Cost estimate

This section presents an investment cost estimate for the described wastewater treatment plant Vlasina of the capacity 6.000 PE.

Apart from the investment cost of construction works and necessary equipment, it is also envisaged to prepare a corresponding detail design of the facility with appurtenant investigations, and to conduct 12-month trial run and operation of the plant, including training of the local staff. These activities are to be funded through suitable technical assistance programme.

These investment costs have been generated on the basis of typical unit costs for this type and size of the facility, and described level of treatment (primary + secondary wastewater treatment).

Table 5.43: Investment costs estimate - construction of WWTP Vlasina (6,000 PE)

1 WWTP Vlasina - capacity 6.000 PE						
	Description	**Civil works (€)**	**Equipment (€)**	**Piping (€)**	**TA (€)**	**Total costs (€)**
1.1	Investigation works and Detail Design				120,000	**120,000**
1.2	Construction of WWTP Vlasina	960,600	560,350	80,050		**1,601,000**
1.3	Trial run and operation of 12 months				80,050	**80,050**
1	**WWTP Vlasina - total**	**960,600**	**560,350**	**80,050**	**200,050**	**1,801,050**

5.2.2.5.2 <u>WWTP Vrla (3.000 PE)</u>

5.2.2.5.2.1 Process design criteria

In accordance to census 2002, as well as projected population increase the following assumptions are made:

Table 5.44: Pollutant loading expressed in population equivalent for the end of Project period

Number of PE	**Year 2006**	**Year 2035**
WWTP Vrla	1,739	3,144

The Capacity of WWTP Vrla is determined in accordance with the actual, measured consumption of water within the studied area, and predicted production of the wastewater, as well as predicted quantity of the wastewater at the and of the project horizon, considering development of sewerage collection system and including infiltration.

In order to define the capacity of WWTP Vrla, various assumptions were made, as shown in the following table.

Table 5.45: Hydraulic load of the WWTP Vrla at the end of the project period, 2035

Flow units	Average Flow	Maximum Daily Flow	Maximum Hourly Flow
(m^3/day)	1,028	1,313	
(m^3/h)	43	55	90
(l/s)	11.9	15.2	25

Table 5.46: Typical pollutant loading of the WWTP Vrla (end of the project period, maximum daily flows)

Contaminants	Units	WWTP Vrla
Suspended solids	kg/d	281
Biological oxygen demand (BOD_5)	kg/d	241
COD	kg/d	482
Total nitrogen	kg/d	44
Total phosphorus	kg/d	8

5.2.2.5.2.2 Process description

See section 5.2.1.5.1.2.

5.2.2.5.2.3 Description of plant process units

A hand-cleaned coarse screen is used ahead of lifting pumping station, to protect pumps, valves and pipelines from damage or clogging by rags and large objects. A screen is composed of parallel bars, with 50 mm clear spacing between bars.

Coarse screenings consist of debris such as rocks, branches, leaves, paper, plastics or rags. The quantity and characteristics of screenings vary depending on the size of the bar screen opening and the type of sewer system. The quantity of screenings ranges between 4 – 11 l/1000 m^3 of wastewater flow.

Wastewater flows into the sump, where two working and one stand-by submersible pumps, each capacity of 15 l/s are installed.

Two compact units, each capacity of 20 l/s, comprised of fine screen followed by grit and grease removal unit are foreseen.

Wastewater passes through a fine screen, with clear openings of 6 mm in order to remove various types of plastic material, un-decomposed food waste, and other material. Screening material is washed, compacted and conveyed by a screw conveyor into container.

Afterwards, the wastewater flows into the grit trap where settling of grit takes place. Settled material from the bottom of the grit trap is lifted and dewatered by a screw conveyor to the container.

Grit trap is equipped with aeration equipment. An air rate is adjusted to create a velocity near the floor low enough to allow grit to settle.

Basic design criteria are presented in the following table:

Table 5.47: Basic Design Criteria for the Preliminary Treatment

Manually cleaned Coarse Screen	
Type	bar screen
Clear openings, mm	50
Number of units	
Quantity of screenings, m^3/day	8
Lifting Pumping station, frequency controlled	
Type	centrifugal, submersible
No of working units	2
No of stand-by units	1
Capacity of the unit, l/s	13
Mechanically cleaned Fine Screen	
Clear openings, mm	6
Screening washing and compaction	included
Number of units	2
Quantity of screenings, m^3/day	59
Sand/grease removal	
Detention time, min	4
Required volume, m^3	4.2
Number of units	2
Grit quantity, m^3/day	39

Wastewater enters the bioaeration tank, where BOD removal takes place. At the inlet of the tank it is combined with sludge solids recycled from settling tanks, mixed and aerated. Two bioaeration tanks, each volume of 550 m^3 are foreseen, providing hydraulic retention time of 23 hours, according to maximum daily flow. Air is introduced through porous diffusers, placed at the bottom of the tank.

Process operates in a continuous flow mode. The mixed liquor from the aeration tank is transferred to a settling tank to allow gravity separation of the MLSS from the system.

Table 5.48: Basic Design Criteria for the Secondary Treatment

Extended aeration	
BOD_5 remain after preliminary treatment, mg/l	174
F/M, kgBOD5/kg MLVSS/d	0.06
MLSS (g/m^3)	4000
SRT, d	20
Number of units	2
Volume of the tank, m^3	550
Depth of the tank, m	5
Diameter of the tank, m	15
HRT, h	23
Recycling rate, %	100
Recirculation pumps capacity, l/s	8
Air requirements	
Oxygen demand, kgO_2/kg BOD_5 removed	2
Oxygen demand, kg/d	435
Final Clarifier	
Overflow rate, $m^3/m^2/d$	32
Number of units	2
Surface per clarifier, m^2	47
Diameter, m	8
Side water depth, m	4
Volume of the clarifier, m^3	187
Excess sludge quantity, kg/d	87
Sludge volume, m^3/d	11
Number of excess sludge pumps	2
Capacity of excess sludge pumps, l/s	3

Clarified water passes through channel where disinfection by UV lamps takes place.

5.2.2.5.2.4 Process diagram

Figure 5.34: WWTP Vrla Wastewater Treatment Process Scheme

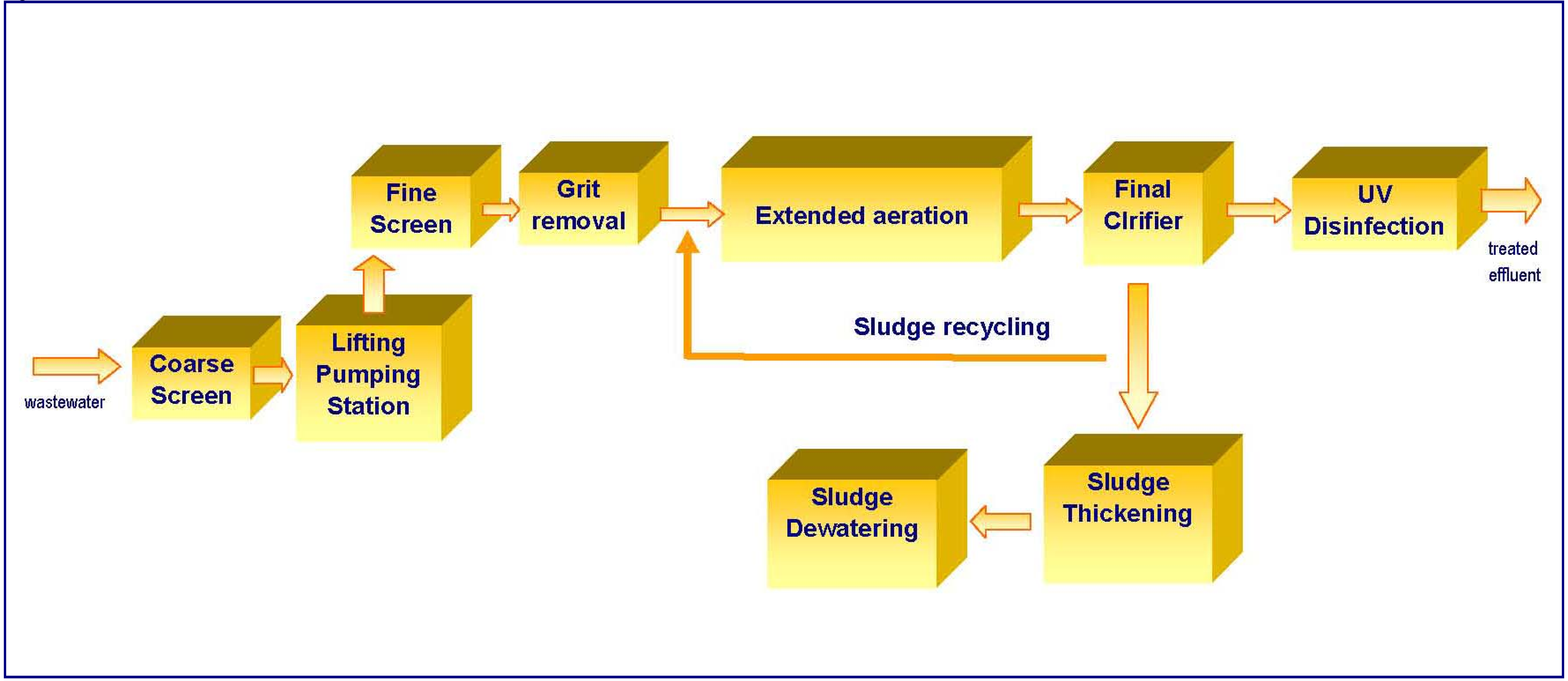
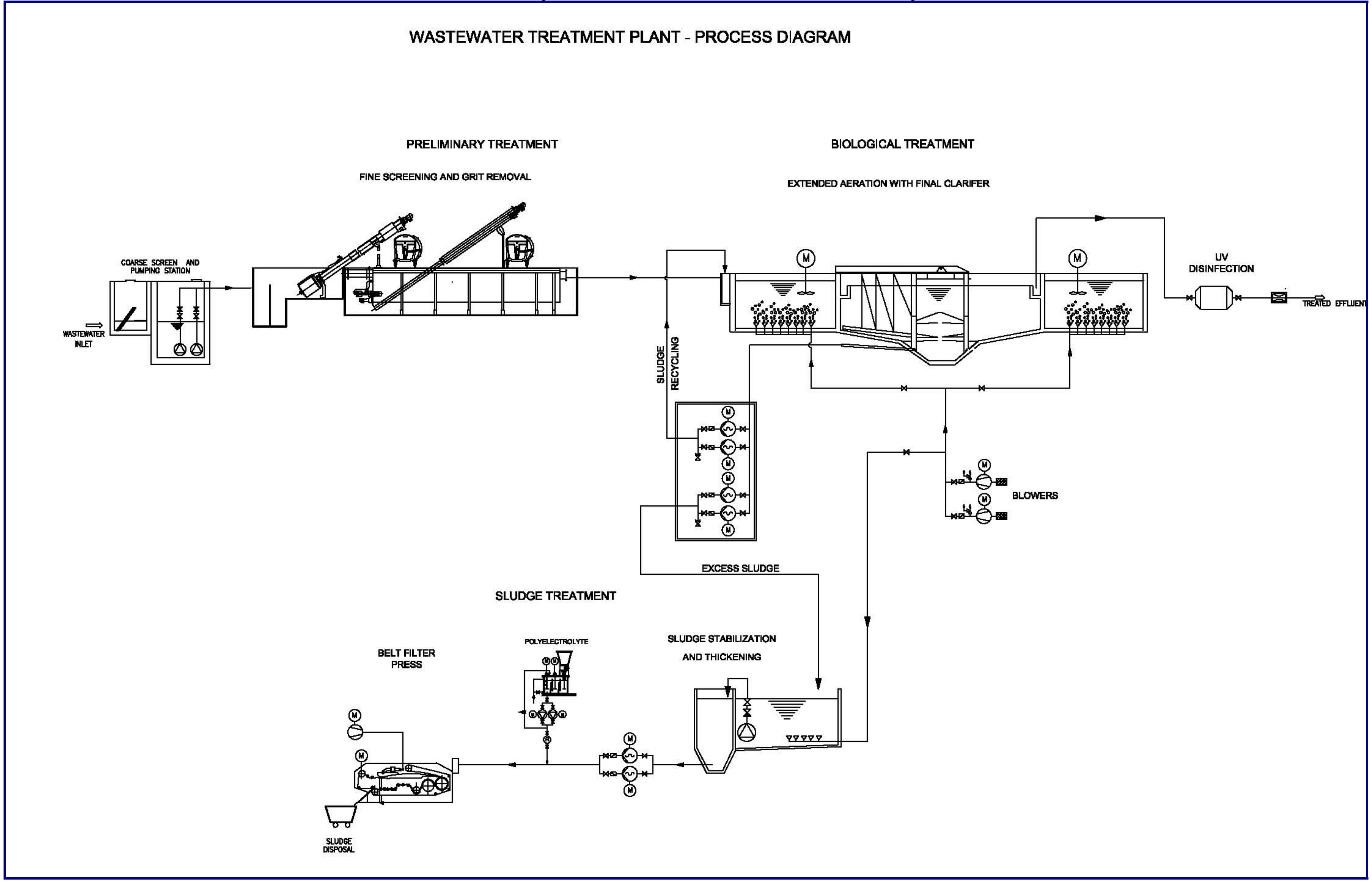


Figure 5.35: Waste Water treatment Plant – Process Diagram



5.2.2.5.2.5 Cost estimate

This section presents an investment cost estimate for the described wastewater treatment plant Vrla of the capacity 3.000 PE.

Apart from the investment cost of construction works and necessary equipment, it is also envisaged to prepare a corresponding detail design of the facility with appurtenant investigations, and to conduct 12-month trial run and operation of the plant, including training of the local staff. These activities are to be funded through suitable technical assistance programme.

These investment costs have been generated on the basis of typical unit costs for this type and size of the facility, and described level of treatment (primary + secondary wastewater treatment).

Table 5.49: Investment costs estimate - construction of WWTP Vrla (3,000 PE)

2 WWTP Vrla - capacity 3.000 PE						
	Description	**Civil works (€)**	**Equipment (€)**	**Piping (€)**	**TA (€)**	**Total costs (€)**
2.1	Investigation works and Detail Design				75,000	**75,000**
2.2	Construction of WWTP Vrla	533,500	388,000	48,500		**970,000**
2.3	Trial run and operation of 12 months				48,500	**48,500**
2	**WWTP Vrla - total**	**533,500**	**388,000**	**48,500**	**123,500**	**1,093,500**

5.2.2.5.3 <u>WWTP-s Klisura and Bozica (500 PE each)</u>

5.2.2.5.3.1 Process design criteria

The Capacities of WWTP-s Klisura and Bozica are determined in accordance with the actual, measured consumption of water within the studied area, and predicted production of the wastewater, as well as predicted quantity of the wastewater at the and of the project horizon, considering development of sewerage collection system and including infiltration.

Table 5.50: Hydraulic load of the WWTP-s Klisura and Bozica at the end of the project period, 2035

Flow units	**Average Flow**	**Maximum Daily Flow**	**Maximum Hourly Flow**
(m^3/day)	120	181.4	
(m^3/h)	5.04	7.56	15.1
(l/s)	1.4	2.1	4.2

Considering specific loads per capita the following assumptions for each plant are made:

Table 5.51: Typical pollutant loading of the WWTP-s Klisura and Bozica (end of the project period, maximum daily flows)

Contaminants	Units	WWTP Vrla
Suspended solids	kg/d	3552.5
Biological oxygen demand (BOD_5)	kg/d	3045
COD	kg/d	6090
Total nitrogen	kg/d	5.58
Total phosphorus	kg/d	12

5.2.2.5.3.2 Process description

The principle of the activated sludge process with intermittent operation consists in the incorporation of all the units, processes and operations normally associated to the conventional activated sludge (primary sedimentation, biological oxidation, secondary sedimentation, sludge pumping) within a single tank. Using only a single tank, these processes and operations become sequences in time and not separated units, such as in conventional processes with continuous flow.

The sequencing batch reactor (SBR) process utilizes a fill-and-draw reactor with complete mixing during the batch reaction step (after filling) and where the subsequent steps of aeration and clarification occur in the same tank. This is accomplished by the establishment of operating cycles with defined duration. The biological mass stays in the reactor, eliminating the need for separate sedimentation and sludge pumping. The retention of biomass occurs because it is not withdrawn with the supernatant (final effluent) after the sedimentation stage, remaining in the tank. The normal treatment cycle is composed of the following stages:

- Fill (entrance of the influent in the reactor)
- React (aeration/mixture of the liquid/biomass contained in the reactor)
- Settle (quiescent settling and separation of the suspended solids from the treated sewage)
- Draw (withdrawal of the treated wastewater from the reactor)
- Idle (removal of the excess sludge from the reactor bottom)

The usual duration of each stage within the cycle can be altered as a function of the influent flow variations, the treatment needs and the sewage and biomass characteristics.

The SBR system also employs preanoxic denitrification using BOD in the influent wastewater. Mixing is used during the fill period to contact the mixed liquor with the influent wastewater. Depending on wastewater characteristics and strength, sufficient BOD and fill time are available to remove almost all of the nitrate remaining in the mixed liquor after the settle and decant steps. Some nitrate removal also occurs during the nonaerated settle and decant periods. Separate mixing provide operating flexibility and is useful for anoxic operation during the aeration period, as well as anaerobic or anoxic contacting during the fill period.

Anoxic operating period is used after a sufficient aerobic time, elapses for nitrification and nitrate production. The nitrate concentration is minimized before settling, and little nitrate is available to compete for readily biodegradable COD (rbCOD) in the fill and initial react period. Thus, anaerobic conditions occur in the fill and initial react period enabling the rbCOD uptake and storage by phosphorous accumulating bacteria instead of rbCOD consumption by nitrate reducing bacteria.

The wasting of excess sludge generally occurs during the last stage (idle), whose purpose is to allow the adjustment of the stages within the operating cycles of each reactor. However, as this stage is optional or may be short, sludge wasting can happen in other phases of the process. The sludge wasting quantity and frequency are established in function with the performance requirements, in the same way as in the conventional continuous flow processes.

There are some variants of the sequencing batch reactor systems related to its operation (continuous feeding and discontinuous supernatant withdrawal) as well as in the sequence and duration of the stages within each cycle. These variations may lead to additional simplifications in the process or to biological nutrient removal.

5.2.2.5.3.3 Description of plant process units

Wastewater passes through manually cleaned bar screen with 25 mm clear openings into the sump. Volume of the sump is designed in order to provide sufficient volume for batch feeding of the SBR reactor.

One duty and one stand-by pumps for SBR feeding, each capacity of 5 l/s are foreseen.

The SBR is a fill and draw reactor with complete mixing during the batch reaction step (after filling) and where the subsequent steps of aeration and clarification occur in the same tank.

Two SBR containers, each dimensions of 6 x 2.4 m are foreseen, in order to provide continuous flow operation (while one tank is in filling mode, another completes its treatment cycle.

The inlet part of the reactor is divided from the reaction part by an baffle. Wastewater enters the inlet part from the top, flows downward and flows under baffle into the reaction tank.

Air is introduced through corse bubble diffusers, placed at the bottom of the tank. One working and one stand-by blowers are foreseen, each capacity of 350 Nm^3/h. The same blowers supply the air for air lift pumps used for sludge wasting.

After aeration cycle, sollids are allowed to setltle under quiescent conditions, resulting in a clarified supernatant that can be discharge as effluent.

Discharge of the clarified effluent starts by opening of the motor driven valve. Effluent flows under the outlet weir into outlet pipe until minimum set level of water in the tank is riched.

The amount and frequency of sludge wasting is determned by performance requirements. Usually occurs during the react phase. Excess sludge is discharged from the SBR reactor by means of air lift pump and transferred into sludge storage tank.

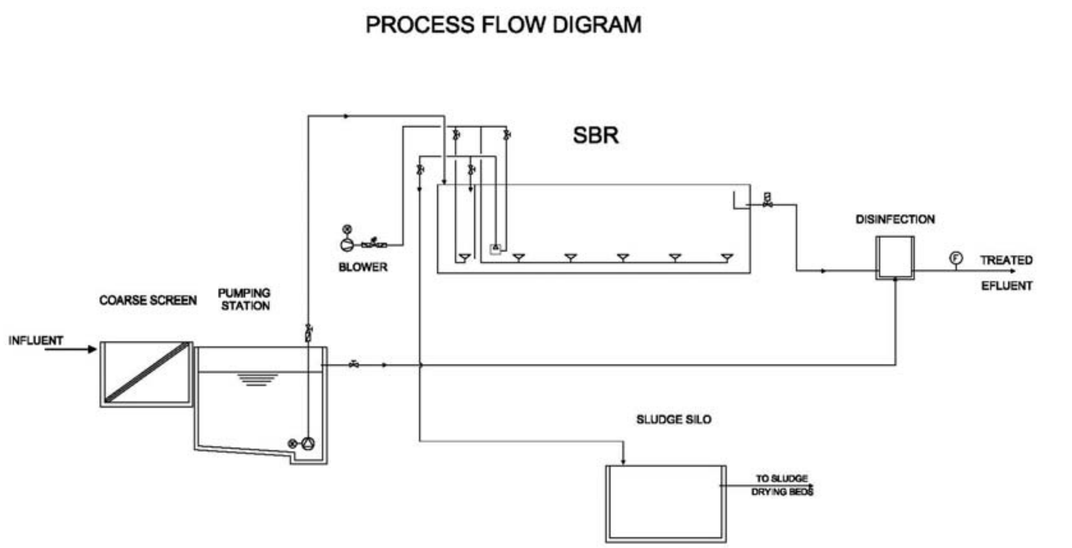
Table 5.52 - Basic Design Criteria for the SBR

Extended aeration	
BOD_5, mg/l	248
F/M, kgBOD5/kg MLVSS/d	0.06
SRT, d	25
Number of units	2
Volume of the tank, m^3	66
Maximum water level, m	2.5
Minimum water level, m	2.0
HRT, h	23
Air requirements	
Oxygen demand, kgO_2/kg BOD_5 removed	2
Oxygen demand, kg/d	90

Clarified water passes through channel where disinfection by UV lamps takes place.

5.2.2.5.3.4 *Process diagram*

Figure 5.36: Process Diagram



5.2.2.5.3.5 *Cost estimate*

Table 5.53: Investment costs estimate - construction of WWTP Bozica &Klisura (500 PE)

Small Compact WWTP Bozica&Klisura - capacity 500 PE						
	Description	**Civil works (€)**	**Equipment (€)**	**Piping (€)**	**TA (€)**	**Total costs (€)**
2.1	Investigation works and Detail Design				35,000	**35,000**
2.2	Construction works	40,000	120,000	7,500		**167,500**
2.3	Trial run and operation of 12 months				21,500	**21,500**
2	**WWTP Vrla - total**	**40,000**	**120,000**	**7,500**	**56,500**	**224,000**

5.2.2.6 Wastewater treatment – Alternative 2

Wastewater collection and treatment alternative 2 includes collection of wastewater in the Project area and its transfer to a single wastewater treatment facility located in the north of Vlasina dam, at the location of the WWTP Vlasina (i.e. current dysfunctional wastewater plant).

5.2.2.6.1 Central WWTP Vlasina (9.000 PE)

5.2.2.6.1.1 Process design criteria

In accordance with the demographic projections and related wastewater flows the following main design parameters have been adopted:

Table 5.54: Pollutant loading expressed in population equivalent for the end of Project period

Number of PE	Year 2006	Year 2035
Central WWTP Vlasina	4,658	9,233

The capacity of Central WWTP Vlasina is determined in accordance with the predicted quantity of the wastewater at the end of the project period, considering development of sewerage collection system and including infiltration. Typical hydraulic design loading of WWTP Vlasina is shown in the following table.

Table 5.55: Hydraulic load of the Central WWTP Vlasina at the end of the project period

Flow units	Average Flow	Maximum Daily Flow	Maximum Hourly Flow
(m^3/d)	2,782	3,758	
(m^3/h)	116	157	261
(l/s)	32.2	43.5	72.5

Table 5.56: Typical pollutant loading of the Central WWTP Vlasina (end of the project period, maximum daily flows)

Contaminants	Units	WWTP Vlasina
Suspended solids	kg/d	873
Biological oxygen demand (BOD_5)	kg/d	748
COD	kg/d	1497
Total nitrogen	kg/d	137
Total phosphorus	kg/d	25

5.2.2.6.1.2 Process description

The treatment process applied at a central WWTP is principally similar to previously described for two separate plants.

Due to higher hydraulic loads, larger units are required, and optimisation of construction costs could be achieved through the optimum number of treatment lines.

After preliminary treatment, consisting of coarse screening, lifting and fine screening wastewater enters the grit removal unit, vortex type, where water enters and exits the chamber tangentially. The rotating turbine maintains constant flow velocity. Adjustable blade promotes separation of organics from the grit. The grit settles by gravity into the hopper, from where it is removed by an airlift pump into the container.

As described in previous sections, the following, secondary treatment is comprised of two biaeration tanks with suspended growth activated sludge, followed by secondary settling facilities. Wastewater and sludge solids are firstly mixed and aerated in tanks, where microbial floc convert biodegradable, organic wastewater constituents and certain inorganic fractions into new cells and byproducts both of which is subsequently removed from the system by settling.

The following items are included:

- aeration tank
- settling tank (secondary sedimentation tank, also called final clarifier)
- pumps for returning the sludge (external recirculation)
- removal of the biological excess sludge

Treated water overflows into the collection channel, where UV disinfection is foreseen.

Excess sludge from settling tanks is concentrated by thickening and transferred by excess sludge pumps to aerated sludge tank, where the thickened sludge is aerated for an extended time. Air is supplied by means of diffusers.

Sludge, introduced continuously or intermittently, is retained in the reactor for varying periods of time.

Thickened sludge will be mechanically dewatered up to a minimum of approximately 20% dry solids and disposed to the landfill. Methods of mechanical dewatering include centrifuges, belt presses and plate presses. The first two are likely to be the most appropriate. Sand drying beds could be used where there is sufficient space available.

If more stringent conditions for effluent discharge would be required by the regulations, biological phosphorous and nitrogen removal would be achieved by additional zones placed ahead of activated sludge aeration tank. In the biological removal of phosphorous, the phosphorous in the wastewater is incorporated into cell biomass which subsequently is removed from the process as a result of sludge wasting. The reactor configuration utilized for phosphorous removal is comprised of an anaerobic tank having a hydraulic retention time 0.5 to 1 hour. The contents of the anaerobic tank are mixed to provide contact with the return activated sludge and influent wastewater.

An anoxic reactor follows the anaerobic reactor and precedes the aerobic reactor. The detention time in the anoxic zone is approximately 1 hour. The anoxic zone is deficient in dissolved oxygen, but chemically bound oxygen in the form of nitrate or nitrite is introduced by recycling nitrified mixed liquor from the aerobic section. Implementation of the anoxic zone minimizes the amount of nitrate fed to the anaerobic zone in the return activated sludge.

5.2.2.6.1.3 Description of plant process units

A hand-cleaned **coarse screen** with 50 mm clear spacing between bars is used ahead of lifting pumping station, to protect pumps, valves and pipelines from damage or clogging by rags and large objects. The quantity of screenings ranges between 4 – 11 l/1000 m^3 of wastewater flow.

Wastewater flows into the sump, where two working and one stand-by submersible pumps, each capacity of 37 l/s are installed.

Wastewater passes through a **fine screen**, with clear openings of 6 mm in order to remove various types of plastic material, un-decomposed food waste, and other material. Screening material is washed, compacted and conveyed by a screw conveyor into container. Two fine screens will be provided, one per each treatment line.

After fine screening, wastewater enters the grit removal unit, vortex type, where water enters and exits the chamber tangentially. The rotating turbine maintains constant flow velocity. Adjustable blade promotes separation of organics from the grit. The grit settles by gravity into the hopper, from where it is removed by an airlift pump into the container.
Basic design criteria are presented in the following table:

Table 5.57: Basic Design Criteria for the Preliminary Treatment

Manually cleaned Coarse Screen	
Type	bar screen
Clear openings, mm	50
Number of units	
Quantity of screenings, m^3/day	23
Lifting Pumping station, frequency controlled	
Type	centrifugal, submersible
No of working units	2
No of stand-by units	1
Capacity of the unit, l/s	37
Mechanically cleaned Fine Screen	
Clear openings, mm	6

Screening washing and compaction	included
Number of units	2
Quantity of screenings, m^3/day	169
Sand/grease removal	
Required volume, m^3	6
Number of units	2
Grit quantity, m^3/day	113

Wastewater then enters a **bioaeration tank**, where BOD removal takes place. At the inlet of the tank it is combined with sludge solids recycled from settling tanks, mixed and aerated.

Two bioaeration tanks, each of 1,600 m^3 are foreseen, providing hydraulic retention time of 20 hours, according to the maximum daily flow. Air is introduced through porous diffusers, placed at the bottom of the tank.

Process operates in a continuous flow mode. The mixed liquor from the aeration tank is transferred to a settling tank to allow gravity separation of the MLSS from the system.

Table 5.58: Basic Design Criteria for the Secondary Treatment

Extended aeration	
BOD_5 remain after preliminary treatment, mg/l	198
F/M, kgBOD5/kg MLVSS/d	0.07
MLSS (g/m^3)	4000
SRT, d	20
Number of units	2
Volume of the tank, m^3	1600
Depth of the tank, m	5
HRT, h	23
Recycling rate, %	100
Recirculation pumps capacity, l/s	22
Air requirements	
Oxygen demand, kgO_2/kg BOD_5 removed	2
Oxygen demand, kg/d	1351
Final Clarifier	
Overflow rate, $m^3/m^2/d$	32
Number of units	2

Surface per clarifier, m^2	98
Diameter, m	11.2
Side water depth, m	4
Volume of the clarifier, m^3	392
Excess sludge quantity, kg/d	270
Sludge volume, m^3/d	34
Number of excess sludge pumps	2
Capacity of excess sludge pumps, l/s	5

Clarified water passes through channel where disinfection by UV lamps takes place. Introduction of a tertiary treatment including nutrient removal could be considered at a later stage, subject to specific requirements that may arise in the future.

Additional volume of anaerobic section, if required, would be 160 m^3. The same volume is foreseen for anoxic section. Submersible mixers will be provided in both anaerobic and anoxic sections, in order to mix recycled flows with the influent wastewater.

Sludge line will be comprised of aerated sludge tank, where aerobic digestion of sludge takes place, followed by sludge conditioning and dewatering on chosen facility. The capacity of the filter press (or centrifuge) is determined by the quantity of sludge produced during 7 days operation of the plant to be treated within 8 hours working time 5 days per week.

5.2.2.6.1.4 Process diagram

Figure 5.37: WWTP Vlasina Central Wastewater Treatment Process Scheme

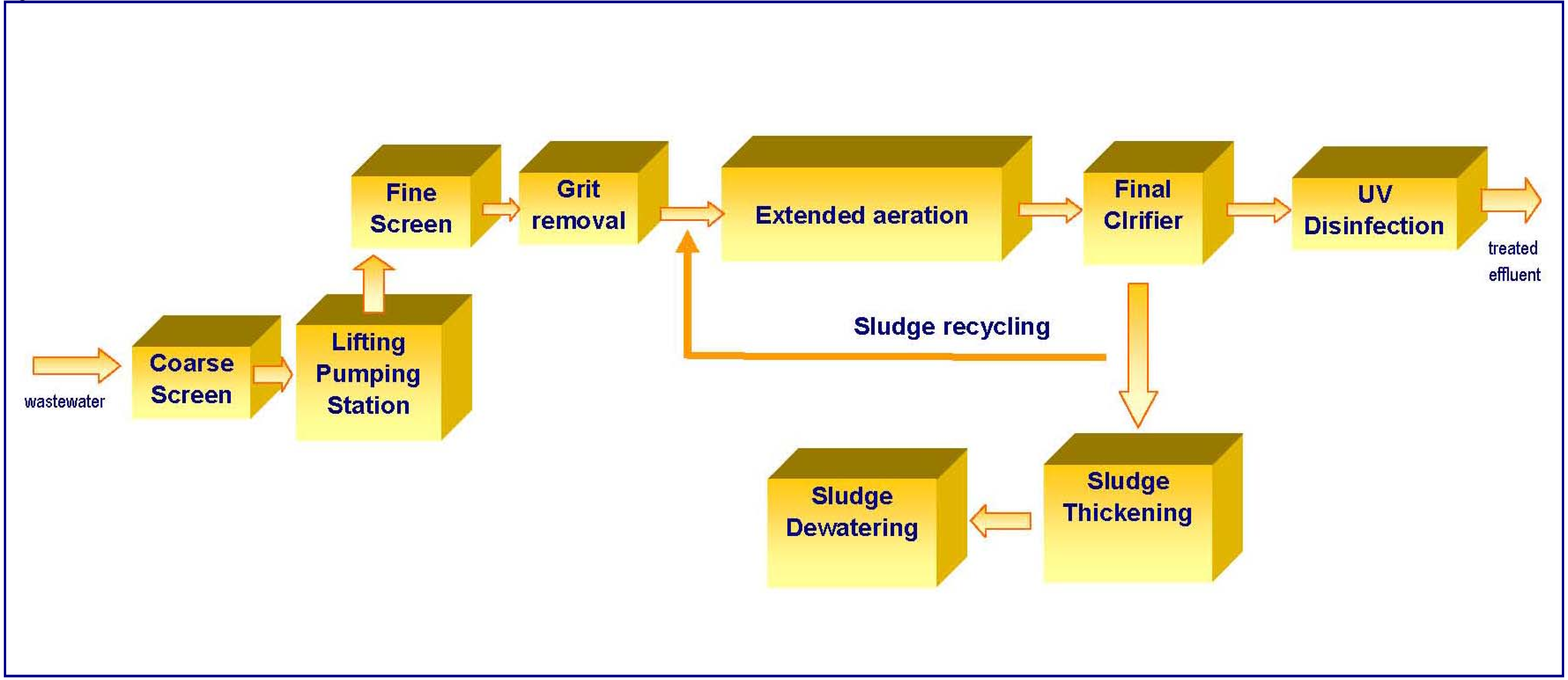


Figure 5.38: WWTP Vlasina Central Wastewater Treatment Process Scheme – Alternative including Nutrients removal

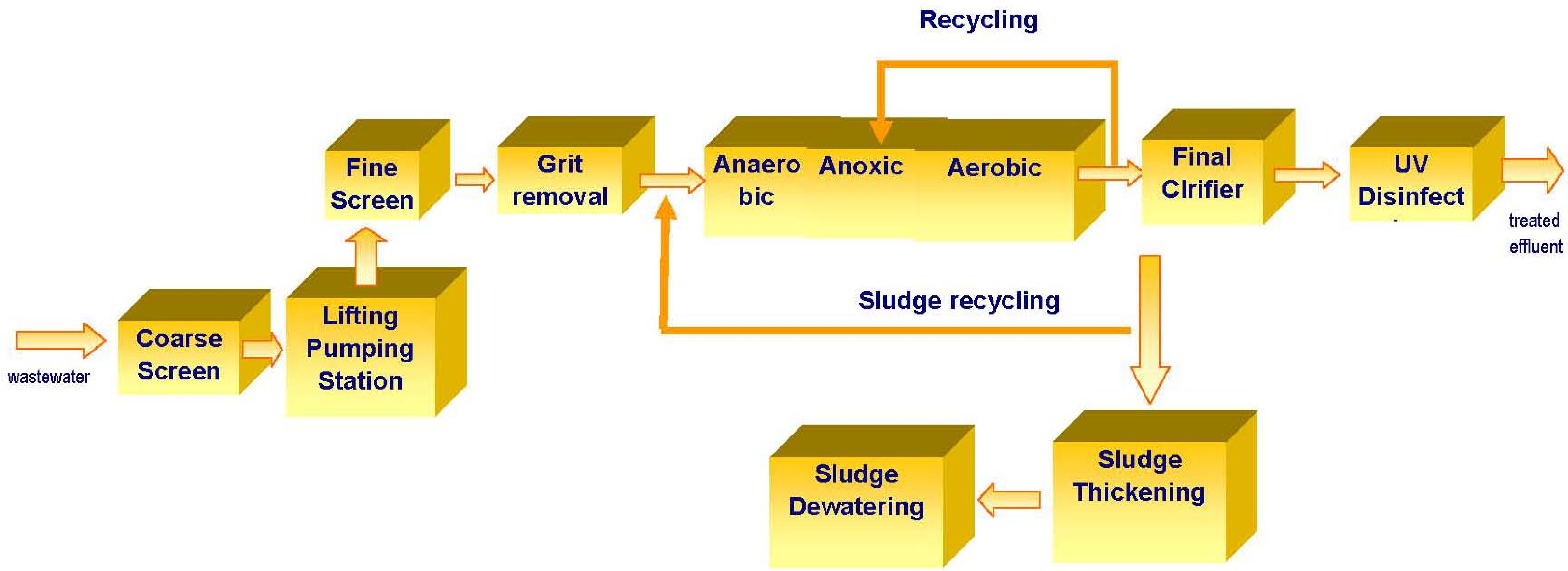
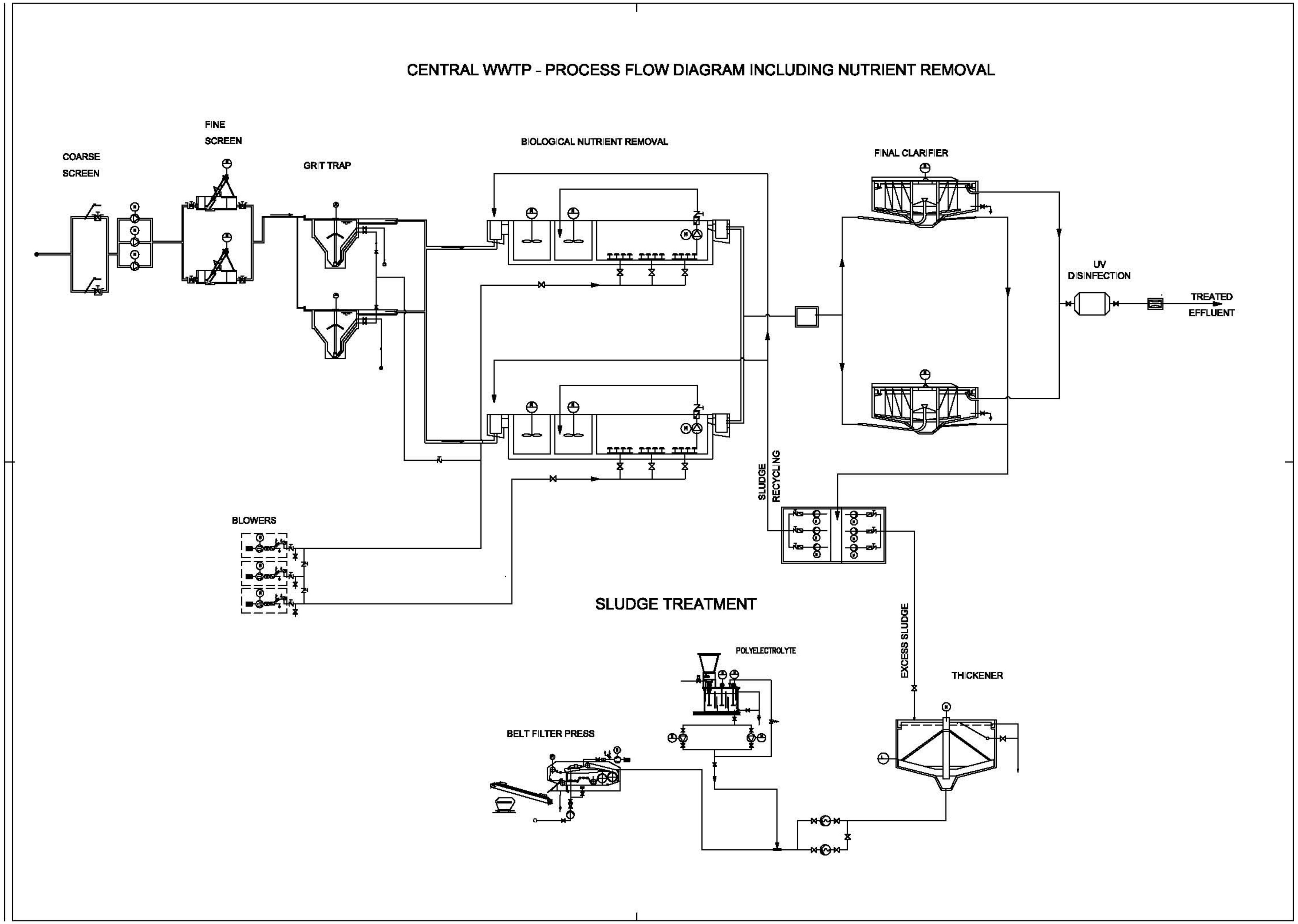


Figure 5.39: WWTP Vlasina Central Wastewater Treatment Process Flow Diagram



5.2.2.6.1.5 *Cost estimate*

This section presents an investment cost estimate for the described wastewater treatment plant Vlasina of the capacity 6.000 PE (Phase I) + 3.000 (Phase II).

Apart from the investment cost of construction works and necessary equipment, it is also envisaged to prepare a corresponding detail design of the facility with appurtenant investigations, and to conduct 12-month trial run and operation of the plant, including training of the local staff. These activities are to be funded through suitable technical assistance programme.

These investment costs have been generated on the basis of typical unit costs for this type and size of the facility, and described level of treatment (primary + secondary wastewater treatment).

Table 5.59: Investment costs estimate - WWTP Vlasina (6,000+3,000 PE)

1 WWTP Vlasina - PHASE 1 - capacity 6.000 PE						
	Description	**Civil works (€)**	**Equipment (€)**	**Piping (€)**	**TA (€)**	**Total costs (€)**
1.1	Investigation works and Detail Design				112,500	**112,500**
1.2	Construction of WWTP Vlasina	954,000	556,500	79,500		**1,590,000**
1.3	Trial run and operation of 12 months				79,500	**79,500**
1	**WWTP Vlasina – Phase 1**	**954,000**	**556,500**	**79,500**	**192,000**	**1,782,000**
2 WWTP Vlasina - PHASE 2 - additional capacity 3.000 PE						
	Description	**Civil works (€)**	**Equipment (€)**	**Piping (€)**	**TA (€)**	**Total costs (€)**
2.1	Investigation works and Detail Design				37,500	**37,500**
2.2	Construction of WWTP Vlasina	318,000	185,500	26,500		**530,000**
2.3	Trial run and operation of 12 months				26,500	**26,500**
2	**WWTP Vlasina – Phase 2**	**318,000**	**185,500**	**26,500**	**64,000**	**594,000**
3 WWTP Vlasina - total capacity 6.000+3.000 PE						
	Description	**Civil works (€)**	**Equipment (€)**	**Piping (€)**	**TA (€)**	**Total costs (€)**
3.1	Investigation works and Detail Design				150,000	**150,000**
3.2	Construction of WWTP Vlasina	1,272,000	742,000	106,000		**2,120,000**
3.3	Trial run and operation of 12 months				106,000	**106,000**
3	**WWTP Vlasina - total**	**1,272,000**	**742,000**	**106,000**	**256,000**	**2,376,000**

5.3 Stormwater drainage system

5.3.1 Introduction

This section elaborates on the proposed stormwater drainage system that is planned to collect and accommodate stormwater runoff in the Project Area, as a part of the infrastructural development for tourism at Vlasina Lake. The proposed system, design criteria, methodology and cost estimates, as described hereinafter, are based on the Technical report on preliminary design of sewerage system for wastewater collection, drainage and treatment in Vlasina region, Book B – Stormwater, prepared by the Faculty of Civil Engineering, University of Nis in 2007 (reference documentation 16).

5.3.2 Background

In the planning of stormwater drainage system earlier proposals and documents were considered and taken into account. Spatial plan of the special purpose region of Vlasina – SPSPR ("Official Gazette of RS" No.47/2003) prepared by Energoprojekt, Town Planning and Architecture S.A. in 2003, envisages a number of activities in the period until 2021, directed towards achievement of sustainable spatial development, tourism development, protection of nature and natural resources in the Vlasina region, as well as safeguarding of this region as a natural source of water supply in compliance with the Water Management Scheme of RS.

In broader Vlasina region in the past period, improper construction and other out-of-plan activities have taken place, producing a considerable effect on natural resources of this region. Particularly adverse environmental impact was produced by cutting of coniferous forests and clearing of beech forest, thus producing potential risk of erosion process development. Despite these adverse effects, natural resources are the greatest value and opportunity for development in this region, but they require application of adequate protection and development measures.

In order to protect the region covered by SPSPR Vlasina, there are envisaged activities in the domain of Water Management infrastructure, i.e. protection and use of water, protection against water, and development of water and water courses.

The centre of the area covered by SPSPR Vlasina is the Vlasina Lake, spreading over undulated alluvial plain, between watersheds of rivers Vlasina, Vrla, Bojnica and Jerma. Maximum backwater level of this artificial storage reservoir is **Z=1212.80 masl**, and the reservoir was created during construction of the dam between 1945-1949 for the requirements of hydropower system Vrla I – IV, as well as construction of storage system Lisina, from which the water is pumped over to the Vlasina Lake.

Mountainous area around the Vlasina Lake has all the properties of mountain climate with cold winters and fresh summers. Creation of the Vlasina Lake had positive effects on temperature amplitudes. Annual precipitation distribution is uniform with average precipitation exceeding the average values for Serbia (**724mm annually**). Maximum precipitation is in May and June. Winds are frequent, especially the west winds, at the beginning of summer and in spring, whereas the east winds blow in winter. The season with the most windless days is summer.

Tasks for development, utilization and protection of water in SPSPR Vlasina are defined according to the degree of water pollution risk in the Vlasina Lake and entire region, through three zones:

- ***a.*** ***Zone of immediate protection of water supply sources, including:*** water courses in the watershed 50m to the left and right of water course banks, and for reservoirs and area 100m wide downstream of the dam and a 50m wide belt along reservoir edge, compared to the normal backwater level. Zone of immediate protection is located within the closer protection zone.
- ***b.*** ***Closer zone of sanitary protection,*** covering the area around the water courses and Vlasina Lake 500m wide compared to normal backwater level.
- ***c.*** ***Broader zone of sanitary protection,*** covering the area which belongs to hydro-geological watershed of the Vlasina Lake.

In SPSPR Vlasina, in the closer zone of sanitary protection of Vlasina Lake area, there are envisaged conditions and restrictions for:

1. *Utilization and development of agricultural and forest land;* the Vlasina region has a tradition of various forms of agricultural production, primarily cattle breeding. Broader belt around Vlasina Lake is covered with meadows and pastures, and partially forest. Sub-mountainous climate of the Vlasina region enables growing of spring grainfalls and potato as well as cattle breeding (primarily horses, sheep, goats).
2. *Construction development of housing buildings, resting houses, tourist, sports and recreational and other facilities.* This construction is envisaged in three spatial units:
   a. Vlasina Rid zone,
   b. Vlasina Stojkoviceva zone,
   c. Vlasina Okruglica zone.

3. *Road infrastructure;* SPSPR envisages removal of road traffic of main and regional rank from the close zone of sanitary protection. By relocation of regional road R 122, on the west side of Vlasina Lake, the existing road Surdulica-Crna Trava shall have local character and rank of tourist traffic line around the Vlasina Lake. For connection of this region to regional road infrastructure, a new traffic line shall be constructed, located out of the close zone of sanitary protection. Also, it is envisaged to relocate a section of main road M1-13 which passes south of the lake through the close zone of protection, thus providing an area about 500m wide along the reservoir edge.

1. *Protection against floods, erosion and torrents*, implying:
   a. protection against water and wind erosion in order to prevent loss of soil;
   b. protection against floods, prevention of mechanical backfilling of reservoirs and mechanical and chemical pollution of water courses;

2. *Development and training of water courses, including:*
   a. training of water courses in construction zones, towards protection against floods and erosion of riparian area, preservation of ambience of settlements and provision of access to water course and horticultural development of riparian area;
   b. control of outflows from pipelines which choke the Vlasina Lake with mud;
   c. remedy of local occurrence of riparian erosion and cleaning of riverbeds from sediments, waste, etc.
   d. removal of existing structures on water courses which do not meet the water management conditions;
   e. repairs, establishment and hydraulic development of profiles for measurements of low water levels
   f. reconstruction of existing torrent partitions and cascades which are more than 0.6m high and represent a problem for fish migration

3. *Accommodation of stormwater discharge,* implying reception and accumulation of water from traffic lines and critical watershed areas – passive flood control, as well as preservation of class I water courses and reservoirs against pollution from endangered areas.

5.3.3 Overview and Review of Solution for Collection of Stormwater in Spatial Plan of Special Purpose Region Vlasina

In the technical document *Spatial Plan of Special Purpose Region of Vlasina*, towards preservation of water quality and protection of Vlasina Lake against pollution by stormwater management (so called small rainfalls), it is envisaged to construct:

- peripheral channels, mainly along traffic lines;
- drain channels on traffic lines themselves;
- collection of stormwater in retention basins;
- transport of stormwater from the retention basins by means of sewage pumping stations towards wastewater treatment facilities.

This proposal however did not fully address two very important issues; settled sediments and oils in the retention basins. Also, this proposal would imply considerable investments in construction of pipelines and pumping stations, as well as permanent costs of pumping.

There is a large number of permanent and occasional water courses, which inflow the Vlasina Lake. Occasional flows appear in the seasons with heavier precipitation.

Apart from permanent and occasional water courses, artificial channels constructed for recharging of Vlasina Lake also inflow into it. On the east side, those are the channels of Lisina and Bozica, and on the north side in the dam zone, the channels of Jerma, Strvna and Cemernik. In the zone of Lisina channel confluence into the Vlasina Lake, Figure 1, considerable bank erosion has been noticed. *Protection of this zone of Vlasina Lake out of the scope of this Project.*

Figure 5.40: Erosion of Vlasina Lake bank in the Lisina channel confluence zone

5.3.4 Proposed Stormwater Collection Concept

5.3.4.1 Concept Outline

On the slopes of Vlasina Lake, on the east bank, there are presently mostly meadows – grass areas, Figure 2, and on the west and south side of the lake, there are settlements of Vlasina Rid and Okruglica, as well as traffic infrastructure. With implementation of SPSPR Vlasina, the east side of the lake is intended for construction of important civil structures for development of housing, tourism and agricultural, as well as local traffic infrastructure.

Figure 5.41: Current View over east slopes of Vlasina Lake

Generally, in the Vlasina Lake region, wind and water erosion are not particularly striking. Turbidity of the lake has not been increased, but there is sediment – mud in the zones where water courses inflow the Vlasina Lake. Erosion occurrence results from undeveloped access roads in existing construction zones, on agricultural lands and in riverbeds.

Figure 5.42: Deposited fine sediments at the confluence of Cvetkova River into the Vlasina Lake

Preliminary design in compliance with Terms of Reference and SPSPR Vlasina, covers the necessary works for protection of Vlasina Lake against stormwater discharge generated in the immediate zone of sanitary protection. Stormwater discharge that is not included in the scheme shall be introduced through open conduits and drain channels into local water courses and Vlasina Lake.

Polluted stormwater (so called small rainfalls) from slopes, wherefrom the water inflows directly the Vlasina Lake, as well as from areas in traffic zones, shall be collected in retention basins before release, and shall be treated by a separate system to the level required for preservation of prescribed class I recipient.

This solution does not include water courses which inflow the Vlasina Lake, as conditions from the SPSPR envisage their separate complex development and erosion control in the watershed.

5.3.4.2 Peripheral Channels

For protection of Vlasina Lake against sedimentation and pollution with atmospheric wastewater (small rainfalls), the Preliminary Design stipulates **construction of two protective channels around the lake**: drain channel along new traffic ring and **peripheral and slope channels and drain-falls** along the existing road Vlasina Okruglica – Vlasina Rid and Vlasina Okruglica – Vlasina Stojkoviceva, as well as immediately under the existing and perspective construction zones on the east side of the lake.

Total length of peripheral channels on east, south and west bank of the lake is L=26.82km. On the east side of Vlasina Lake, channels only partially, up to the Lisina channel confluence, follow the existing traffic line M1-13, Vlasina Okruglica – Vlasina Stojkoviceva, and their position in other sections is defined by topography of slopes and altitude of existing and perspective construction zones.

On the west and south side of the lake, altitude of peripheral channels depends on grade line of existing traffic line R 122. Channels are located at the upper side of the road, and retention basins at the lower side of the road, facing the lake. Stormwater from traffic lines is accepted in drain channels and conveyed to peripheral channels and retentions at the lowest points of traffic line.

For clearer presentation of the Vlasina Lake protection against stormwater pollution, layout of peripheral channels is presented in layout plan of the Vlasina Lake region, in scale 1:25000, graphic enclosure 1, and their position compared to the existing and planned construction and tourist zones according to the SPSPR.

Longitudinal terrain grade, length and other geometrical features of peripheral channels are presented in the following table.

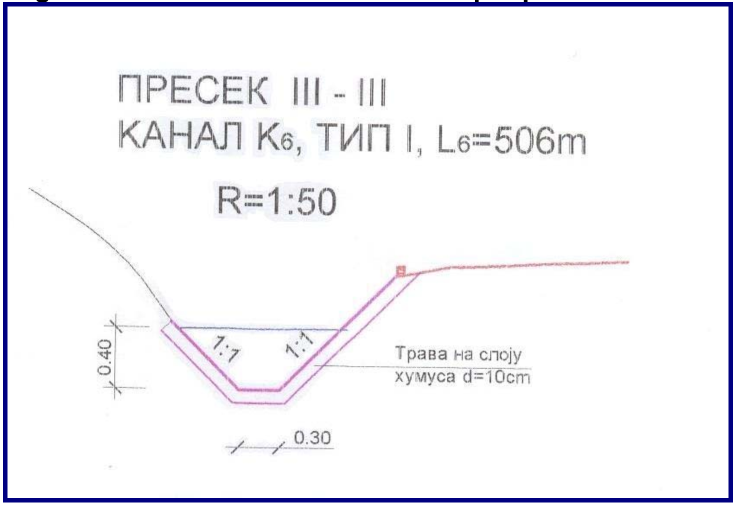
Table 5.60: Features of peripheral canals

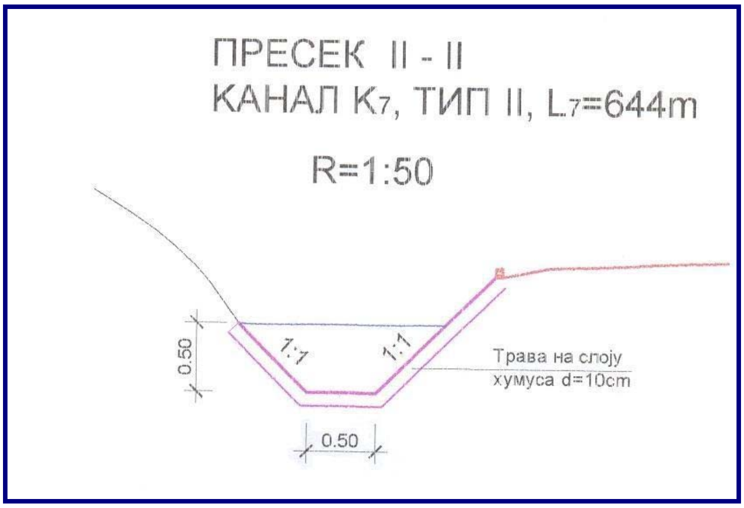
Canal label	L(m)	I (%)	Canal catchment area (ha)	Belonging retention basin
K1	430	0.70	11.8	**R1**
K2	688	1.31	22.1	
K3	249	3.82	11.1	**R2**
K3a	152	2.96	7.8	
K4	181	2.49	34.4	**R3**
K5	294	2.89	12.4	
K6	506	1.19	10.2	**R4**

K7	644	1.40	26.6	
K8	1312	0.99	39.5	**R5**
K9	1276	1.57	38.4	**R5a**
K10	868	0.86	32.1	
K11	1020	1.23	10.1	**R6**
K12	2262	2.63	13.1	
K13	1327	3.62	8.4	**R7**
K14	964	2.07	37.7	
K15	1075	4.56	20.9	**R8**
K16	425	2.82	17.9	
K17	1435	1.67	46.6	**R9a**
K18	838	1.79	34.5	**R9b**
K19	1280	1.56	24.8	**R10**
K20	1248	1.28	32.9	
K21	606	1.32	13.3	**R11**
K22	1080	1.48	34.1	
K23	1006	1.99	23.9	**R12**
K24	480	4.17	22.7	
K25	938	1.28	11.3	**R13**
K26	1921	0.62	44.1	
K27	1252	0.80	25.9	**R14**
K28	1058	1.89	28.5	
			697.1	

Peripheral channels – K have a trapeze cross section, and only sporadically when a channel intersects a traffic line and where the conditions require it, closed conduit culverts are envisaged. Geometrical features of channels are defined according to hydrological and hydraulic calculation. Generally, two types of channels are envisaged, as shown in the following figure.

Figure 5.43: Geometrical features of peripheral channels





Designed longitudinal grade of peripheral channels – J_0, ranged from 0.5% to 1.5%, and it is determined in order to provide subcritical flow regime in channels. Longitudinal grade of each particular channel shall be determined depending on topographic terrain conditions. At such sections where topographic features require so, one or more drop structures up to 0.5-0.7m high shall be constructed in order to ensure the designed longitudinal grade.

Detailed hydraulic analysis and identification of geometrical features for each channel shall be possible after verification of Hydrologic Analysis and definition of referent discharges, i.e. after obtaining the Opinion of the Republic Hydro-meteorological Institute and Water Management Conditions.

In the said Preliminary Design, the hydraulic analyses include capacity of channels.

The channels are earthen and should be executed by excavation in terrain, slopes and bottom of the channels should be secured by planting of adequate grass species on a humus layer at least 10cm thick. In the zones of culverts and cascades, the channels should be secured with a lining made of arrayed crushed stone in cement mortar with thickness of d=25-30cm. The lining should be executed on a layer of fine-grained gravel, of thickness d=10cm.

5.3.4.3 Retention Basins

Stormwater is conveyed by channels to retention basins intended for sedimentation of suspended matters and bed load and removal of oils that might be draining from the roads.

No organized measurements of sediment transported in water courses inflowing the Vlasina Lake have been conducted so far. During prospecting of terrain in confluence zones, it was observed that there is a considerable amount of very fine sediment – sludge. It is estimated that after the dam construction and forming of Vlasina reservoir, i.e. from 1950 to this day, it has been backfilled with about 0.5 to 2.0 m.

Retention basins are impermeable mini reservoirs, to be located next to existing water courses. They shall be formed in compliance with natural terrain configuration and closed by impermeable embankments of required height. Position of retention basins, and size of appurtenant watersheds, from which stormwater is collected to particular retention basins, are presented in following table.

Table 5.61: Main features of retention basins

Canal label	Canal catchment area (ha)	Belonging retention basin	Retention basin catchment area (ha)
K1	11.8	R1	33.9
K2	22.1		
K3	11.1	R2	18.9
K3a	7.8		
K4	34.4	R3	46.8
K5	12.4		
K6	10.2	R4	36.8
K7	26.6		
K8	39.5	R5	39.5
K9	38.4	R5a	70.5
K10	32.1		
K11	10.1	R6	23.2
K12	13.1		
K13	8.4	R7	46.1
K14	37.7		
K15	20.9	R8	38.9
K16	17.9		
K17	46.6	R9a	46.6
K18	34.5	R9b	35.0
K19	24.8	R10	57.6
K20	32.9		
K21	13.3	R11	47.4
K22	34.1		
K23	23.9	R12	46.6
K24	22.7		
K25	11.3	R13	55.4
K26	44.1		
K27	25.9	R14	54.4
K28	28.5		
	697.1		

Location of the retention basins is next to natural water courses. Size, water discharge and cleaning have been defined in the Preliminary design, based on hydrological and psalmological calculations, also taking into account:

- referent precipitation
- rainwater runoff from appurtenant watersheds – channels draining to retention basins;
- estimated quantity of sediment.

After being retained for one hour and after sedimentation of suspended matters and bed load, water from the retention basins is released through culverts into the existing water courses and it inflows the lake. By controlled release of water from the retention basins, the possibility of release of polluted water and oil (reaching the retention basins from traffic lines) into the Vlasina Lake shall be eliminated.

Water release from retention basins into the Vlasina Lake is envisaged by a culvert made of concrete sewerage pipes of diameter 500-600mm. In front of the culvert pipe, it is envisaged to place a fitting – concrete T of diameter 800-1000mm. This fitting acts as an oil separator. It is closed by a chequer plate cover. On the lower part of the fitting, it shall be perforated with six openings D=200mm. The fitting shall be placed on concrete base MB20, dimensions 2.2x2.2x0.6m. In the lower zone of the fitting, around the opening, a geo-textile lining surrounded with rubble stone d=15-30cm should be placed, of minimum volume of $V=3.0m^3$.

According to the estimated quantity of sediment and provided space in retention basins for collection of sediment, occasional mechanical cleaning of retention basins shall be provided (control once a year and cleaning once in three years). However, exact maintenance and cleaning intervals shall be determined during the system operation, based on exact operational status.

Typical detail of retention basins is shown in the following figure.

Figure 5.44: Typical retention basin – layout

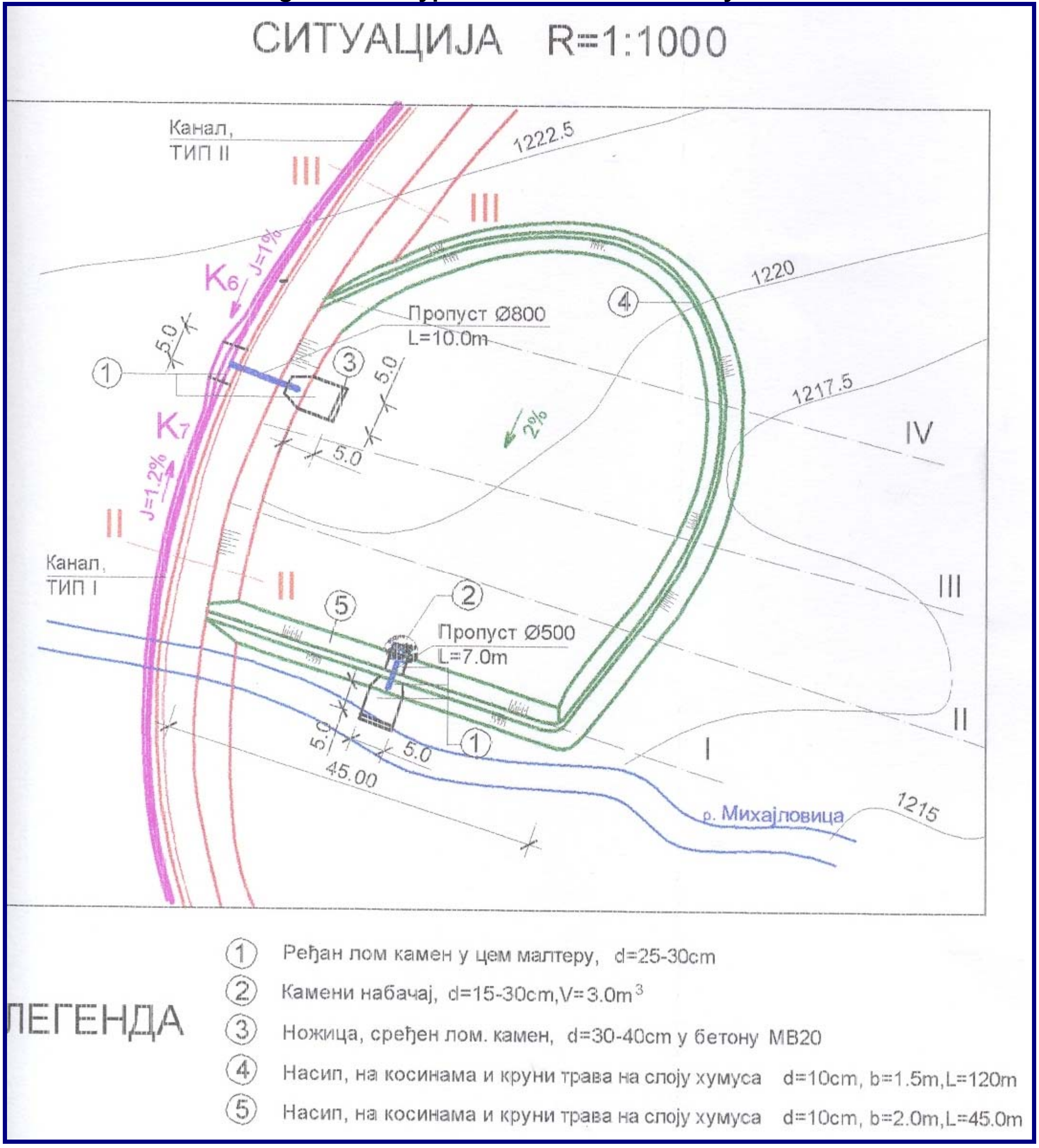
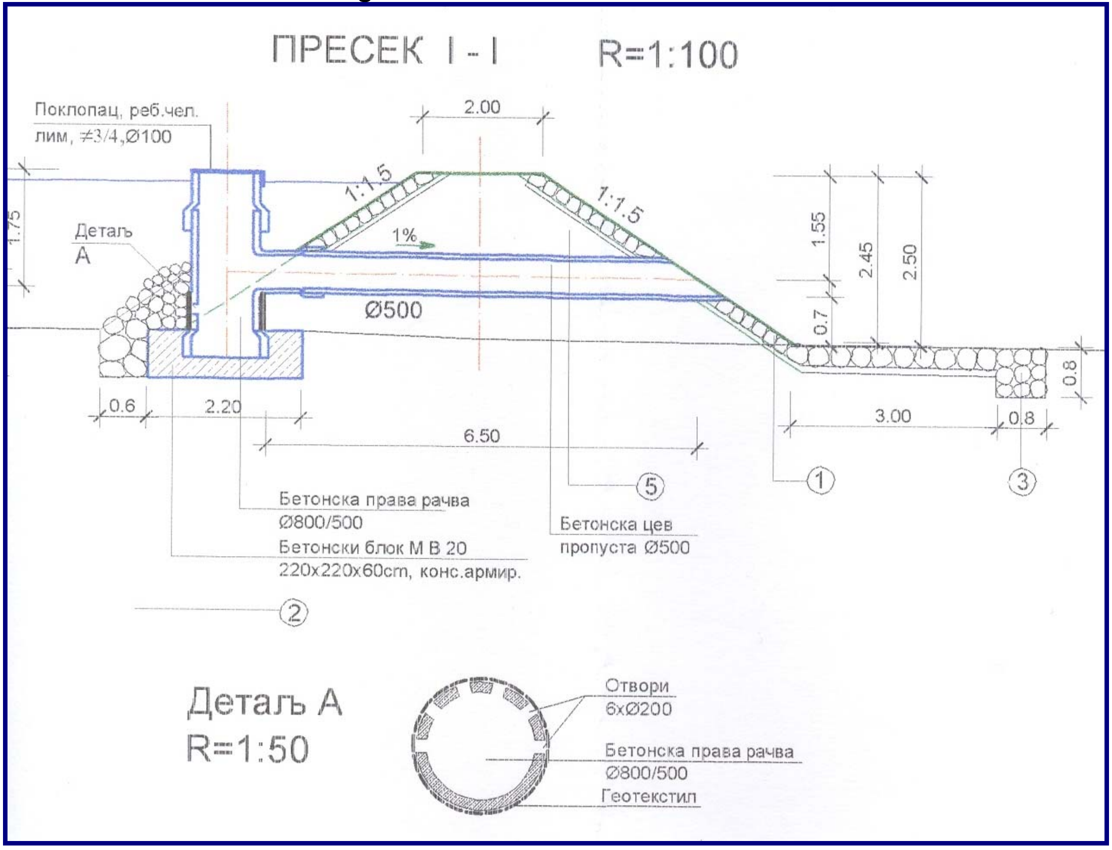


Figure 5.45: Retention basin outlet



5.3.5 Basic Calculations – an Overview

This section presents an overview of basic calculations used for analysis and sizing of the concerned stormwater drainage system.

5.3.5.1 Hydrological Calculations

The subject of this report is calculation of hydrological values relevant for:

1. Assessment of water courses impact on planned wastewater treatment plants (WWTP) Vlasina and Vrla;
2. Assessment of impact of planned WWTP Vlasina and Vrla on recipients;
3. Dimensioning of structures for accommodating and evacuation of stormwater runoff including channels (along existing road Vlasina Rid – Vlasina Okruglica – Vlasina Stojkoviceva) and sloped channels in the zone of existing and planned settlements (Vlasina Stojkoviceva), as well as planned retention basins.

The following figure presents locations of relevant precipitation stations and catchment areas of the WWTP-s, whereas position of the WWTP-s and channels are presented in the corresponding general layout drawing.

Figure 5.46: Position of precipitation stations and catchment areas of WWTP

Analysis of Precipitation

In compliance with **Hydro-meteorological Conditions**, and letter – **Amendment to Technical Documents** – item 4 (Hydrology Sector, RHMI of Serbia – letter number 92-I-1-5/2006-22 dated 11/01/2006) data from **GMS Vranje** were used for calculation of **short duration rainfalls**, and data from precipitation stations **Vlasina Rid** and **Stojkovica Mahala** for analysis of **maximum annual precipitation**.

<u>**Processing of Maximum Annual Precipitation Sequences**</u>

The results of basic statistic processing of maximum annual precipitation sequences are presented in table 5.62:

Table 5.62: Overview of basic statistical parameters related to maximum daily precipitation for Vlasina region

	Vlasina Rid	Stojkovica Mahala	Vlasina	Okruglica
Number of data	51	48	46	40
Maximum	80.6	98.4	88.7	85.2
Minimum	16.5	24.5	21.5	12.6
Mean value	38.2	44.9	44.8	44.0
Standard deviation	13.1	16.0	15.4	16.7
Coefficient of variation	0.34	0.36	0.34	0.38
Coefficient of asymmetry	0.91	1.23	0.94	0.67

For variation array of observed maximum annual precipitation and smoothening of the obtained empirical function of distribution, was performed with compromise probability by the **Weibull's** formula. Regarding theoretical probability of distribution, the following were considered: **normal, log-normal, Gumbel's, Pearson type 3 and Log-Pearson type 3** probability distribution.

Based on the statistical analysis conducted using available precipitation data the adopted referent theoretical function of probability distribution is the **Logarithm-Pearson type-3 distribution.** Theoretical values of the maximum annual precipitation for return periods of 50 and 100 years (which shall be taken into account when determining the characteristic discharges at WWTP-s), as well as values for T=5 years (for which the stormwater drainage facilities shall be dimensioned) are presented in the following table 5.63.

Table 5.63: Adopted theoretical values Pmax,dn T(mm) for various return periods, for entire Vlasina lake region

T (years)	5	50	100
Vlasina Rid	48.1	71.2	77.6
Stojkovica Mahala	55.4	90.5	102.3
Vlasina	56.0	85.2	93.9
Okruglica	55.4	90.5	102.3
Adopted	**55**	**90**	**105**

Using the regional reduction curve of rain layer Ψ(t) for GMS Vranje, rainfalls of various duration were obtained for return periods T=5, 50 and 100 years, for the Vlasina region, as presented in Table 5.64.

Due to large difference in altitude of Vlasina region (1190 masl) and Vranje (458 masl), representative character of GMS Vranje for Vlasina region was checked, by linear correlation of series of average monthly precipitation for standard normal period (1961-1990). The coefficient of linear correlation R=0.893 was obtained, which confirmed compliance of the use of precipitation data from GMS Vranje for Vlasina region.

Table 5.64: Design rainfalls of various durations PT(τ) (mm) for return periods T=2, 5, 50 and 100 years for Vlasina region

τ	(min)	10	20	30	60	90	150	360	720	1440	Pmax,dn $_T$(mm)
	(h)	0.17	0.33	0.5	1	1.5	2.5	6	12	24	
T (years)	**100**	47.4	58.2	63.6	71.7	74.4	86.6	111	123	131	105
	50	36.7	46.8	53.1	58.1	60.7	72.1	92.3	104	111	90
	5	16.6	24.0	26.3	30.9	33.2	38.9	49.2	57.2	66.3	55
	2	10.5	14.5	17.1	19.2	21.7	22.8	23.5	30.8	36.8	41

Calculation of Characteristic Flows of Recipients for Outlet Profiles

Physical-geographical and morphometric features of the belonging catchments of WWTP-s are presented in the following table.

Identification of Effective Rainfalls

Taking into account relatively small surface areas and character of catchments, both for WWTP-s and for belonging catchment areas of stormwater drainage channels, the Soil Conservation Service method (SCS method) was adopted for calculation of effective precipitation.

The value of hydrological soil cover complex number (CN) for the condition of average humidity, was determined depending on planned purpose of areas from the Spatial Plan (SP), in all cases when it differed from existing status.

Physical-geographical and morphometric features of the WWTP-s catchments for outlet profiles are shown in the following table.

Table 5.65: Physical-geographical morphometric features of the WWTP-s catchments

Feature	WWTP Vlasina	WWTP Vrla
Catchment surface area (km^2)	0.23	1.35
Catchment length (km)	0.95	1.41
Density of water courses network (km/km^2)	6.58	3.07
Maximal altitude (masl)	1220	1282
Minimal altitude (masl)	1189	975
Total head (m)	31	307
Length of water course (km)	0.78	1.37
Hydraulic length of catchment (km)	1.17	1.69
Spring altitude (masl)	1215	1258
Average slope (%)	2.0	19.9
Average adjusted slope (%)	2.7	21.5
Distance from the centre point of the catchment to the outlet point (km)	365	735

Effective rainfalls are calculated by the formula of SCS method $Pe = \frac{(P-0.2d)^2}{P+0.8d}$, wherein P-gross rainfall (mm) ($P_T(\tau)$=$P_{max,dn,T}$ •Ψ(τ)) presented in table 5.66. Values of calculated effective precipitation for rainfalls of various durations are presented in the following table.

Table 5.66: Effective rainfall of various durations Pe, T(t) (mm) for appurtenant catchment areas of WWTP

WWTP	τ	(min)	10	20	30	60	90	150	360	720	1440	Pmax,dn
		(h)	0.17	0.33	0.5	1	1.5	2.5	6	12	24	$_T$(mm)
Vlasina	T (years)	100	1.7	4.3	5.9	8.8	9.9	15.1	27.9	35.1	40.2	24.5
		50	0.2	1.6	2.9	4.3	5.0	8.9	17.9	23.8	28	16.8
Vrla	T (years)	100	3.1	6.5	8.6	12.1	13.3	19.5	33.9	41.9	47.4	30.1
		50	0.8	2.9	4.8	6.5	7.4	12.2	22.6	29.3	34.0	21.3

High flow analysis

According to the factual status, and in compliance with the **Hydro-meteorological Conditions**, and letter – **Amendments to Technical Documents** – item 4. (Hydrology sector, RHMI of Serbia – letter no. 92-I-1-5/2006-22 dated 11/01/2006), the calculations made use of methods for high flow analysis on catchments without hydrological observations. Maximum discharge was estimated:

- based on empirical reduction formula (Graphical SCS - TR-55) method) and
- according to the theory of synthetic unit hydrograph

Based on the calculations using the graphical SCS – TR-55 method, the maximum calculated discharges for WWTP 1 and WWTP 2 sections are respectively:

- **WWTP 1 (Vlasina) - Q_{max} (T=100 years) = 0.92 m^3/s**
- **WWTP 2 (Vrla) - Q_{max} (T=100 years) = 7.19 m^3/s**

On the other hand, based on the Synthetic Unit Hydrograph method, the maximum discharges were obtained for a T=30min rainfall:

- **WWTP 1 (Vlasina) - Q_{max} (T=100 years) = 0.55 m^3/s**
- **WWTP 2 (Vrla) - Q_{max} (T=100 years) = 3.82 m^3/s**

Low and medium flows

With regard to low and medium flows, the following values have been estimated:

For WWTP Vlasina, flows are directly affected by operational regime of the reservoir Vlasina, i.e. current discharges over the dam. Therefore, minimal flows correspond to the prescribed minimal discharges from the reservoir.

For WWTP Vrla section, the average flow is estimated at app. 20 l/s.

5.3.5.2 Hydraulic Analysis

Channels

For peripheral channels, two types of channels with different geometrical characteristics are selected, as shown earlier. Geometrical characteristics of channels are selected so that the flow regime in channels is subcritical for selected design discharge. Longitudinal grade of the channels is J=1-1.5%.

For defined hydrological, hydraulic and geometrical parameters of the channels, normal water depth in channel – h, may be analytically determined by Chezy-Manning equation, and the velocity of water flow by Chezy's expression:

$$v = C \cdot \sqrt{RJ}$$

Chezi's velocity coefficient – C includes hydrodynamic factors and hydraulic resistances of flow in channel. Velocity coefficient – C may be identified by various expressions, but most often, when the flow regime in channel is calm, the Manning's expression is used:

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

Manning's coefficient in channel – n, due to prismatic shape of channel and anticipated characteristics of channel, lining of slopes in low grass, was adopted based on data from the literature:

$$n=0.030\ m^{-1/3}s$$

Discharge capacity of a channel, according to the Chezy-Manning's expression, is:

$$Q = \frac{1}{n} A R^{\frac{2}{3}} J^{\frac{1}{2}}$$

Detail results of channel discharge capacity calculation, selection of type and geometrical features of channels, are presented in the aforesaid Preliminary Design.

Culverts

Retention Basin Inlet Culverts

Inlet culverts are sized according to referent precipitation and total runoff in channels inflowing the retention basin.

Detailed presentation is given only for a selected retention basin – R4. This basin collects water by channels K6 and K7. Channel properties are the following:

Channel length:

- K6 – L_6 = 506m;
- K7 – L_7 = 644m;

Surface area of appurtenant catchments of channels K6 and K7 is:

- A = 36.8 ha

Runoff coefficients:

- From traffic lines: 0.95;
- From other catchment: 0.30;

According to hydrologic analysis, referent precipitation for sizing of the culvert is the one with return period T=5 years and duration t=2h. Therefore, the resulting referent discharge for this culvert is 0.90 m^3/s. Selected size of the concerned culvert is D=0.50m, culvert length is 7.0m, and its grade is 1 to 3%.

Retention basin volume has been calculated taking into account minimum retention period of 1h and necessary provision for sediment disposal in the basin.

Road Culverts

Peripheral channels are generally on the upper side of the roads, and retention basins which receive stormwater from channels are on the lower side of the roads. Therefore, a culvert constructed of concrete sewer pipes must be constructed through the road body. In front of culvert, a channel for water reception should be formed – expanded.

General geometry of selected culvert is as follows:

▪ Concrete pipe, diameter	Ø 800mm;
▪ Longitudinal grade	J=1%
▪ Culvert length	L=10.00m;
▪ Water depth in front of culvert	H=0.6m;

Flow regime in the culvert corresponds to free-surface flow.

Based on presented hydraulic analyses for the selected retention basin and appurtenant channels, required capacity of all designed retention basins are given in the following table. The layout of the proposed stormwater drainage system is shown on the enclosed general layout map.

Table 5.67: Main features of retention basins

Canal label	Canal type	Canal discharge (l/s)	Belonging retention basin	Retention basin volume (m^3)
K1	1	11.8	**R1**	2550
K2	2	22.1		
K3	1	11.1	**R2**	1450
K3a	1	7.8		
K4	2	34.4	**R3**	2565
K5	1	12.4		
K6	1	10.2	**R4**	1125
K7	2	26.6		
K8	2	39.5	**R5**	1630
K9	2	38.4	**R5a**	2705
K10	2	32.1		
K11	1	10.1	**R6**	1455
K12	2	13.1		
K13	1	8.4	**R7**	1120
K14	2	37.7		
K15	2	20.9	**R8**	1585
K16	2	17.9		
K17	2	46.6	**R9a**	2835
K18	2	34.5	**R9b**	2295
K19	2	24.8	**R10**	2310
K20	2	32.9		
K21	1	13.3	**R11**	620
K22	2	34.1		
K23	2	23.9	**R12**	2385
K24	2	22.7		
K25	1	11.3	**R13**	1940
K26	2	44.1		
K27	2	25.9	**R14**	1780
K28	2	28.5		
				30350

5.3.6 Stormwater drainage – cost estimate

This section presents an overview of the stormwater drainage cost estimate. Detail cost estimate breakdown is included in Annex 5.9.

Table 5.68: Stormwater drainage system – Investment cost summary

STORMWATER DRAINAGE - INVESTMENT COST SUMMARY (€)		
1	Preliminary works	**26,126**
2	Earthworks	**413,952**
3	Crushed-stone works	**113,851**
4	Concrete works	**3,701**
5	Miscellaneous works	**41,400**
6	Bio-technical works	**101,421**
7	Contingencies (10%)	**70,045**
	Gross total - no VAT, no supervision	**770,497**
11	Construction supervision - 3%	**2,311**
12	Gross total - no VAT, construction supervision included	**772,808**

5.4 Local roads

5.4.1 Introduction

As a part of development of infrastructure for tourism at Vlasina Lake it is also planned to undertake construction and/or upgrade of a number of local roads. Namely, reconstruction/construction of all local roads in the territory of Surdulica municipality has been elaborated in the Preliminary Design with Feasibility Study of local road network in the territory of Surdulica (reference documentation 21). Consequently, corresponding detail project design has been prepared, as well (reference documentation 22). This chapter provides an overview of planned works related to construction and/or upgrade of local roads in the zone of Vlasina Lake, as outlined in the aforesaid technical designs and investigates feasibility of these works.

It should be noted that the scope of this Feasibility Study is limited to a number of local roads in the Project Area, unlike the aforesaid designs that consider all local roads throughout the municipality.

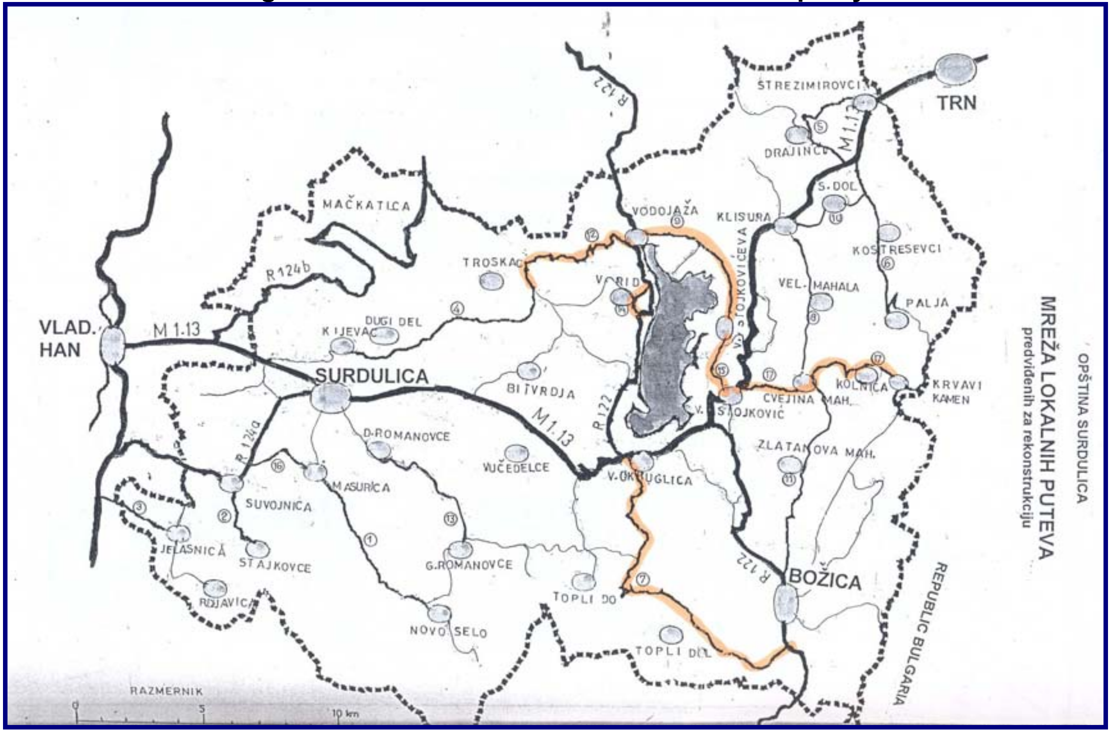
5.4.2 Current status of local road network

The total length of roads considered in the abovementioned Preliminary Design is around 132km. It should be noted that all road directions subject to the preliminary design are existent, and therefore practically these roads need to be upgraded and extended in order to improve traffic capacity and enable for better communications.

These roads are currently characterized by: quite a lot of “sharp” horizontal curves where it is not possible to reach the speed over 10 km/h, they are mostly suitable for one-way traffic and elements of longitudinal grade are unsuitable, with grades up to 30%. Pavement structure, except for 2 traffic directions with asphalt, is mostly earth or partially spread crushed stone or gravel. In most cases there are no drainage facilities.

General layout of all local roads in Surdulica municipality included in the Preliminary Design, and the roads that are in the scope of this study is shown in the following figure. The following local roads pertaining to the Project Area are considered in this study: **7, 9, 12, 14, 15** and **17**. Furthermore, staging of the roads reconstruction would be also considered in order to match proposed strategy of tourism development.

Figure 5.47: Local roads in Surdulica municipality



5.4.3 Design concept

Design concept is harmonized with the planning documents of the Surdulica municipality and in accordance with the standard for road design JUS.U.C4. 301-310, specifically rules for design of access and roads with low traffic circulation. The following table presents basic technical elements of road sections within the scope of this study:

Table 5.69: Design elements of local road network considered in the Study

No	Road direction	Road type	Length L(km)	Width W(m)	Horizontal curve		K	Longitudinal grade		V_0 (km/h)
					R_{min}	R_{mean}		I_{max}	I_{mean}	
7	Promaja-Topli Dol-Drajcina Mahala	3	17.65	3.0	15.0	170.0	459	18.0	5.3	57
9	Vodojaza-Taraija-Vlasina Stojkoviceva	2	8.54	4.0	15.0	178.0	207	18.0	4.8	69
12	Vodojaz-Cemernik	2	6.81	4.0	15.0	169.0	290	18.0	7.6	61
14	R122-Vl.Rid-R122	1	2.36	4.75	15.0	107.0	356	18.0	7.3	53
15	M1-13-Vlasina Stojkoviceva	2	1.55	4.0	45.0	229.0	233	16.4	4.5	71
17	M1-13-Vlasina Stojkoviceva-Kolinica-Krvavi Kamen	3	8.66	3.0	15.0	219.0	416	18.0	7.4	62

Notes:

- *Numbering in accordance with the Preliminary Design*
- *K-curve characteristic (km/^{0})*
- *V_0 – expected driving speed (km/h)*

The abovementioned roads have been grouped in three basic categories, as per type of typical cross section of the road structure. The following figures show basic characteristics of road types 1, 2 and 3 which are included in this study.

Figure 5.48: Typical cross section – Road type 1

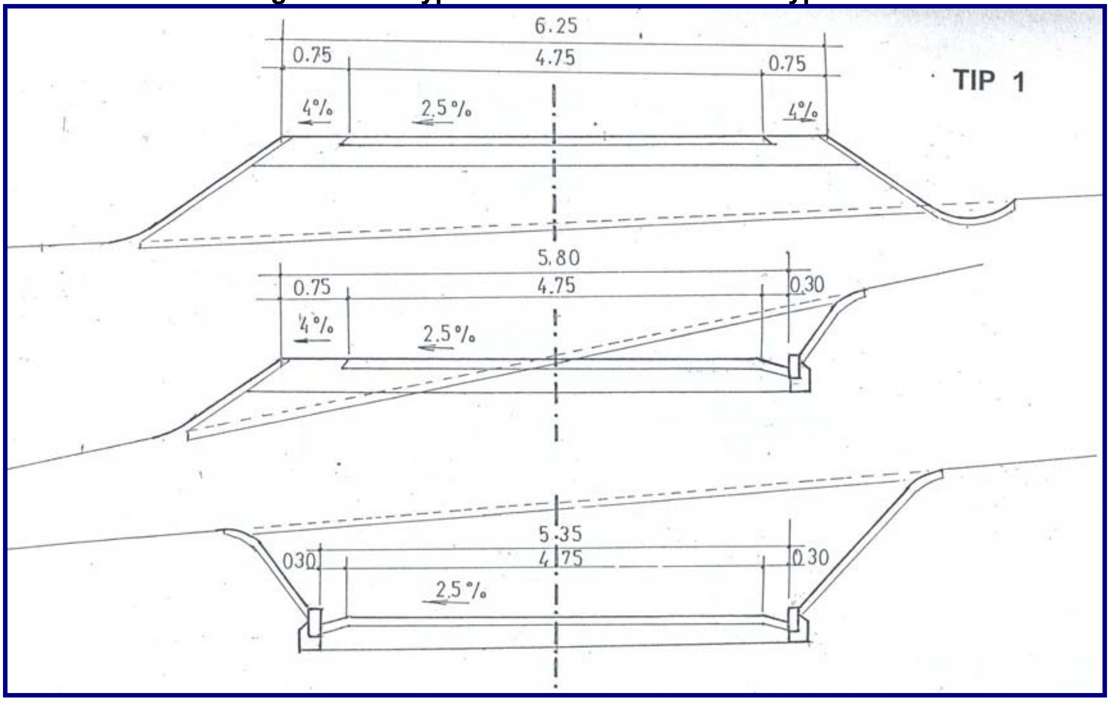


Figure 5.49: Typical cross section – Road type 2

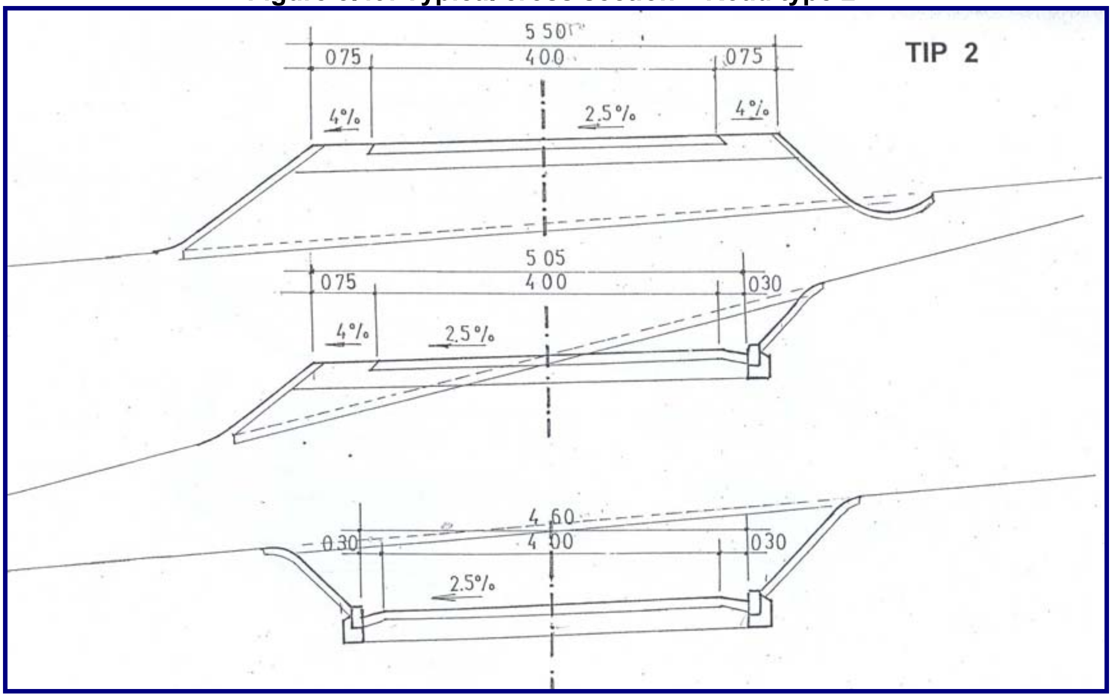
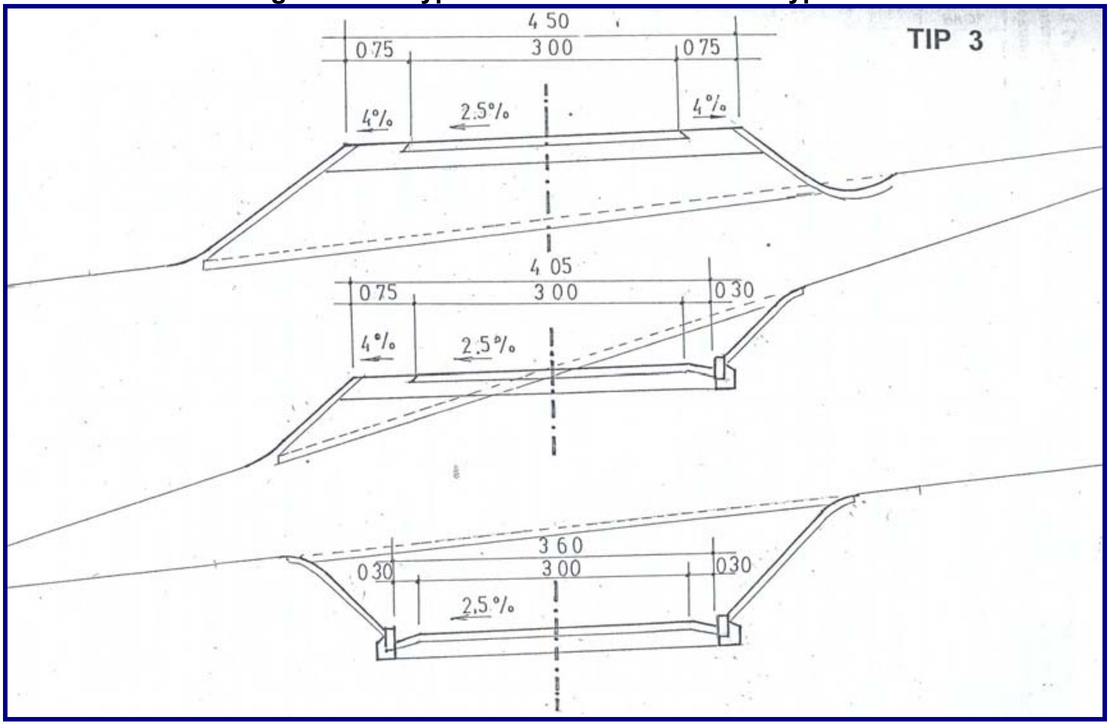


Figure 5.50: Typical cross section – Road type 3



On road directions with one-way traffic, with roadway width of W=3.0m and W=4.0m, there are planned passing places.

Pavement structure is flexible with broken stone base and asphalt paving type BNHS 16:

- Paving - asphalt BNHS 16, h=7cm
- Pavement base - broken stone 0/31, h=25cm
- Total - H=32 cm

At such points where it is assessed that road bed is unsuitable for frost impact, it shall be replaced by a material which can sustain frost impact in a minimum 30cm thick layer.

Drainage: passage of surface water through the road body is envisaged by means of pipe culverts of Ø800mm with front walling both on inlet and outlet side.

Junction to Regional and Main Roads: in the zone of junction about 50m long, all local roads have pavement width for two-way traffic.

In line with the proposed realistic scenario of tourism development, and with the consultations with the representatives of the municipality of Surdulica, the following phasing of the roads reconstruction is envisaged.

Table 5.70: Proposed phasing of local roads reconstruction

No	Road direction	Road type	Length (km)
	Phase 1 reconstruction		
7	Promaja-Topli Dol-Drajcina Mahala	3	2.70
12	Vodojaz-Cemernik	2	6.81
14	R122-Vl.Rid-R122	1	2.36
15	M1-13-Vlasina Stojkoviceva	2	1.55
17	M1-13-Vlasina Stojkoviceva-Kolinica-Krvavi Kamen	3	3.90
	Total Phase 1		**17.32**
	Phase 2 reconstruction		
7	Promaja-Topli Dol-Drajcina Mahala	3	14.95
9	Vodojaza-Taraija-Vlasina Stojkoviceva	2	8.54
17	M1-13-Vlasina Stojkoviceva-Kolinica-Krvavi Kamen	3	4.76
	Total Phase 2		**28.25**

5.4.4 Feasibility of roads reconstruction

5.4.4.1 Objectives

Prime objective of the Preliminary Design and corresponding study (reference documentation 21) was to justify establishment of traffic conditions for demographic recovery and tourism development based on sustainable economic, social and cultural development of the Surdulica municipality, formulated by Spatial Plan of the Republic of Serbia in Official Gazette of RS no. 13/96.

Specific goals were the following:

1. Improvement of spatial organization of traffic and transport system within the municipality through modernization of local and unclassified roads, as well as more efficient inclusion in the system of the existing main and regional roads.
2. Provision of higher degree rationality and cost efficiency in transport of passengers and goods with higher level of traffic safety.
3. Enabling establishment of public passenger transport lines within the municipality on the local road network, as well as better link with administrative centre in Surdulica.
4. Improvement of traffic network enables more intensive utilization of local potentials for upgrading of tourism, agriculture, cattle breeding, craftwork and other activities which require good quality of road network. This would create realistic conditions for survival of autochthonous population at the existing locations.
5. Establishment of conditions for normal growth of population in the settlements where drastic decrease has been registered in a long past period.

6. Growth of population income, motorization degree and intensity of vehicles use.

The aforesaid objectives are rather complaint with the objectives of this Feasibility Study. In particular it refers to the objectives 1 and 4.

5.4.4.2 Assessment of current status

Existing main and regional roads constitute the skeleton of traffic links in the territory of the municipality, as well as a link between this municipality and other parts of the Republic of Serbia. Characteristics of the road network, ranging from primary to local, in the considered area resulted from terrain configuration. Road directions are located in the valleys of river courses or crest routes suitable for terrain surmounting.

Local road network is characterized by low quality of structural elements. According to available data, there are about 264 km of local roads in the municipality, out of which 42km with asphalt paving, and about 222 km with earthen paving, i.e. without pavement structure.

The scope of the aforesaid Preliminary Design included 17 sections of local roads with total length of 132km, while scope of this Feasibility Study includes 5 sections with total length of 43km, split further in two phases.

All local roads under considerations are located in hilly-mountainous terrain at altitudes over 1000 masl. Pavement structure is, as a rule earthen, with occasionally spread gravel and stone. Pavement surface is rough, without gutters, so that the water flows over the pavement, creating gullies in the pavement itself.

5.4.4.3 Methodology

5.4.4.3.1 General

The feasibility study investigates spatial, environmental, social, financial and economic feasibility for the selected solution elaborated in the Preliminary Design.

The traffic line construction feasibility study analyzes all the above aspects within defined planning period up to 2035. Costs and benefits of traffic line construction/reconstruction are subject to analysis. The costs include construction and operation costs for the planning period.

Two basic options were considered, namely:

1. **Zero Option "O";** implies no construction and no reconstruction of envisaged local road network in the Surdulica municipality, and calculation of total costs of utilization of such road network in the planning period.

2. **Construction Option "G";** construction according to the design of road network reconstruction, which implies investments but also improvement of structural road elements which effect in lower costs.

Economic Benefits (EK)

Economic benefits are calculated as difference between undiscounted total costs of option O (no construction) and option G (construction of road network). In order to identify the benefits, total costs of option G should be lower than those of option O.

However, in addition to these direct benefits based on the difference between costs of operation of road network before and after construction, there are also indirect economic benefits arising from reconstruction of road network in the gravity zone of a road.

Internal Rate of Return (IRR)

Internal rate of return is the value which shows whether the economic benefits are sufficient to justify investments in construction of designed structure, in this case – reconstruction of local roads in the Surdulica municipality.

It is expressed in percentages as discount rate of investments for construction in the planning period. In other words, what would be the interest rate in percentages for expressed economic benefits (EK) if the Investor should invest that amount in a bank instead of construction.

Thus calculated percentage (fictive interest) is called internal rate of return (IRR) which is compared to the opportune cost of capital (OCK). If IRR ≥OCK, it is concluded that investments in construction of this structure are justified.

Opportune Costs of Capital (OCK)

Opportune costs of capital have the character of referent criterion for assessment of determined rate of fructification of the capital required for implementation of considered project. Value of OCK depends on social orientation and establishment of a balance between offer and demand, accumulation and investments, import and export, vacant job positions and those who are looking for jobs, etc. In practice, this value is variable in time and it is expressed as a discount rate for long-term investment loans.

Reliability of Construction Feasibility

Since the internal rate of return is determined based on economic benefits, which result from the difference in total costs, and since some costs are based on percentages, it is necessary to bear in mind unreliability of such estimates.

In practice, average values are taken, because they are the most probable ones, but there are always pessimistic and optimistic scenarios, which are extreme values and thus less probable.

Given the importance of decision made on the basis of IRR indicators, it is necessary to provide a reliable answer to the question of absolute reliability of subject project. A reliable answer requires sensitivity analysis of parameters in realistic cases of achieving the values of all costs and benefits, which means in the range of pessimistic and optimistic values of forecasted parameters which generate such costs.

Absolute feasibility exists if all the three reliability levels ($IRR_{minimum}$, $IRR_{average}$, $IRR_{maximum}$) ≥ OCK. Otherwise, there is relative feasibility. Absolute unfeasibility exists if all the three levels of IRR < OCK.

5.4.4.3.2 Costs

5.4.4.3.2.1 Construction Costs (G)

Construction costs included in this study are based on a detail bill of quantities as elaborated in the Detail Design of local road upgrade. The following basic items are related to construction costs:

- design and supervision
- land acquisition
- earth works
- pavement structure
- culverts and bridges
- supporting walls
- crossroads
- miscellaneous works

The following table shows estimated investment costs for proposed phased reconstruction of local roads within the scope of this study.

Table 5.71: Estimated investment costs for roads reconstruction

Item No.	Description	Civil works (€)	Local costs (€)	Total costs (€)
1 Local road No 7: Promaja - Topli Dol - Drajčina Mahala (L=17.65km)				
	Description	**Civil works (€)**	**Local costs (€)**	**Total costs (€)**
1.1	Preparatory works (route marking, cutting and clearing of shrubs and trees)	24,547		**24,547**
1.2	Earth works (machine excavation, compaction, backfilling and leveling)	376,671		**376,671**
1.3	Concrete works (compacted gravel layer, concrete grade 30, installation of pipes DN400 and DN800)	268,560		**268,560**
1.4	Road superstructure (removal of existing asphalt layer, road sub-base, bearing layer of bituminized stone material BNHS-16, d=7cm)	1,089,320		**1,089,320**
1.5	Land acquisition (8,83ha)		52,980	**52,980**
1.6	Contingencies - 10%	175,910		**175,910**

1	**Sub-total - Local Road No 7**	**1,935,009**	**52,980**	**1,987,989**
		L(km)	**Type**	**Unit cost (€/km)**
1	**Local Road No 7 - unit cost**	**17.65**	**3**	**112,634**

2 Local Road No 9: Vodojaža - Taraija - Vlasina Stojkovićeva (L=8.54km)				
	Description	**Civil works (€)**	**Local costs (€)**	**Total costs (€)**
2.1	Preparatory works (route marking, cutting and clearing of shrubs and trees)	10,948		**10,948**
2.2	Earth works (machine excavation, compaction, backfilling and leveling)	245,519		**245,519**
2.3	Concrete works (compacted gravel layer, concrete grade 30, installation of pipes DN400 and DN800)	58,403		**58,403**
2.4	Road superstructure (removal of existing asphalt layer, road sub-base, bearing layer of bituminized stone material BNHS-16, d=7cm)	682,125		**682,125**
2.5	Land acquisition (4,27ha)		25,620	**25,620**
2.6	Contingencies - 10%	99,700		**99,700**
2	**Sub-total - Local Road No 9**	**1,096,695**		**1,122,315**
		L(km)	**Type**	**Unit cost (€/km)**
2	**Local Road No 9 - unit cost**	**8.54**	**2**	**131,419**
3 Local Road No 12: Vodojaž - Čemernik (L=6.81km)				
	Description	**Civil works (€)**	**Local costs (€)**	**Total costs (€)**
3.1	Preparatory works (route marking, cutting and clearing of shrubs and trees)	8,699		**8,699**
3.2	Earth works (machine excavation, compaction, backfilling and leveling)	240,400		**240,400**
3.3	Concrete works (compacted gravel layer, concrete grade 30, installation of pipes DN400 and DN800)	51,820		**51,820**
3.4	Road superstructure (removal of existing asphalt layer, road sub-base, bearing layer of bituminized stone material BNHS-16, d=7cm)	541,928		**541,928**
3.5	Land acquisition (3,40ha)		20,400	**20,400**

3.6	Contingencies - 10%	84,285		**84,285**
3	**Sub-total - Local Road No 12**	**927,132**		**947,532**
		L(km)	**Type**	**Unit cost (€/km)**
3	**Local Road No 12 - unit cost**	**6.81**	**2**	**139,138**
4 Local Road No 14: R122-Vlasina Rid-R122 (L=2.36km)				
	Description	**Civil works (€)**	**Local costs (€)**	**Total costs (€)**
4.1	Preparatory works (route marking, cutting and clearing of shrubs and trees)	4,441		**4,441**
4.2	Earth works (machine excavation, compaction, backfilling and leveling)	71,284		**71,284**
4.3	Concrete works (compacted gravel layer, concrete grade 30, installation of pipes DN400 and DN800)	14,390		**14,390**
4.4	Road superstructure (removal of existing asphalt layer, road sub-base, bearing layer of bituminized stone material BNHS-16, d=7cm)	234,899		**234,899**
4.5	Land acquisition (1,41ha)		8,460	**8,460**
4.6	Contingencies - 10%	32,501		**32,501**
4	**Sub-total - Local Road No 14**	**325,015**	**8,460**	**333,475**
		L(km)	**Type**	**Unit cost (€/km)**
4	**Local Road No 14 - unit cost**	**2.36**	**1**	**141,303**
5 Local Road No 15: M1-13 - Vlasina Stojkovićeva (L=1.55km)				
	Description	**Civil works (€)**	**Local costs (€)**	**Total costs (€)**
5.1	Preparatory works (route marking, cutting and clearing of shrubs and trees)	2,926		**2,926**
5.2	Earth works (machine excavation, compaction, backfilling and leveling)	51,003		**51,003**
5.3	Concrete works (compacted gravel layer, concrete grade 30, installation of pipes DN400 and DN800)	25,895		**25,895**
5.4	Road superstructure (removal of existing asphalt layer, road sub-base, bearing layer of bituminized stone material BNHS-16, d=7cm)	131,929		**131,929**
5.5	Land acquisition (0,77ha)		4,620	**4,620**

5.6	Contingencies - 10%	21,175		**21,175**
5	**Sub-total - Local Road No15**	**232,928**	**4,620**	**237,548**
		L(km)	**Type**	**Unit cost (€/km)**
5	**Local Road No 15 - unit cost**	**1.55**	**2**	**153,257**

6 Local Road No 17:M1-13-Vlasina Stojkovićeva - Kolinica - Krvavi Kamen(L=8.66km)				
6.1	Preparatory works (route marking, cutting and clearing of shrubs and trees)	15,583		**15,583**
6.2	Earth works (machine excavation, compaction, backfilling and leveling)	228,936		**228,936**
6.3	Concrete works (compacted gravel layer, concrete grade 30, installation of pipes DN400 and DN800)	163,271		**163,271**
6.4	Road superstructure (removal of existing asphalt layer, road sub-base, bearing layer of bituminized stone material BNHS-16, d=7cm)	542,180		**542,180**
6.5	Land acquisition (3,46ha)		20,760	**20,760**
6.6	Contingencies - 10%	94,997		**94,997**
6	**Sub-total - Local Road No 17**	**1,044,967**	**20,760**	**1,065,727**
		L(km)	**Type**	**Unit cost (€/km)**
6	**Local Road No 17 - unit cost**	**8.66**	**3**	**123,063**
INVESTMENT COST SUMMARY - PHASE 1				
1	**Local Road No 7, L=2.70km**			**304,112**
3	**Local Road No 12**			**947,532**
4	**Local Road No 14**			**333,475**
5	**Local Road No 15**			**237,548**
6	**Local Road No 17, L=3.90km**			**479,946**
	Gross total - no VAT, no supervision			**2,302,613**
	Construction supervision - 3%			**69,078**
	Gross total - no VAT, construction supervision included			**2,371,691**
INVESTMENT COST SUMMARY - PHASE 2				

1	Local Road No 7, L=14.95km			1,683,878
2	Local Road No 9			1,122,315
6	Local Road No 17, L=4.76km			585,781
	Gross total - no VAT, no supervision			3,391,973
	Construction supervision - 3%			101,759
	Gross total - no VAT, construction supervision included			3,493,732

INVESTMENT COST SUMMARY - GROSS TOTAL				
1	Local Road No 7			1,987,989
2	Local Road No 9			1,122,315
3	Local Road No 12			947,532
4	Local Road No 14			333,475
5	Local Road No 15			237,548
6	Local Road No 17			1,065,727
	Gross total - no VAT, no supervision			5,694,586
	Construction supervision - 3%			170,838
	Gross total - no VAT, construction supervision included			5,865,423

5.4.4.3.2.2 Maintenance Costs

Road maintenance costs should be spread over the planning period. They can be generally divided into the following items:

1. Maintenance of road body, implying maintenance of shoulders, grass mowing, cutting of shrubs and cleaning of deposits
2. Maintenance of drainage systems, implying cleaning of ditches and gutters, maintenance of drainage systems and cleaning of culverts
3. Maintenance of road signalization and furniture, implying maintenance of protective fence, marker posts, vertical signalization, horizontal signalization, etc.
4. Winter maintenance, implying removal of snow and applying road salt,
5. Maintenance of pavement structure, depending on the type of pavement structure and traffic load, usually consisting of:
    a. local repairs, grouting of joints, cracks and patching,
    b. periodical renewal of wearing course by thin layers,
    c. improvement with preventive maintenance measures,

d. improvement with corrective maintenance measures.

5.4.4.3.2.3 Operation Costs (E)

Road operation costs on transport of goods and passengers consist of initial-final costs which arise during loading, handling, unloading and costs of pure transport (e) arising during transport under influence of structural elements of road and kind of vehicle.

Costs of Pure Transport (e)

1. Costs of resources
    a. fuel consumption
    b. oil consumption
    c. tire consumption
    d. vehicle maintenance and repairs
2. Weather-related costs
    a. vehicle depreciation
    b. interests to engaged funds
    c. drivers' salaries
    d. administrative fees
    e. vehicle registration and insurance
3. Type of vehicle; in public transport, there are three kinds of vehicles:
    a. passenger vehicles (PA)
    b. freight vehicles and buses (TV)
    c. car-trains and trailer trucks (AV)

In this specific case of local roads in Surdulica municipality, where most roads are in mountainous terrains, expected traffic implies tourist vehicles and trucks (buses), whereas car-trains are not envisaged.

4. Model passenger vehicle (PA)
    a. medium-weight vehicle Gbr≈11kN
    b. motor power N ≈55kW
5. Model heavy vehicle (TV)
    a. medium-weight vehicle Gbr≈160kN
    b. motor power N ≈150 kW

6. Average unit consumption per vehicle

Basic average unit consumption of resources is taken on horizontal road (i=0), flat asphalt pavement ($w_k \leq 0.02$) and referent speed V_0=60 km/h.

Resource	Measurement unit	PA	TV
Fuel	litres/ vehicle km	0.070	0.206
Oil	litres/vehicle km	$1.40 x 10^{-3}$	$12.00 x 10^{-3}$
Tires	set/vehicle km	$2.00 x 10^{-5}$	$1.60 x 10^{-5}$
Spare parts	car/vehicle km	$1.10 x 10^{-6}$	$7.80 x 10^{-8}$
Mechanic's working hours	hours/vehicle km	$6.60 x 10^{-4}$	$34.90 x 10^{-4}$

7. Price of time-dependent costs

Depreciation (A)

A= $\frac{Cv}{axV+b}$ [€/vehicle km]

PA; a = 250, b = 65,000
TB; a = 2000, b = 160,000

Cv- vehicle price

Interest (K)

K = $\frac{0.5x(Cv+Cg)xI}{axV+b}$ [€/vehicle km]

Where:
PA; a = 150, b = 39,000
TV; a = 1200, b = 96,000
I – interest
Cv – vehicle price
Cg – price of tire set

Drivers' salaries

LD = $\frac{BLD+D}{K_B xfxK_B x(V-70)}$ [€/vehicle km]

BLD – gross monthly salary of driver (€)
D – monthly per diems (€)
K_B-average annual mileage of truck, TV ≈ 280,000 km
f – driving factor ≈ 0.60
V – driving speed

Administrative costs (UR)

UR = 3.21 x 10^{-4} BLD [€/vehicle km]

Vehicle registration and insurance (R)

R = 0.015 for PA [€/vehicle km]
R = 0.040 for TV [€/vehicle km]

8. Correction of unit consumption of resources per vehicle compared to V_0=60 km/h, i=0, wk≤0.02 Speed reduction V_0<60 km/h

In case of speed reduction on horizontal road with asphalt paving from V_0=60 km/h to smaller speed, due to distortion of road, there occurs higher consumption of resources and consequently higher unit costs of transport.

For specific calculation of that effect, driving diagrams for characteristic PA and TV vehicles were used.

Impact of Longitudinal Grade of Grade Line (i) - K_i

Due to additional resistance of ascent, the consumption of resources which is not time-dependent rises (fuel, oil, tires, spare parts and mechanic). Coefficient of increase is determined based on driving diagrams of vehicles PA and TV.

Impact of Increased Rolling Resistance due to kind and condition of surfacing (w_k) - K_k

Coefficients of increased consumption of resources which are not time-dependent pertain to referent rolling resistance on asphalt surfacing, the resistance of which is w_k≤0.02 and K_k=1.00.

Final Unit Cost of Pure Transport - e

$e = e_0 + e_0' \times K_i \times K_w$ [€/vehicle km]

Operation Costs in the Planning Period

$E = T \times 365 \times (Q_{PA} \times E_{PA} + Q_{TV} \times e_{TV}) \times L$ [€]

Wherein:
T – planning period (years)
Q_{PA} – average daily number of passenger vehicles in the section
Q_{TV} – average daily number of freight vehicles (buses) in the section
e_{PA} – unit cost of pure transport PA
e_{TV} – unit cost of pure transport TV
L – length of section (km)
E – total costs of operation in ther planning period in examined section

5.4.4.3.2.4 Costs of Travelling Time (P)

Costs of travelling time spent by passengers on the road are a part of total costs in the process of assessment of economic benefits expected from utilization of road during the planning period. The costs depend on the number of passengers, travelling time and purpose of travel.

According to the purpose of travel, passengers are usually grouped in three groups, as follows:

1. **A group – Business trips (S)**, where the value of travel is taken as 100% cost of working hour in a company;
2. **B group – Daily travel to work and back (R)**, for which the cost of hour is reduced by 25-50%;
3. **C group – Tourist travel (T)** is inherently for leisure. For these travels, the cost of working hour is reduced by 50-75%.

Among vehicles in traffic, passenger transport is in passenger cars and buses:

- occupancy of passenger vehicle is 2.2 passengers
- occupancy of bus is (due to local transport) 20 passengers

General formula for total costs is:

$$P = T \text{ x } 365 \text{ x } \frac{L}{V_{av}} [(Q_{PA} \text{ x } 2.2 + Q_A \text{ X } 20) \text{ x } (P_S \text{ x } C_S + P_R \text{ x } C_R + P_T \text{ x } C_T)] \ldots\ldots \quad [€]$$

Wherein:

T - planning period (years)
L - length of examined road section
V_{av} - average, weighed speed in the section
Q_{PA} - average daily number of passenger cars in the section
Q_A - average daily number of buses in the section
P_S - percentage of business trips (in working process)
P_R - percentage of travels to work and back
P_T - percentage of tourist trips
C_S - cost of hours of business trip
C_R - cost of hours of travel to work and back
C_T - cost of hours of tourist trip

5.4.4.3.2.5 Costs of Traffic Safety (B)

Costs of traffic safety include all damages from traffic accidents, namely:

- material damage
- costs for injured persons

- costs for dead persons

Causes of traffic accidents are multiple and usually simultaneous, with predominant human factor. In open road, it is most often driving speed unadjusted to road conditions. As curves are very frequent on local roads, with fine surfacing, those are preconditions for speeding and more frequent accidents.

Since there are no statistical data on traffic accidents on local road network in Surdulica, data for similar conditions network shall be used.

Out of the total number of accidents (N) the structure of consequences is the following:

- only with material damage 57.30%
- with material damage and injured persons............ 39.0 %
- with material damage and dead persons................ 3.7 %

Forecasts:

- for asphalt road with outstanding curves, when the vehicles move at maximum allowable speed in free circulation, accidents are reasonable to expect: Nr ≈ 0.6 [accident/million vehicles-kilometres]

Total Costs of Traffic Safety

(E) $B = B_m + B_P + B_S$ [€]

$$B = Nr \times \frac{365 x Q_{AV} x \sum L}{10^6} = (0.573 \times m + 0.390 \times p + 0.0375 \times s) \times 20.... \quad [€]$$

Wherein: Nr - expected number of accidents on the road expressed as the number of accidents per million vehicle kilometres
Q_{av} - average number of vehicles in traffic circulation
$\sum L$ - total length of all sections
m - costs of material damage
p - costs of material damage and one injured person
s - costs of material damage and one dead person

5.4.4.3.2.6 Costs Of Environmental Protection

General

A road, due to motor traffic, produces several risks to the environment:

- water pollution
- pollution and degradation of soil around the road
- air pollution

- noise and vibrations
- aerial pollution
- risk to flora and fauna
- risk to natural and cultural heritage

At the Preliminary Design stage, based on traffic forecasts, zones of particular impacts are identified in layout plan and road sections. In the sections where intensity of emissions exceeds the allowable values according to predetermined criteria, there are protective structures which are also subject to the Preliminary Design. The protective structures can only mitigate the hazardous impacts of traffic on environment to the tolerance limit, but they cannot completely eliminate them. Therefore, some impacts remain permanent, causing permanent damage and costs.

Extent of environmental impacts depends on:

a) traffic load through the number and structure of vehicles
b) speed of traffic circulation
c) spatial position of road (layout plan, longitudinal profile and transverse profile)
d) contents around the road exposed to hazardous traffic emissions (type of soil, hydrological system, inhabited areas, fauna, flora, etc.).

Impact of Local Road Network in Surdulica Municipality

On all sections of local roads envisaged for reconstruction, there is only a minimum traffic load, i.e. below 500 vehicles/day (5th class traffic). Besides, operating speeds do not exceed 40 m/h. In such conditions, no special environmental protection against hazardous emissions is necessary.

5.4.4.3.3 Spatial Consequences

5.4.4.3.3.1 General Consequences

Undoubtedly every supplementing of a space with new road directions or registration of existing road leaves certain consequences on such a space. Such consequences may be favourable or adverse.

Favourable consequences on the surrounding are:

1. Initiation of new economic activities which did not exist before
2. Utilization of natural resources which were previously inaccessible
3. Consequently to revival of economic activities, increased employment of population and national income
4. Increase in value of construction land next to the road for new industrial and other attractive facilities
5. Enabling better communication with administrative and industrial centres

6. Stopping of migrations from the area without functional road communications
7. Revival of natural increase of autochthonous population
8. Increase in gross income of autochthon population

Unfavourable effects of road construction and reconstruction are very rare, and concern possible development of spatial units and emigration. In this case, there shall be no such effects.

Population in Gravity Area

In gravity area of the local road network in Surdulica municipality designed in the Preliminary Design, there are the following settlements: Bitvrdja, Bozica, Vlasina Stojkoviceva, Vlasina Rid, Vlasina Okruglica, Gornje Romanovce, Donje Romanovce, Drajinci, Groznatovci, Jelasnica, Kijevac, Klisura, Kostresevci, Kolunica, Masurica, Novo Selo, Palja, Stajkovce, Strezimirovci, Suvojnica, Suhi Dol, Topli Dol and Troskac.

Except for the settlements of Donje Romanovce, Jelasnica, Masurica and Suvojnica, where the number of inhabitants is mildly stagnant, in all the other settlements there is a dramatic drop of inhabitants with tendency of total disappearance.

Explanation for this situation is primarily that asphalt local roads pass through those four settlements, and then the fact that they are in lowlands. Other settlements are in hilly-mountainous area without good road communications, for which reason each economic activity is on a very low level, resulting in low income and emigration.

Gross Domestic Product per Capita

Gross domestic product per capita in considered gravity area in the Surdulica municipality is not expressed, so that average value of all inhabitants in the municipality shall be used. Statistical data for 2006 shall be used.

Official data for Surdulica municipality are the following:

- number of inhabitants 22,190
- employed population 6008 employed persons
- gross personal income of the employed in 2006 25,751
- income from agriculture (beyond the employed) is 10.3% of total gross product

Calculation of Gross Domestic Product

- Gross personal income of the employed - 25,751 RSD
- VAT ≈ 18% - 4,635 RSD
- Appropriation for expansion of material labour base and investments-estimate≈10% 2.500 RSD
- Total – 32,886 RSD (app. 411 €; assumed 1€=80 RSD)

- Total domestic gross product of the employed in 2006: 6,008 x 411 x 12 = 29,631,456€
- From agriculture ≈ 10.3%. - 3.520.399€
- Total – 33,152,955€
- Per capita: 33,151,955/22,190 = 1,494 ≈ 1500 €/capita/year

Such a low gross product per inhabitant in the Surdulica municipality is primarily the result of low percentage of the employed (≈ 27%), and also falling of salaries behind the average for the Republic of Serbia.

Calculation of Effect of Spatial Effect (PP)

Assumptions for option "O", no construction: the number of inhabitants in settlements within the gravity region still drops rapidly, but the population in such settlements continues, the same as until now, to work and increase their gross income at expected percentage for the whole country, i.e. about 4.5% a year.

Assumptions for option "G", construction of the roads: implies that objective conditions shall be established for utilization of potential capacities in the gravity region with all the effects listed earlier.

Population decrease would stop in the named settlements within the gravity region. Depending on age structure in such settlements, it is expected that in the forthcoming period the natural increase shall be at realistic percentage of about 1.0-1.2% a year. Due to more favourable newly established conditions for all kinds of economic activities (intensive cattle breeding, agriculture, forestry, tourism, etc.) the gross product shall increase a little more intensively than before (estimated at 5% a year).

Based on the above presented assumptions, specific values of spatial consequences are obtained as differences in gross income of population for the next planning period:

Option "O"; - PP_0 = (inhabitants x forecast – now x gross income $\frac{\text{forecast + now}}{2}$) x T

which constitutes a loss due to decrease of population.

Option "G"; -PP_G = (inhabitants x forecast – now x gross income $\frac{\text{forecast + now}}{2}$) x T

which constitutes additional income due to population increase.

Total effect of spatial consequences is obtained as sum of

$PP = PP_G – (P_{po}) = PP_G + P_{po}$

5.4.4.4 Traffic Forecast

5.4.4.4.1 Population Motorization System (M)

Motorization system is calculated as number of vehicles per one thousand inhabitants.

- In 2007, as confirmed by the Ministry of Internal Affairs, there were freshly registered 560 passenger vehicles, 75 freight vehicles and 15 tractors, which together with earlier registered vehicles equals 7,800 vehicles in the municipality, with approximate structure as the freshly registered ones (85% passenger and 15% freight).
- Motorization degree $M = \frac{7{,}800}{22{,}190} = 351 \approx 350$ veh/1000 inhabitants

Such a high motorization degree is not in compliance with the income of population (1500 €/capita per annum) and it is the result of large number of vehicles older than 10 years which are not unregistered, and which were bought in earlier period when the income per inhabitant was higher than nowadays.

5.4.4.4.2 Forecast of Traffic Circulation in the Part of Local Road Network Which is Envisaged for Reconstruction

Counting of traffic on local roads in the Surdulica municipality has not been done. The only possibility is to obtain the possible number of vehicles on local roads on the basis of motorization degree and intensity of vehicle use.

Intensity of vehicle use or population mobility is expressed through frequency of vehicles use covering of a number of kilometres in specific time unit. Given the low income of population, poor local road network and obsoleteness of the car pool, intensity of vehicles use is also low, which is obviously the case on the existing local road network in the Surdulica municipality.

According to current traffic situation on local roads, and also the very high motorization degree, it may be reasonably estimated that the coefficient of daily use of available vehicles is about $\xi = 0.30$, and that after construction of network with modern pavement it shall grow to app. $\xi = 0.40$.

Accordingly, it is calculated that the number of vehicles on the move in the settlement on the local network is:

$$[\text{vehicle/day}] \times Q_L = \frac{\text{number of inhabitants}}{1000} \times \text{motorization degree} \times \text{intensity of vehicle use}$$

$$Q_L = \frac{N}{1000} \times M \times \xi$$

In addition to these vehicles on the local network, there shall be other vehicles from other regions which arrive for business or tourist visit to settlements in the gravity region (so called target traffic).

Distribution of traffic from settlements to other settlements is estimated on the basis of activities of such destinations and possible number of tourist vehicles in the regions where intensive tourist attraction is expected (region around the Vlasina Lake, Cemernik, Vlasina Rid and Vlasina Okruglica).

In the Preliminary Design – Feasibility Study (reference documentation 21), there is a calculation of number of vehicles in settlements, and the distributed traffic per sections, as average number of vehicles a day in the planning period, for calculation of operation costs in options “O” and “G”.

In separate appendices for all the sections there is a traffic structure with vehicle structure as average daily number of vehicles in planning period, as well as forecasted PGDS for the end of the planning period, by the formula

$PGDS = 2Q_G - Q_O$

wherein:
Q_G – mean value of forecasted traffic for option “G”
Q_O – mean value of forecasted traffic for option “O”

5.4.4.5 Calculation of Costs

This section presents basic assumptions of cost calculation.

5.4.4.5.1 Construction Costs (G)

- design and supervision - per km
- land acquisition - per ha
- earth works - per m^3
- pavement structure - per m^2
- culverts - per piece
- supporting walls - per m^3
- crossroads - piece

5.4.4.5.2 Road Maintenance (OD)

- road body maintenance - per km
- maintenance of drainage system - per km
- traffic signalization and furniture - per km
- winter maintenance - per km
- maintenance of pavement structure - per m^2

5.4.4.5.3 Operation Costs (E)

- vehicle price
  - PA - 10,000 €
  - TV - 70,000 €
- tire price
  - PA - 250 €
  - TV - 1200 €
- oil price - 2.5 €/lit
- fuel price
  - petrol -1.1 €/l
  - diesel -1.0 €/l
- working hour of mechanic - Cm = 7.0 €
- gross monthly salary of driver - BLD=700 €
- per diem of driver (a month) - D = 200 €
- registration and insurance of vehicle
  - PA = 0.015 €/vehicle km
  - TV = 0.015 €/vehicle km
- administrative fee - Ur = 3.21 x 10-4 BLD
- interest to loan - I = 8%

5.4.4.5.4 Costs of Driving Time

- price per hour of business trip or hour in labour process - Cs=4.0 €
- price per hour of travel from work and back - C_R=2.0 €
- price per hour in tourist travel - C_T=1.0 €

5.4.4.5.5 Costs of Traffic Safety

- price of one material damage - Ms = 9,900 €
- price of material damage with one injured passenger - M_E = 9,165 €
- price of material damage with one dead passenger - M_P = 255,590 €

5.4.4.5.6 Spatial Consequences

- average gross domestic income per one inhabitant for option "O" depending on road section - 1500-3800 €/year;
- average gross domestic income per one inhabitant for option "G" depending of road section -1500-4800 €/year;

5.4.4.5.7 Costs of Environmental Protection

Permanent structures of environmental protection included in construction costs.

5.4.4.5.8

Costs Overview

Reference documentation 21 provides a comprehensive presentation of costs, including construction costs, maintenance and operational costs.

5.4.4.6 Internal Rate or Return (IRR)

Internal rate of return (IRR) is calculated by solving exponential equation in spread form:

$$\frac{EK/T}{(1+IRR)^1} + \frac{EK/T}{(1+IRR)^2} + \frac{EK/T}{(1+IRR)^3} + \quad + \frac{EK/T}{(1+IRR)^T} - G = 0$$

Extreme values for IRR are obtained by the following scenario:

For IRR_{max}	$EK_{max} - G_{min} = 0$
For IRR_{mean}	$EK_{mean} - G_{mean}=0$
For IRR_{min}	$EK_{min} – G_{min} = 0$

5.4.4.6.1 Variations of Economic Benefits

As mentioned earlier, economic benefits are formed based on the difference in costs of option “O” (no road construction) and option “G” (construction of the proposed road sections). Since all the costs are based on certain estimates, and each estimate, as a matter of fact, contains some uncertainties, the economic benefits inevitably have some characteristic, i.e. optimistic (EK_{max}) and pessimistic (EK_{min}).

5.4.4.6.1.1 Variations of Construction Costs

Inaccuracy of construction costs may result from quantity of works, while the costs of such works, the same as any other costs are market costs, and may be considered reliable.

For this particular case it has been adopted that variations of the quantities of works can be ±10%.

5.4.4.6.1.2 Variations of Maintenance Costs

The same as in construction costs, the error in maintenance costs results from unreliability of quantity of works. Quantities of works in road maintenance are affected by two important factors: traffic load, and selected strategy of maintenance to ensure the required level of traffic services.

In conditions when there was no recording of traffic on local road network, and since the traffic was forecasted based on current degree of motorization in the municipality and forecast of future growth of population and gross domestic product, all import quite a lot of uncertainty estimated within 35% i.e. mean value ±17.5%.

5.4.4.6.1.3 Variations of Operation Costs

Due to possible deviations in estimate of the number of vehicles in particular settlements, coefficient of mobility and distribution of vehicles per particular sections, as well as percentages of target traffic in tourist season and future economic activities, necessarily indicated to high variation, about 40% i.e. mean value ±20%.

5.4.4.6.1.4 Variations in Travel Costs

Apart from the number of vehicles, the costs of time travel are affected by estimated structure and number of passengers. The traffic percentage is also 40%.

5.4.4.6.1.5 Variations of Safety Costs

The number of traffic accidents on the roads where recording was not done before, is pure statistical value with maximum possible variation about 60% i.e. mean value ±30%.

5.4.4.6.1.6 Spatial Consequences

Effects of spatial consequences are based on assumption that events shall take place by the following scenario:

- If there is no reconstruction of local road network (option “O”), population in settlements in gravity region shall continue the tendency of decrease, until complete abandoning of some settlements in hilly-mountainous areas. The remaining population in settlements shall maintain extensive economic activities with tendency of average increase in domestic product in the municipality, per capita. In the settlements which are in lowlands, and now have asphalt road and link to regional and main roads, no decrease in the number of inhabitants is expected, with moderate growth of 1% and larger increase in domestic product per one inhabitant.
- In case of road reconstruction (option “G”) and enabling of efficient link with roads of higher rank (regional and main roads) and administrative centre in Surdulica. In that case, emigration of inhabitants from settlements in gravity region is not expected. Autochthonous population is expected to grow in compliance with age structure, at the same time using more intensively the potential local possibilities for all forms of economic activities, and accelerated growth of domestic product compared to option “O”.

The financial effects of thus envisaged scenario are spatial consequences (PP) through total difference in domestic product in the planning period.

Option “O”

$$\pm PP_o = [\text{population (forecast-now)} \times \text{domestic product} \times \frac{\text{forecast + now}}{2}] \times T$$

Option "G"

$$\pm PP_G = \text{population (forecast-now)} \times \text{domestic product} \times \frac{\text{forecast + now}}{2}\,] \times T$$

$$PP = PP_0 + PP_G$$

Given the nature of estimate of all factors of this strategy, it is reasonable to expect variation in the range of 45%, i.e. mean value ±22.5%.

5.4.4.6.2 Economic Benefits and IRR

Reference documentation 21 provides an overview of economic benefits and IRR for the proposed project.

5.4.4.7 Conclusions

From table no. 3.X (14), where extreme values of economic benefits were varied, as the basis for calculation of extreme values of internal rate of return, the following values were obtained:

- ⇒ **IRR_{min} = 19.8%**
- ⇒ **IRR_{mean} = 31.1%**
- ⇒ **IRR_{max} = 45.5%**

Since the opportune (actual) cost of capital is

- ⇒ **OCK = 10-12 %**

It may be stated that in all cases

- ⇒ **IRR > OCK**

so it may be concluded that there exists **FEASIBILITY** of investment for construction of local road network included in the project.

5.5 Solid waste management

5.5.1 Current status, background

The municipality of Surdulica is surrounded with the municipality of Vladičin Han in the west, municipality of Crna Trava in the north, Bulgaria in the east and municipalities of Vranje and Bosilegrad in the south.

Its surface area is of 623 km^2 and the average sea level is 475 m. The territory is intersected by numerous water courses including: Vrla, Romanovska, Masurička, Jelašnička and Jerma.

In 2002 the population of the town Surdulica was 10,914 while the population of the whole municipality was 22,190, out of which 11,276 rural.

With regard to collection and disposal of communal solid waste in Surdulica current status can be described as follows:

- There are 3 collection trucks, each of 9 m^3 capacity, in regular operation;
- Local PUC have placed 87 containers of 1.1 m^3, but they also collect waste from 15 containers of 1.1 m^3, owned by industries/commercial sector, and 26 containers of 1.1 m^3, used by public institutions, as well as around 4,000 bins of 120 lowned by households.
- With regard to service coverage, the PUC covers app. 100% of urban population (10,914), plus around 2,500 rural inhabitants in the villages: Alakince, Masurica, Božica, Vlasina and Klisura, which presents 60% coverage of the total population in the municipality.

A single bulldozer is used on contract-basis for waste reshaping on the existing city landfill. Also, an open truck of 4 m^3 capacity and one shovel/skipper are used for waste collection in the town.

On a short-term basis the PUC would like to replace around 5,000 rubbish bins in the town, and to increase the service coverage to app. 70%.

EkoPlast company from Vladičin Han has placed 33 containers of 1.1 m^3 for PET collection in city Surdulica, as well as 25-30 containers in other villages in Surdulica municipality.

Table 5.72: Solid waste collection – service coverage

	Total population	Urban population	Rural population	Total coverage	Urban area coverage	Rural area coverage
Surdulica	22,190	10,914	11,276	13,414 (61%)	10,914 (100%)	2,500 (22%)

The PUC's in the region do not have recycling/separate collection plans. EkoPlast, a private company, started recently with separate collection of PET, plastic foils and paper/cardboard. EkoPlast plans to increase the separate collection system by placing additional boxes/containers Surdulica.

At the local landfill in Surdulica there is registration of type and volume of waste.

There is no air and water pollution control measurement equipment on the landfill, except for 3 piezometers.

Waste is generally disposed by sprawling manner, without much organization, but waste is daily covering with inert material is applied.

Table 5.73: Current waste management for Vlasina area

Institution in charge:	PUC Surdulica
Number of vehicles in charge for Vlasina and frequency of visit to the Vlasina site:	3 garbage trucks are in charge for the entire municipality, originating from the year 1985. There is only one vehicle intended for Vlasina lake, visiting the site once per week. During summer season the number of visit is 3 times per week.
Number of containers:	30
Number of waste collection workers in charge of Vlasina	5 persons (1 vehicle driver, 2 cleaners and 2 hygiene workers) during the year and additional 10 persons in the summer period
People covered:	600 households (1.400 inhabitants in the area) during the whole year, while during the summer period there are 1500 households.
Separate plan of waste collection:	Separate plane of waste collection has not been present yet. At the present time waste is defined as public waste and there is no any plan regulating waste collection and recycling.
Landfill position:	App. 25 km distant from Vlasina
Landfill capacity:	Surface area of 4,2 hectares.
Landfill measuring and composition:	Solid waste weight per week amounts to 6/7 tons, while during summer period the weight of produced waste increase up to 20 tons per week.
Environmental protection at landfill	Advanced devices for the environmental protection has not been present in landfill area

Table 5.74: Waste collection equipment

Containers	**7 m3**	
	1.1 m3	87 pcs for households owned by PUC, 15 pcs for industry/commerce and 26 pcs for public institutions. EkoPlast placed 33 containers of 1 m^3 for PET
	120 l	5,000
Vehicle pool	**Bull-dozer/ compactor**	1 bulldozer part time contracted for work on landfill
	Collection vehicles	3 of 9 m3, in use, 1 open truck of 4 m3 partly in use for waste relocation on the landfill TAM 130, 1990 FAP 1213, 2004 DAF 6232, 1991 1 time / container/week, working days: 6 days/ week, 312 days/year
	Truck-lifter of 7m³	-
	Tractor of 2m³	-
	Other	1 shovel/skipper partly in use onthe landfill

Table 5.75: Waste collection data in 2007

User category	V (m^3)	G (ton) ($\rho_{average}$=285kg/m^3)
Households	6299	
Commercial & industry1	1700	
Public areas (schools etc.)	425	
Total	8424	2401
Waste per person served (13,414) per day (kg)		0.49

Surdulica landfill

The Surdulica landfill called Bubavica is located in village Zagužanje, high in the hilly area, close to the stream Vlaški Do which flow in River Vrla. It is a large area that has sufficient space for the coming years.

The PUC disposes about 8,400 m^3 of waste/year on landfill. This landfill belongs to K3 group of landfills (*K3 group - Official disposal sites - landfills which may still be used up to 5 years, provided renovation is done, with minimal prevention measures*).

Sanitation of this landfill is being processed. Recently the PUC cleaned lot of illicit dumps in urban and suburban area, collecting about 1,200 m^3 of waste which was disposed on the landfill.

Surdulica municipality invested in the design of sanitation and closing of the site. A detail design of sanitation, re-cultivation and further exploitation up to closing, of the existing landfill/dump, was in September 2006 and approved by the Ministry for environmental protection, on 02/11/2006. The design included technology of waste disposal and treatment, leachate treatment, gas management, re-cultivation, civil engineering works.

The design actually promotes conversion of the existing rubbish dump into a three-phased sanitary landfill with estimated life-time of around 16 years.

The investment required is estimated at € 1.3 million. Corresponding funding application was made to the GoS, and EU but there has not been any response so far. The PUC already made a tank for supply of water for fire protection, the fence and gate are placed at the access side, a guard engaged but only during first-shift working hours. First half of the landfill is covered with a layer of soil (20-80 cm) and the trees are planted.

Every collection vehicle is registered in the log-book (date, volume of waste, number of vehicle, type of waste). Also three piezometers are installed, but no wastewater analyses are carried out.

The design foresees wastewater (leachate) treatment plant and degassing wells.

With regard to planning of solid waste management in Surdulica municipality a Preliminary design for evacuation of solid waste from the villages of the municipality

Surdulica, was prepared in 2006. This design includes full service coverage of the municipality, which is in line with the aforesaid landfill design.

There is a modern chemical and microbiological laboratory in Surdulica donated by UNDP.

The landfill is some 5.2 km from the centre of Surdulica, in the hilly area, next to village Zagužanje.

Figure 5.51: Current status of Bubavica site

Table 5.76: Characteristics of Surdulica Bubavica dump site

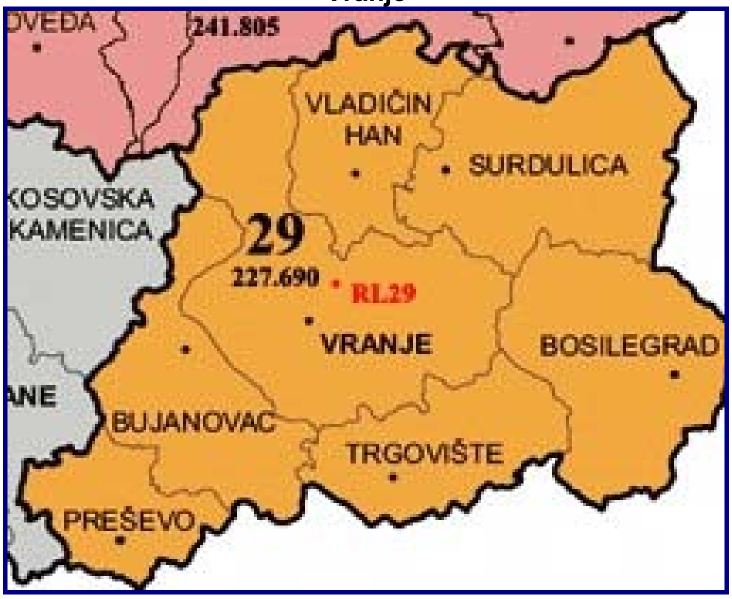
Opened: 1981
Size: 28,400 m^2
General site remarks: Fence and gate are present, security during day time, no drainage, no degassing wells. Bulldozer present for basic daily covering. Vehicles registration. First half of landfill is covered with layer of soil (20-80 cm) and the trees are planted, three piezometers are installed.
Type of waste: municipal waste, some medical. No slaughter waste / dead animals.
Equipment on site: Bulldozer (hired) new one has been ordered.
▪ Leachate 3 piezometers present. Will start soon with water analyses
▪ Fires In the past. No fires recently.
▪ Smell No, closest resident housing: 500m
Upgrade and closure plan: Yes
The PUC Surdulica is user of the site. Owner is the municipality.

5.5.2 National and regional prospective

In accordance with the National Solid Waste Management Strategy, 2003, the municipalities pertaining to Pčinja administrative district are organized in a solid waste management region oriented towards the regional sanitary landfill in Vranje municipality – figure 5.52.

The Pčinja district is located in southern part of Serbia. It has got seven municipalities, including Vranje, Vladičin Han, Surdulica, Bosilegrad, Preševo, Bujanovac, and Trgovište with 7 main urban centres and 356 rural settlements. The total land area is 3,520 km^2 and in accordance with 2002 census its total population was app. 227,690 out of which 108,325 urban and 119,365 rural.

Figure 5.52: Municipalities in Pčinja district pertaining to regional sanitary landfill in Vranje



The existing sanitary landfill site, named "Meteris", is located at the place Suvi Dol some 1,300 m from the from high-way Belgrade-Athens (E-75, corridor 10) and some 5.8 km from the city centre of Vranje. It is well accessible and its exploitation of the landfill started in September 2002. The designed surface of the landfill body is 30,670 m2; the volume of the landfill body is 348,120 m3. It is designed to be filled in 3 phases:

- phase I: some 131,120 m^3 covering half of the landfill area
- phase II: some 121,200 m^3 covering the remaining landfill area;
- phase III: some 95,800 m^3 on top of the area of phase I & II.

Household waste and some industrial waste (mainly from wood processing and textile) are land-filled in Meteris. Limited amounts chemical waste are also land filled. No medical and slaughter waste is noticed. In 2007, 73,000 m^3 of waste was collected (Vranje and Vladičin Han). As this is not weighed (there is no weighbridge) the tonnage is not known. Taking an average of some 235 kg/m^3 (collected) this means some 17,200 ton.

The bottom construction consists of a 2 mm thick HDPE membrane with leachate drainage system in place. No protective geo-textile layer has been applied.

The following figure shows layout of the existing Vranje sanitary landfill called Meteris.

Figure 5.53: Existing Vranje Landfill – Meteris layout

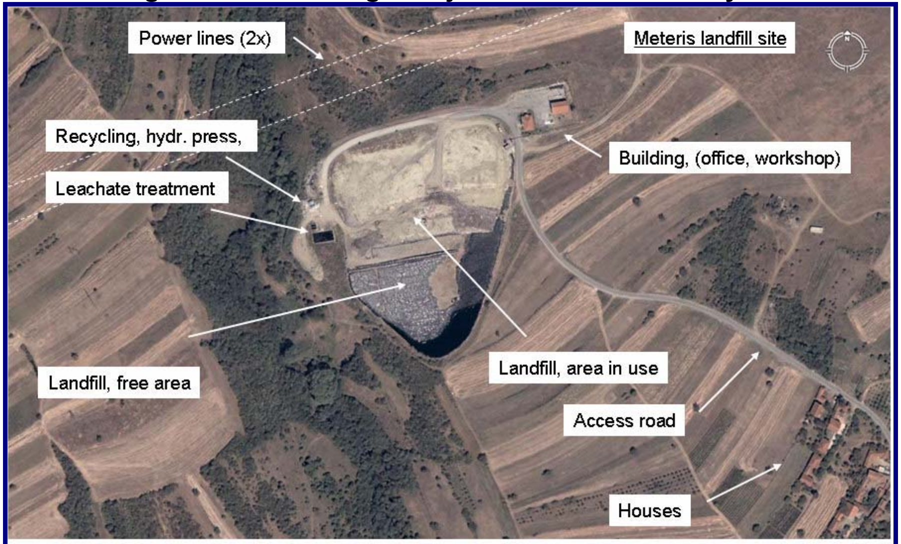


Figure 5.54: View of Vranje landfill

The collected waste-water (leachate) is treated in two lagoons (sedimentation and aeration) with treated water recycling on landfill body. Most of wastewater is, however, discharge into Batlijski Potok stream, flowing into the Southern Morava River.

Landfill gas venting wells have been installed. Waste disposal is done in a contemporary way with compacting of waste by a compactor and with forming of cells, daily covering of waste by inert material, collecting of surface water by drainage channels.

The landfill includes mechanizations: compactor and skipper (for soil preparing), fence, gate, parking area, shed for washing of collection vehicles, administration building with safeguard, offices, toilets and modern laboratory for water and air analysis, transformer as well as a hydraulic press.

A weighbridge and a plant for filtration of the water from lagoons were designed but no installed, because of lack of finances. Length of the new installed water supply line is about 1,300 m. It was observed that the volume of the landfill is filled for some 40-45%, covering almost 1.5 hectares.

The second phase (phase II) is being prepared for filling. Assuming that III phase will be placed over 1st and 2nd phase in line with the design size of the landfill body is sufficient for another 9 years (till the end of 2016), based on receiving waste only from Vranje municipality.

The remaining landfill capacity appears to be one of the major constraints for developing of the defined regional waste management strategy for Pčinja district.

Till date no Regional Solid Waste Management Strategy has been prepared for Pčinja District. This should be one of the first activities to be carried out. The aforesaid regional solid waste management context has been explored in the Feasibility Study Solid Waste Management Pčinja District, South Serbia, Royal Haskoning, 2008 (reference documentation 20).

The following parameters that have impact on the waste amount and composition scenario were considered in the Study:

- Population growth/decrease – generally negative trends have been recorded over the last five years;
- Increase in collection coverage – in urban areas up to 100%, in rural areas limited increase;
- Economic growth – positive economic growth of 3 to 6%, affecting waste quantities, has been assumed for the following period;
- Waste reduction due to separate collection at source and composting at landfill – for Surdulica reduction of 30% was assumed for PET bottles, while reduction of 10% was assumed for plastic foils.

At Meteris landfill waste from Vranje has been land-filled since 2002 and since April 2007 also Vladičin Han is land-filling its waste in Meteris.

The landfill is filled up in three phases with a total volume of 365,000 m^3. 20% of this volume is required for daily covering with soil. The net waste volume therefore is 292,000 m^3. At the beginning of 2008 the first phase was filled-up (some 125,000 m3). Assumed density of the disposed waste after compacting is 750 kg/m^3.

In accordance with the Feasibility Study (reference documentation 20) the remaining life-time of Meteris landfill when only Vranje and Vladičin Han waste is being land filled is some 6 years - second half of 2013. Converting the Meteris landfill in a regional landfill receiving waste from all Pčinja municipalities the remaining life-time will be some 4 years. This practically implies, that, unless significant extension of the current landfill site is done, there is only a very limited possibility to include other municipalities, including Surdulica.

Therefore, in order to be able to propose a viable option of strategically defined regional waste management, the consultants for the aforesaid Feasibility Study envisaged substantial extension of the existing sanitary landfill in Vranje other with appurtenant measures and works.

Namely the consultants for the Feasibility Study have reviewed all studies & investigations available to confirm whether that Meteris location represents the best solution both economically and environmentally in accordance with local requirements and EU environmental rules for category “A”-screened projects and based on the Serbian legal requirements and EU Directives.

The consultants concluded that almost all relevant permitting requirements were fulfilled.

Regarding the *present landfills (dumps)* in the seven municipalities of Pčinja District the main findings are:

- Only Meteris landfill approaches EU standards;
- All other landfill sites are in fact uncontrolled dump sites without any basic
- Environmental protection facility;
- No weighbridge / no registration of type and amount of waste, except registration of type and volume of waste at landfills Vranje and Surdulica;
- Environmental problems exist regarding fires, leachate and smell;
- There are no air and water pollution control measurement equipment on the landfills, except piezometers at landfills Vranje, Surdulica, and Bujanovac;
- In general: Non-compliance with environmental legislation except Meteris landfill;
- The municipalities don’t have any alternative than ongoing dumping on their sites (no alternative sites available);

In the aforesaid Feasibility Study the shortcomings and findings have been addressed a/o:

- Waste scenarios have been developed;
- Proposal have been made for collection vehicles and equipment;
- Proposal and costs estimates are given for the closure of the dumps;

Collection equipment

It is proposed to supply all municipalities in the region with new collection vehicles and containers, to overcome present shortcomings and anticipating the increase in service coverage. It was observed that the current collection practise can be organised in a more efficient manner.

Transfer Stations

A financial/economical analysis in the Study showed that a transfer station can not be justified for Surdulica municipality, and therefore all collected solid waste from the municipality would be brought directly to the regional sanitary landfill.

Recycle line (Waste Separation Plant)

A basic financial/economical analysis shows that a Waste Separation Plant at Meteris landfill site (or any other location) can be justified only because of the relative high revenues from the metals despite the limited waste quantities. I case the metal content is less than assumed the WSP is not feasible.

New sanitary landfill

The existing Meteris landfill will be filled-up around 2012 in case it becomes a regional landfill from 2009 onwards. Otherwise land-filling with Vranje and Vladičin Han waste can continue till 2014.

The most favourable location in the Pčinja district for the regional landfill capacity is Meteris. The new regional sanitary landfill Meteris is proposed to be constructed next to the present landfill. A total area of 5 ha is required and available on site.

The new regional landfill shall have a net volume of 660,000 m^3 which give sit a life-time of around 10 years.

The landfill should be constructed according the EU Directive “land filling” that gives guidelines for preventing pollution of the soil, groundwater or surface waters and ensures sufficient collection and treatment of the leachate. Most of the infrastructure already exists at Meteris. Missing facilities such as a weighbridge are proposed to be constructed.

Figure 5.55: Proposed extension of regional sanitary landfill Meteris (source: Feasibility Study – reference documentation 20)

Regional Waste Management Strategy

A Regional Waste Management Strategy does not exist. It is recommended that all municipalities participate in drawing a (Regional) Waste Management Strategy that sets a/o targets for separate collection.

Financing and investment

The total investment cost for the phase 1 amounts to 16.14 M€, including upgrade of solid waste collection equipment and closure of all existing dumpsites.

5.5.3 Waste quantities in the Project Area – projection

This projection of waste quantities within the project period is based on the following input parameters and assumptions:

- Number of users (permanent population, temporary residents – visitors, tourists in different types of accommodation) is in full accordance with the proposed tourism development strategy – realistic scenario;
- Unit, per capita, waste generation has been adopted based on the records in the existing scheme, and typical quantities per type of user: for rural population 0.35 kg/capita/day, for tourists 0.50 kg/capita/day;

- Occupancy rates which are important for calculation of realistic annual waste quantities are adopted in accordance with the proposed realistic tourism development scenario;
- Considered Project Area has been already covered with solid waste collection service (Vlasina, Klisura, Bozica). It is assumed to retain and improve collection rate, to include all new tourist accommodation capacities as well as some rural settlements that are now not part of the scheme.
- Compaction rate of compactor at the landfill 100% to 35%;
- Required quantity of inert material is 18% of the total required volume for land-filling;
- Compacted waste in compactor truck density 0.85 t/m^3;

The following figures show adopted projection for waste quantities in the Project Area.

Figure 5.56: Total annual solid waste quantities for the Project Area (t/year)

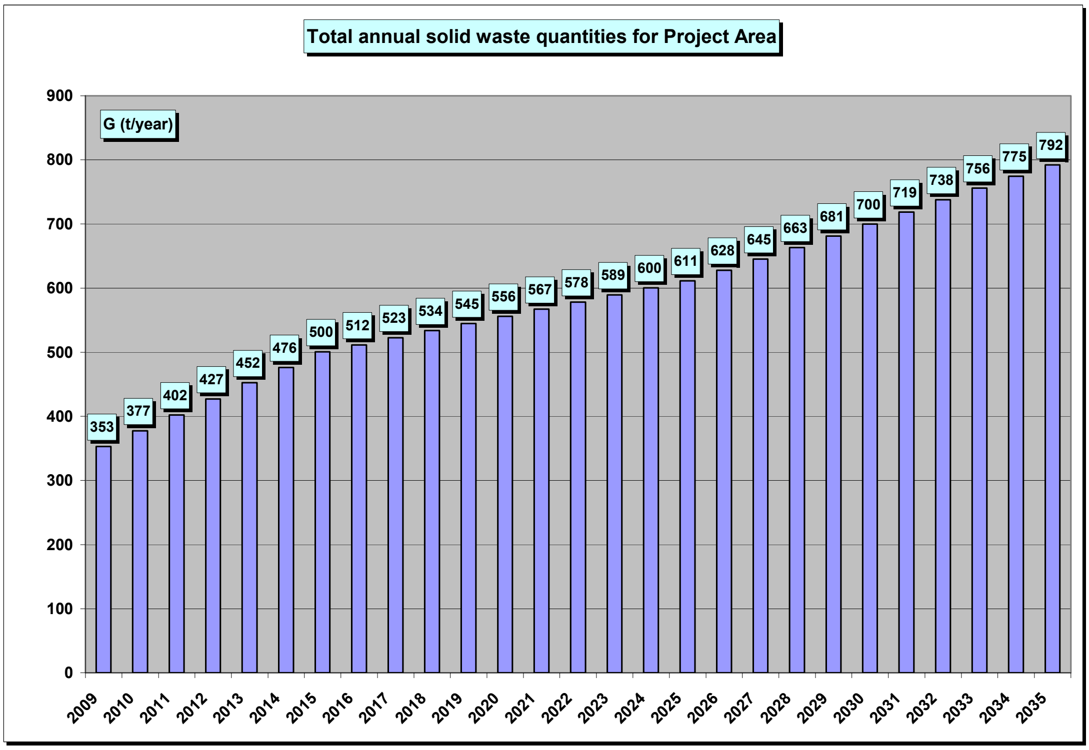


Figure 5.57: Total annual solid waste quantities for the Project Area (m^3/year)

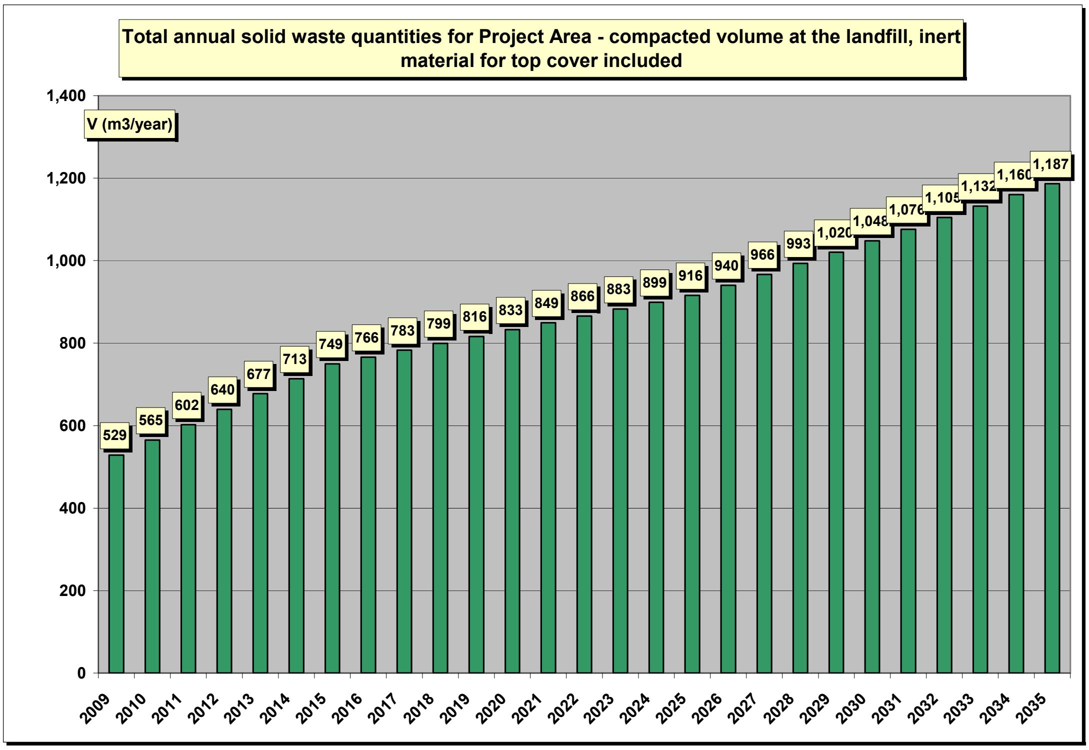


Figure 5.58: Annual solid waste quantities for the Project Area(t)- cumulative

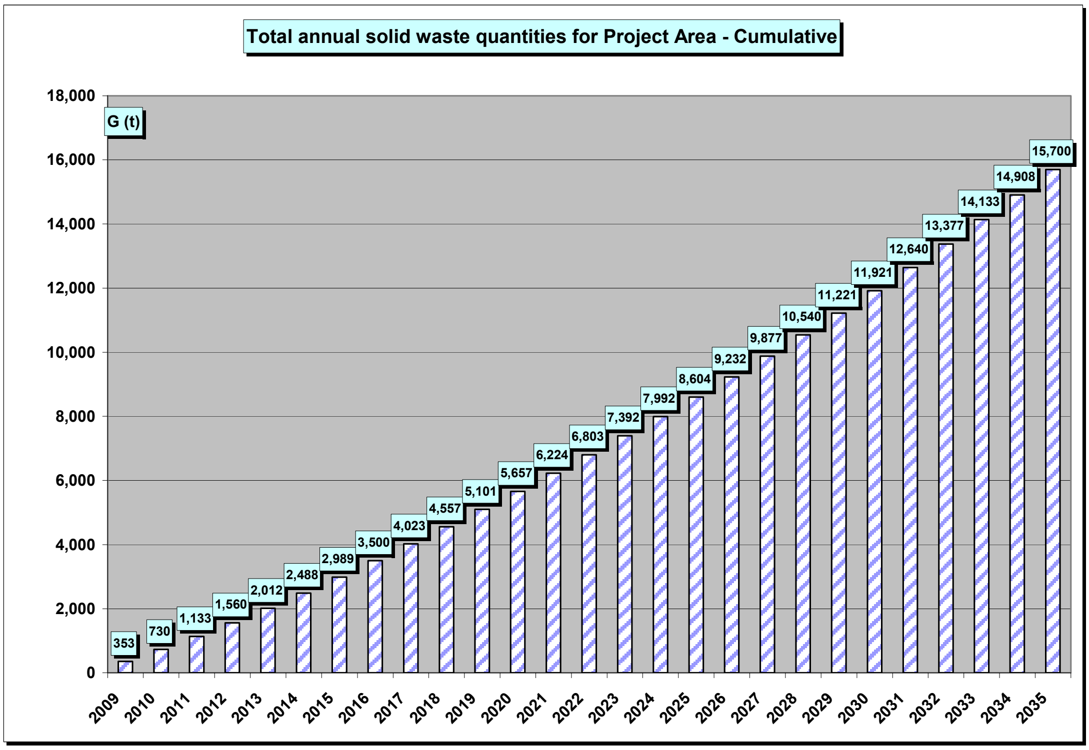


Figure 5.59: Annual solid waste quantities for the Project Area(m^3)-cumulative

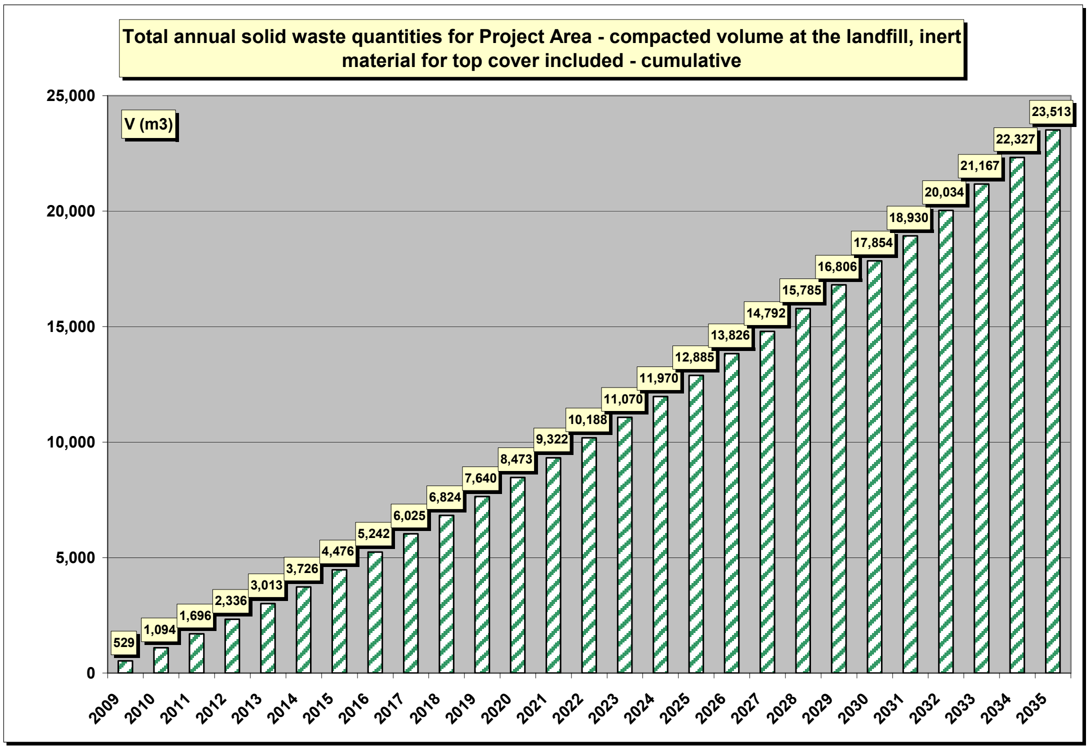


Figure 5.60: Average daily solid waste quantities for the Project Area (t/day)

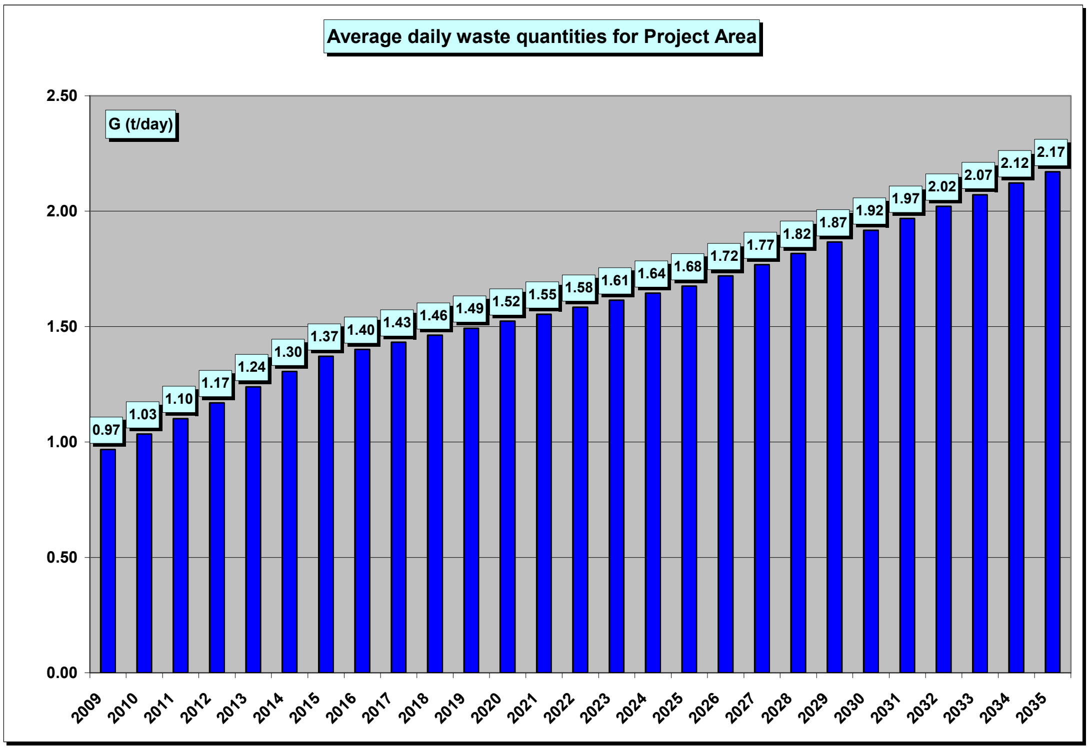
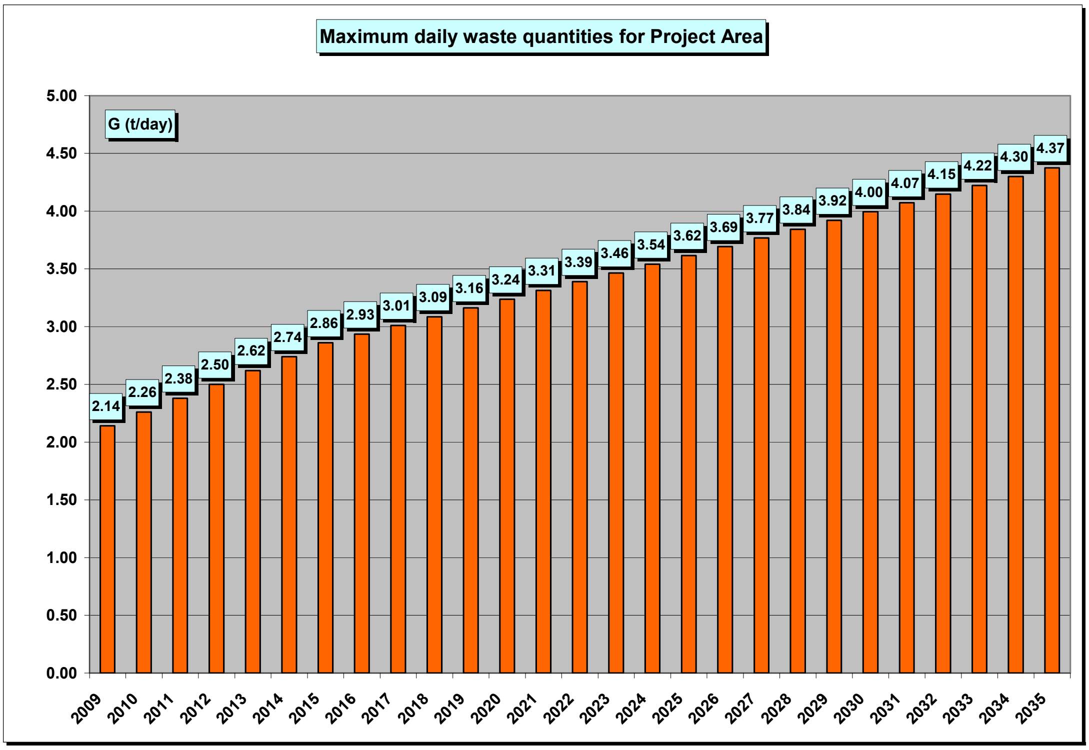


Figure 5.61: Maximum daily solid waste quantities for the Project Area (t/day)



Based on the previous figure, the following conclusions can be made regarding the projection of solid waste quantities in the Project Area:

- At the end of the project period annual solid waste quantity is estimated at 792 t/year;
- At the end of the project period the average daily solid waste production is estimated at 2.17 t/day;
- At the end of the project period the maximum day solid waste generation is estimated at 4.37 t/day;
- The total required volume for land-filling of solid waste till the end of the project period for the whole Project Area is app. 23.500 m^3, including inert material for top covering. This volume is required provided that no source separation or recycling is exercised. In case that source separation or recycling is introduced (PET, plastic foils, paper, metal, etc.) the requirement for landfill volume can be significantly reduced.
- Based on the maximum daily waste quantities, the whole of the Project Area can be served by a single compactor truck of 13m3, with service frequency from 3 times weekly (low season) to once daily (high season);

5.5.4 Proposed solid waste management

Based on the information gathered from the PUC, municipality, site reconnaissance and available studies and technical documentation, it may be concluded that two possible realistic waste management scenarios for the municipality of Surdulica, including the Project Area under consideration (covering rural areas and planned tourist developments in the zone of Vlasina lake):

1. Regional option – assuming full implementation and creation of the Regional Solid Waste Management Scheme, as outlined in the Feasibility Study (reference documentation 20) and in accordance with the National Solid Waste Management Strategy;
2. Municipal option – in a short run developing local solid waste management scheme with upgrade of the local landfill, improvement of the existing collection equipment and other measures, and if and when the abovementioned Regional Scheme becomes reality, joining the Regional Solid Waste Management Scheme.

Both options assume full compliance with the relevant EU and Serbian regulations and implementation of the required environmental standards related to protection of the environment and human health.

In fact, so called municipal option can be seen as the initial phase of significant improvement of the existing solid waste management practices, leading ultimately to implementation of the full-scale Regional Scheme. The regional scheme has been outlined earlier in the report. This section presents elements of the so called municipal option that would introduce and reinforce elements of contemporary solid waste management practices in the Project Area and in the municipality of Surdulica.

In order to be able to cover with solid waste collection services entire project area it is necessary to provide additional solid waste collection equipment and vehicle, as shown in the following table.

Provision of additional solid waste collection equipment in the Project Area promotes source separation introducing separate containers for PET and paper.

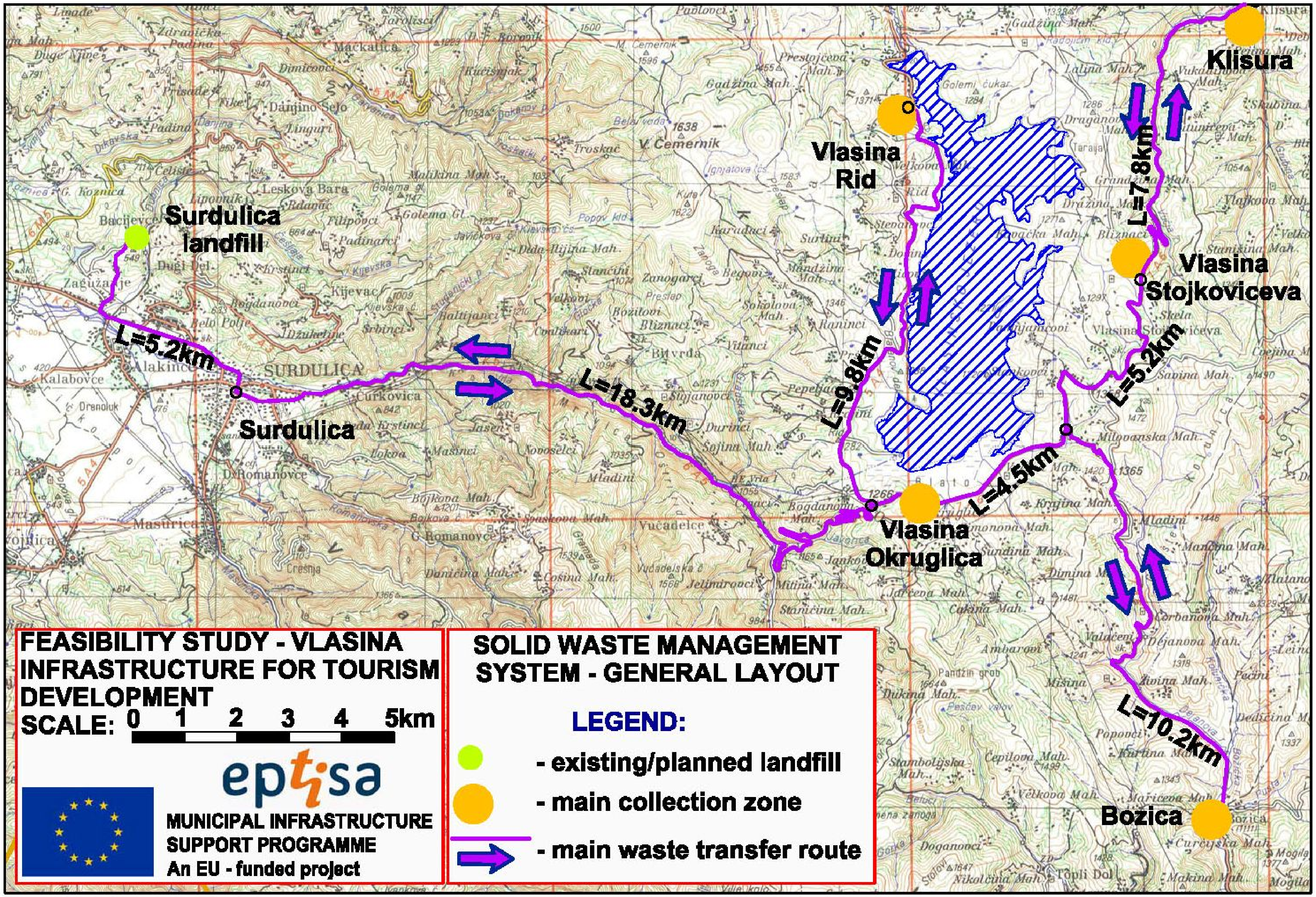
Table 5.77: Required waste collection equipment

	Unit of measure	Quantity
Collection vehicles		
Compactor truck of 13m^3	pieces	1
Containers		
Of 1.1 m^3 with pertaining plateau of mass concrete	pieces	140
Of 1.0 m^3 for PET	pieces	15
Of 2.0 m^3 for paper	pieces	25
Bins 120 l	pieces	1200
Total collection equipment		

So called municipal solid waste management option assumes full development of the municipal sanitary landfill Surdulica, as described in the project designs, reference documentation 19. If solid waste collected in the considered Project Area is to be disposed at the new municipal sanitary landfill its operational lifetime would be between 12 and 15 years, depending on the scenario of increase of solid waste generation. Investment cost for the development of the municipal sanitary landfill for Surdulica is estimated at € 1.3 M (reference documentation 19). Construction of the municipal sanitary landfill is not part of this study.

The layout of the proposed solid waste management scheme is presented in the following figure. The municipal solid waste management scheme would be transformed into the regional solid waste management scheme, as described earlier, when appropriate arrangements and financing of the regional scheme become realistic.

Figure 5.62: Proposed Solid Waste Management Scheme



5.5.5 Cost estimate

This section presents investment cost for additional solid waste collection equipment in the Project Area.

Table 5.78: Required waste collection equipment – investment costs

	Unit of measure	Quantity	Unit cost (€)	Total cost (€)
Collection vehicles				
Compactor truck of 13m^3	pieces	1	110,000	110,000
Tractor	pieces	1	20,000	20,000
Containers				
Of 1.1 m^3 with pertaining plateau of mass concrete	pieces	140	250+100	49.000
Of 1.0 m^3 for PET	pieces	15	300+100	6,000
Of 2.0 m^3 for paper	pieces	25	350+100	11,250
Bins 120 l	pieces	1200	25	30,000
Contingencies				20,000
Total collection equipment				246,250

Therefore the abovementioned investment costs refer to the improvement of the collection equipment (vehicles and containers) only, whereas necessary investments of general importance for the municipality or region are to be implemented and financed through other projects.

As explained earlier in the report, general solid waste management orientation could imply two potential approaches:

- Municipal – improvement and extension of the existing municipal landfill site and its transformation into a sanitary landfill – estimated investment costs € 1.3 M;
- Regional – introduction of a regional solid waste management scheme for all municipalities of Pčinja administrative district (Vranje, Vladičin Han, Surdulica, Bujanovac, Preševo, Trgovište and Bosilegrad) with the focal point at the Vranje landfill including a separation line – estimated investment costs app. € 16.5 M;

In a short run, with regard to solid waste management for the considered Project Area it is logical to join the municipal scheme, which is already in operation and which serves Surdulica municipality.

In a long run however, depending on the implementation schedule, financing and corresponding inter-municipal arrangements, Surdulica municipality, including the Project Area, may opt for the regional scheme.

Whichever approach is selected, operational costs of running the solid waste management would consists of two basic components:

- Operational costs (including fixed and variable) that are related to current solid waste management practices; including collection and dumping at the existing landfill;
- Additional operational costs required for planned introduction of contemporary, environmentally sound solid waste management practices including: efficient collection, selection at source, transportation to transfer station (where applicable), transfer to a regional landfill, operation of a separation line, sanitary land-filling;

Current unit operational costs (per unit weight of waste) are obtained from the questionnaire produced by the PUC Vodovod, Surdulica, which is current solid waste operator in the municipality.

Namely, based on the said questionnaire, current unit operational costs for solid waste collection and disposal in Surdulica municipality are:

- For 2007 - 4,810 RSD/ton
- For 2008 - 4,840 RSD/ton

These costs are fairly consistent with the average regional operational costs for solid waste collection and disposal (5,000 RSD/ton), as calculated in the Regional Solid Waste Management Feasibility Study (reference documentation 20). Unit cost of 5,000 RSD/ton is adopted as a baseline for further analysis in this study.

On the other hand the basis for calculation of tipping fees has been adopted in accordance with a comprehensive analysis carried out in the reference documentation 20, as shown in the following table.

Table 5.79: Average tipping fee landfill and transfer station

Unit cost of waste	Units RSD/ton €/ton	2010 1,906 22	2011 1,975 23	2012 2,016 23	2013 1,832 20	2014 1,806 20	2015 1,786 19	2016 1,782 19	2017 1,779 19	2018 2,166 22	2023 2,740 26
Increase Proposed average tipping fee	**RSD/ton €/ton**	1,572 18	4.8% 1,648 19	4.6% 1,723 19	4.8% 1,806 20	5.0% 1,897 21	5.0% 1,992 21	5.0% 2,091 22	5.0% 2,196 23	5.0% 2,305 24	5.0% 2,942 28