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1. HYDROLOGICAL INVESTIGATION

1.1 CLIMATE

1.1.1 Climate zone

The climate of the project area is subject to the influence of the Adriatic and Ionian Sea. In Albania there are four climatic zones, divided into 13 subzones, where climatic fluctuations differ on relatively small changes. This climatic diversity influences the unbiased biological diversity of the country. The project is located in the southern Mediterranean mountain climatic zone, according to the climatic division of Albania, characterized by mild and wet winters and hot and dry summers.

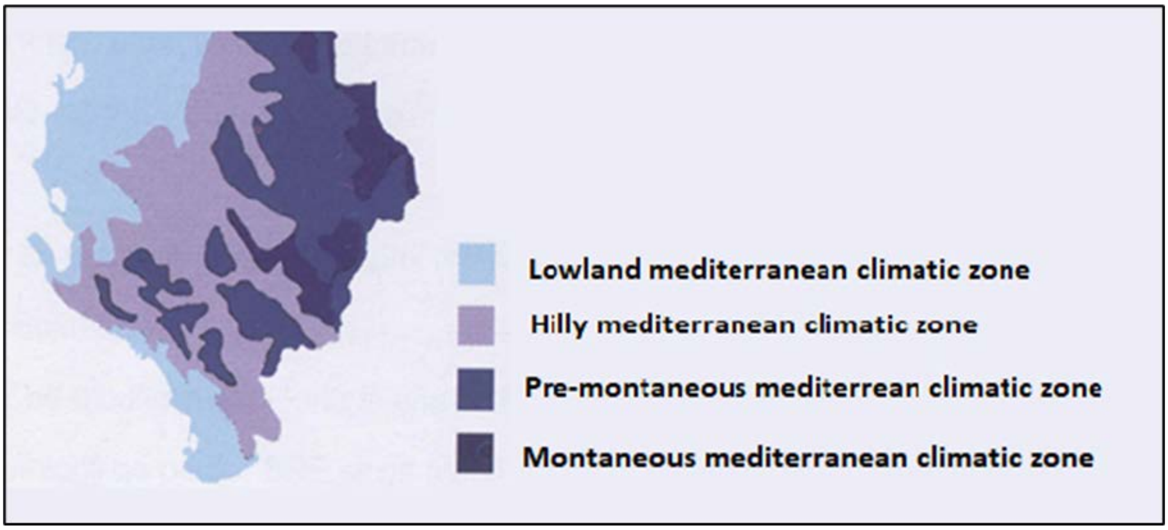

Figure 1-1 Climate zoning south Albania

1.1.2 Temperature

As mentioned above, the geographic position and impact of the sea are reflected in the climatic conditions of the area, and especially in the air temperature regime. During the summer the sea breeze can be quite refreshing, while in winter it can mild the cold. Consequently, for the coldest month (January) the average temperature is around 9.5 ° C and i July is about 25° C, while the maximum average temperature varies from 13.0 ° C in January to 30° C in July-August.

The minimum average temperatures range from 6.2 ° C in January to 20.0 ° C in July. Temperature extremes in this area are relatively mild. Thus, the absolute minimum air temperature value for the entire recording period is -4 ° C (4 January 1979), while the absolute maximum value reaches + 42.1 ° C (26 July 1987). The temperature annual performance of the temperature mode indicators is shown in the figure.

Table 1-1 Average temperature

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Daily Max Av	13	14	16	19	23	27	30	30	27	23	19	15	21.3
Daily Av	9.5	10	12	15	19	22	25	24.5	22	19	15	11.5	17
Daily Min Av	6	6	8	10	14	17	19	19	16	14	11	8	12.3

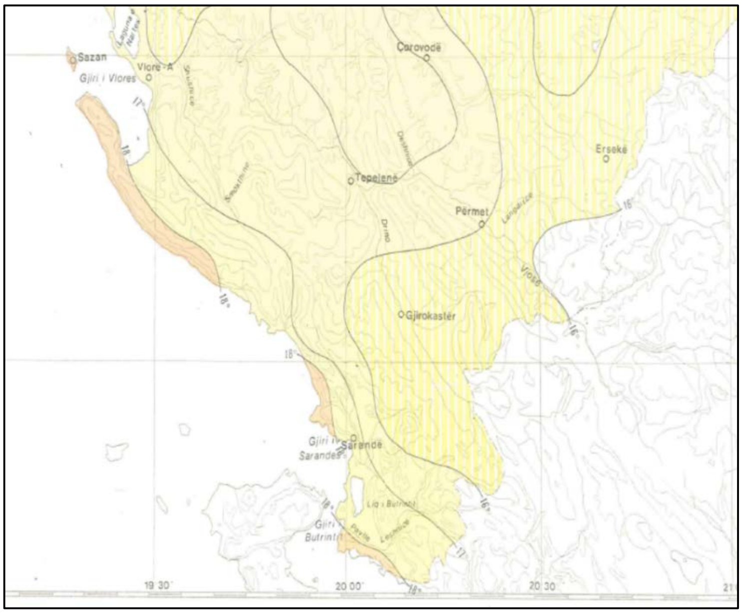

Figure 1-2 Average temperature south Albania (project area 17-18 °C)

1.1.3 Solar radiation

The data show that the annual amount of total solar radiation reaches the value of 1540.3 kW / m^2. The annual performance of te solar radiation is given in the figure below, which shows that its highest value is reached in July (216.5 kWh / m^2) and the lowest in December (52.1 kWh / m^2). The average daily radiation in the area is 4-4.5 kWh/ m^2.

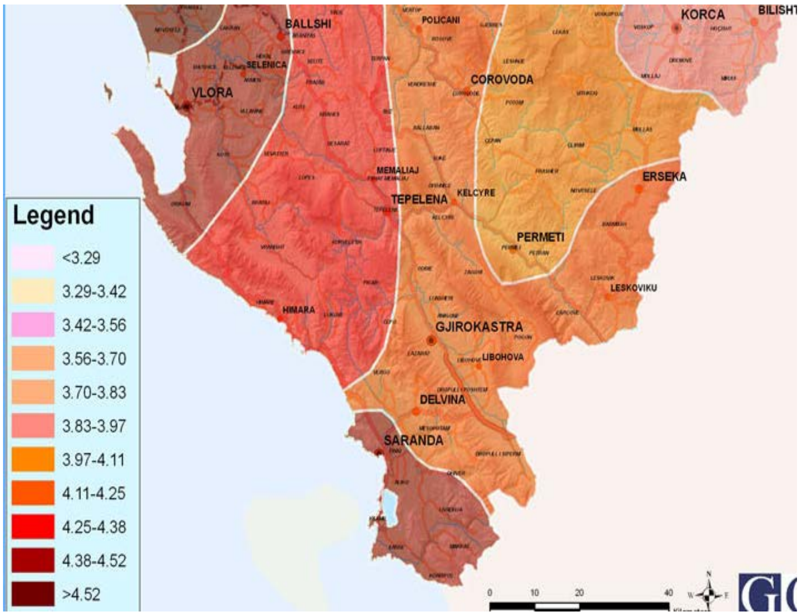

Figure 1-3 Average daily radiation south Albania

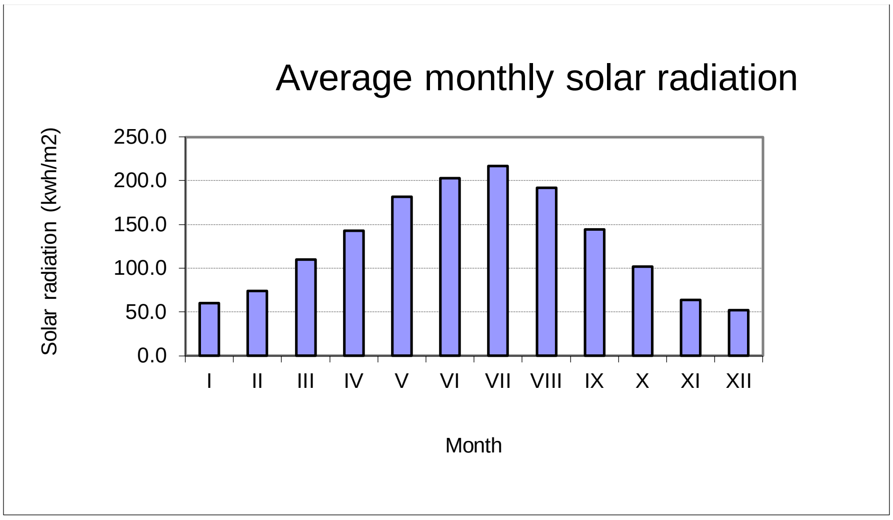

Figure 1-4 Monthly solar radiation

This area, as in the case of solar radiation, is characterized by a large number of hours of sunshine. On average during the year there are 2761 hours of sunshine, with the highest value in July with 371.5 hours and the lowest in December with 131.3 hours.

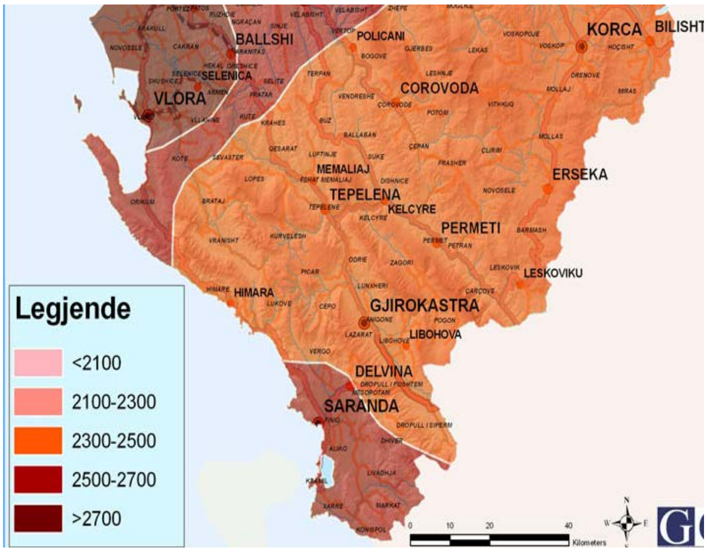

Figure 1-5 Average sunshine hours south Albania

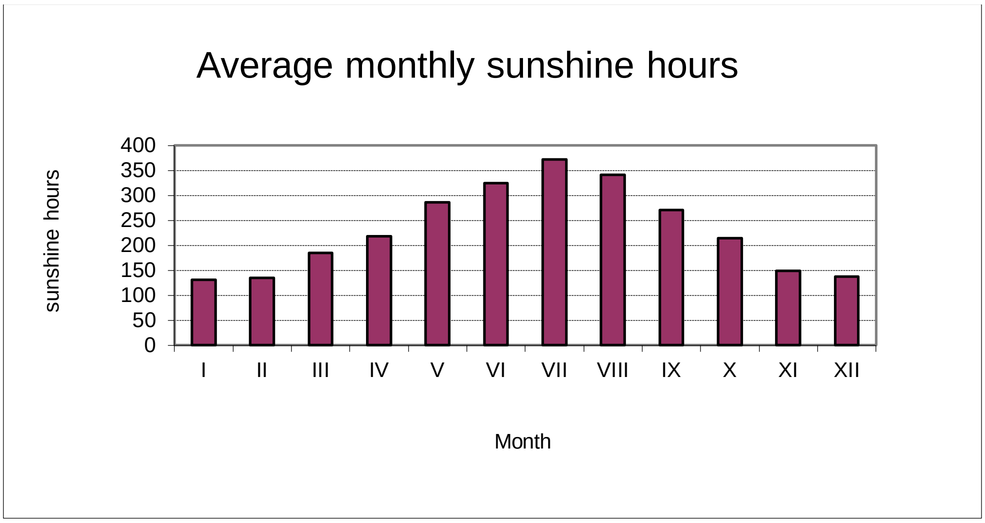

Figure 1-6 Monthly sunshine hours

1.1.4 Humidity

The average relative humidity during the year is almost at the same levels. It ranges from 63% (July) to 68% (October, November), while the average annual value of humidity is 65%.

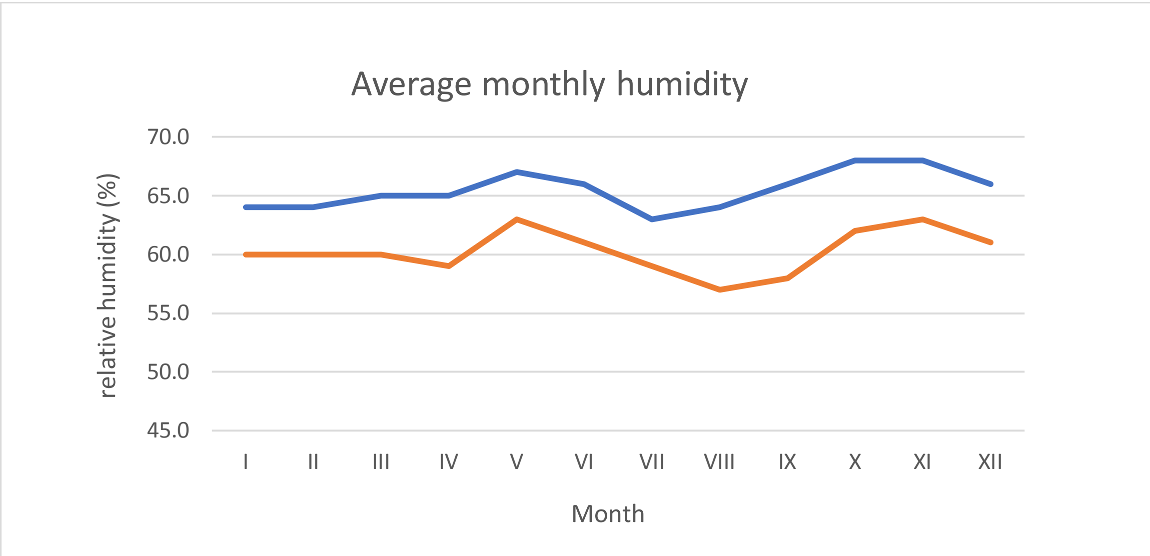

Figure 1-7 Average monthly humidity & average monthly humidity at 2:00 PM

The curve that presents the average humidity at 2:00 PM presents the same performance as that of the average humidity, but with values are about 5% lower. Specifically, the values of this index reach the minimum in July and August (about 57%) and the maximum in November (about 63%).

1.1.5 Wind

In addition to the climate elements presented above, the wind regime is of special importance for the formation of climatic conditions as well as for practical purposes. To describe the wind regime, for lack of regular observations, we will rely on the observations of Vlora Airport meteorological station, which we think can serve as a guide.

From the analysis of the annual wind rose it results that calm is encountered in 42.5% of the observations. Along with it in the area is observed the predominance of east winds (with a frequency of 13.8%) and northwest winds (with a frequency of 11.5%). East winds appear as the first prevailing direction during the cold half of the year (October-March period) with frequencies ranging from 13-23% (See figures below). In April it starts to appear NW wind. Initially NW direction appears as the second predominant direction and then with a frequency almost equal to the east winds (10.3-17.2%) during the summer months.

The average wind speed varies from 2.0 m / s (September) to 2.8 m / s (February). The average annual value reaches 2.6 m / s. The highest average speed throughout the year is observed according to the direction south S (7.7 m / s, frequency 7.2%) and SE (5.8 m / s, frequency 3.7%). The maximum speed recorded has reached 40 m / s during the winter months. Regarding Radhima metereological station (near the project area) , in the short series of data that it has, it can be said that during this period, with except July and August, maximum speeds > 40m / sec were observed in all the other months.

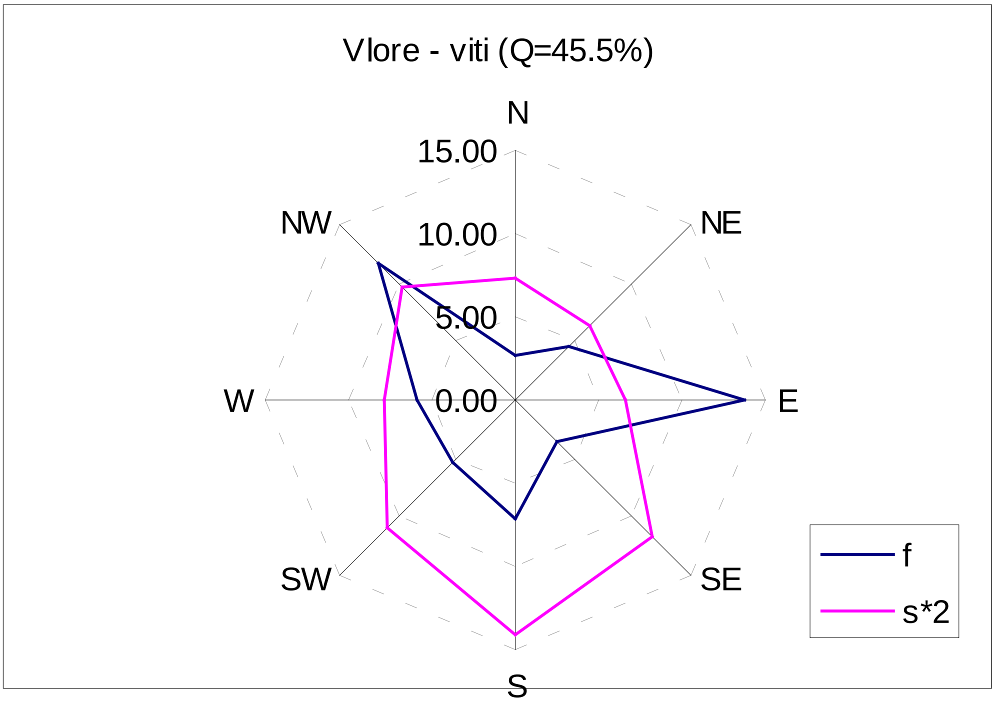

Figure 1-8 Wind rose, Vlora Airport meteorological station

**(Q = percentage of time where wind veloxity V=0m/s)*

1.1.6 Rainfall

Due to the wide cyclone activity, the most frequent rainfall is observed in the colder half period of the year, and the lowest ones in its warm period. This is a typical rainfall regime of a Mediterranean region. This area is characterized by relatively low annual rainfall. The precipitations in the area are almost completely rainfall events. Snowfall is rare in the altitude less than 600 masl. The average annual rainfall in the area varies from 950 mm at its lower part up to 1400-1500 mm in the uplands surrounding it. The area in which the project is supposed to be developed has an annual average amount of rainfall ranging from 1200 mm to 1500 mm/year.

Table 1-2 Average rainfall

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Average rainfall (mm)	120	106	92	79	54	28	9	26	32	116	192	141	995

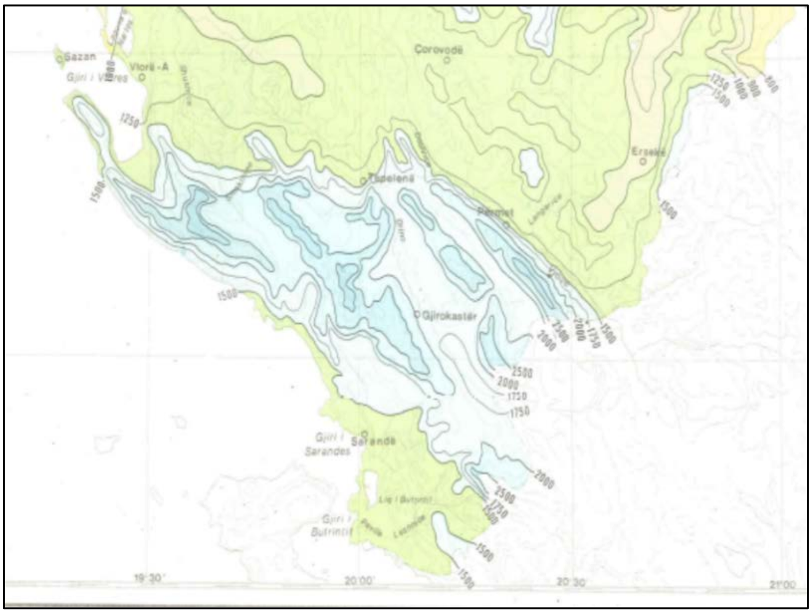

Figure 1-9 Map of annual average rainfall south Albania

On average in this area, there is an increase of annual rainfall of about 110mm for every 100m altitude. The number of days with ≥1.0 mm precipitation ranges from 1.6 days (July) to 9.1 days (November). During the year there is an average of 76.9 days with rainfall ≥1.0mm.

1.2 WATER STREAMS

There are several streams in the area crossing the alignement of the project. These streams are typical mountain creeks. They are characterized by very steep slope and their bed is composed by large size boulders to coarse gravel. Also typical for all of the streams in the area is that they are dry during non-rainy days. They get activated during moderate and heavy rains or during continuous precipitations in general. Below is given a map of the nearby streams and their respective watersheds.

Figure 1-10 Shatait stream, Downstream view (tha photo taken during a sunny day shows a dry bed)

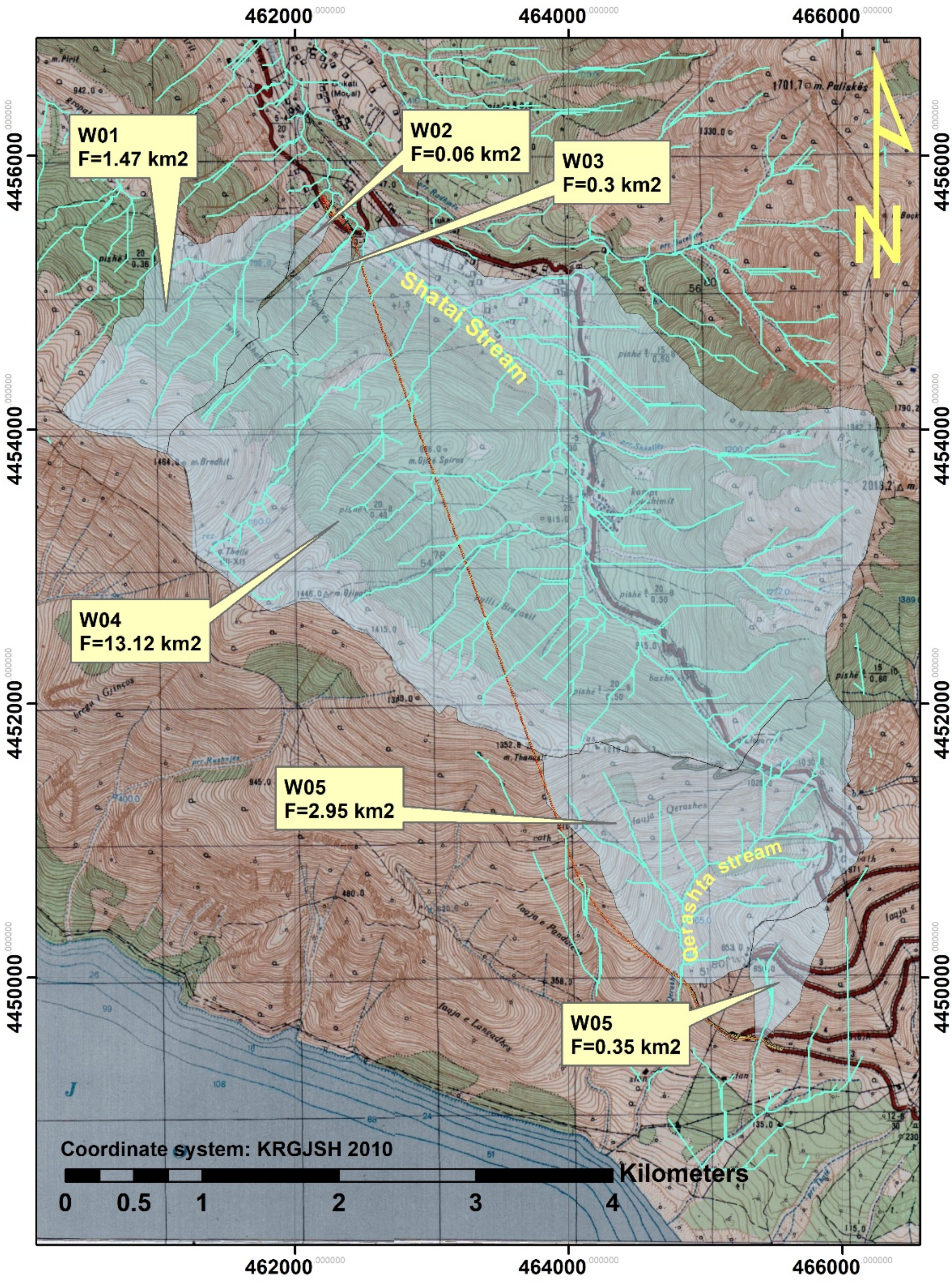

Figure 1-11 Map of streams in the surrounding area

2. FLOOD FREQUENCY

The flood frequency used to design or review culverts and bridges shall be based on:

- the level of risk associated with failure of the crossing, increasing backwater, or redirection of the floodwaters.
- an economic assessment or analysis to justify the flood frequencies greater or lesser than the minimum flood frequencies listed herein.
- location of mapped floodplains.
- MoPWTT design criteria (see Table below)

Table 2-1 Summary of hydraulic design criteria for culverts and bridges

MoPWTT STRUCTURE CLASS ***	DRAINAGE AREA km^2	DESIGN FREQUENCY (year)	CHECK FREQUENCY (year)	BACKWATER m	MINIMUM ** FREEBOARD m
Minor	< 2.50 (no established watercource)	25	-	-	0.3
Small	< 2.50	50	100	-	0.3
Intermediate*	≥ 2.50 < 25.9	100	500	≤0.3	0.3
Large*	≥ 25.0 < 2500	100	500	0.3	0.6

* *The designer shall also consider bridge alternative for this class when area > 2.50 km^2*

** *Freeboard is defined as the vertical distance between the design water surface and the upstream control such as the low point of the roadway edge, sill of a building or other controlling element.*

3. PEAK DISCHARGE EVALUATION

3.1 Rational Method

The rational method is recommended for estimating the design storm peak runoff for small areas. Even though it has frequently come under criticism for its simplistic approach, no other drainage design method has received such widespread use.

Some precautions should be considered when applying the rational method.

• The first step in applying the rational method is to obtain a good topographic map and define the boundaries of the drainage area in question. A field inspection of the area should also be made to determine if the natural drainage divides have been altered.

• In determining the runoff coefficient C value for the drainage area, thought should be given to future changes in land use that might occur during the service life of the proposed facility that could result in an inadequate drainage system.

• The charts, graphs and tables included in this section are not intended to replace reasonable and prudent engineering judgment which should permeate each step in the design process.

3.1.1 Characteristics of the rational

1. The rate of runoff resulting from any rainfall intensity is a maximum when the rainfall intensity lasts as long or longer than the time of concentration. That is, the entire drainage area does not contribute to the peak discharge until the time of concentration has elapsed. This assumption limits the size of the drainage basin that can be evaluated by the rational method. For large drainage areas, the time of concentration can be so large that constant rainfall intensities for such long periods do not occur and shorter more intense rainfalls can produce larger peak flows

2. The frequency of peak discharges is the same as that of the rainfall intensity for the given time of concentration. Frequencies of peak discharges depend on rainfall frequencies, antecedent moisture conditions in the watershed, and the response characteristics of the drainage system. For small and largely impervious areas, rainfall frequency is the dominant factor. For larger drainage basins, the response characteristics control or drainage areas with few impervious surfaces (less urban development), antecedent moisture conditionsusually govern, especially for rainfall events with a return period of 10 years or less.

3. The fraction of rainfall that becomes runoff (C) is independent of rainfall intensity or volume. The assumption is reasonable for impervious areas, such as streets, rooftops and parking lots. For pervious areas, the fraction of runoff varies with rainfall intensity and the accumulated volume of rainfall. Thus, the art necessary for application of the rational method involves the selection of a coefficient that is appropriate for the storm, soil and land use conditions. Many guidelines and tables have been established, but seldom, if ever, have they been supported with empirical evidence.

4. The peak rate of runoff is sufficient information for the design.

Modern drainage practice often includes detention of urban storm runoff to reduce the peak rate of runoff downstream. With only the peak rate of runoff, the rational method severely limits the evaluation of design alternatives available in urban and in some instances, rural drainage design.

3.2 Equation

The rational formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area, runoff coefficient and mean rainfall intensity for a duration equal to the time of concentration (the time required for water to flow from the most remote point of the basin to the location being analysed). The rational formula is expressed asfollows:

$$Q = 0.00278 * C * I * A$$

Where:
Q = maximum rate of runoff, m3/s;
C = runoff coefficient representing a ratio of runoff to rainfall;
I = average rainfall intensity for a duration equal to the time of concentration, for a selected return period, mm/h;
A = drainage area tributary to the design location, ha.

3.2.1 Time of Concentration

The time of concentration is the time required for water to flow from the hydraulically most remote point of the drainage area to the point under investigation. Use of the rational formula requires the time of concentration (tc) for each design point within the drainage basin. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity (I). This method shall be used for the rational method.

Note: Under certain circumstances, where tributary areas are very small or completely paved, the computed time of concentration would be very short. For design purposes the minimum time of concentration for paved areas shall be 5 minutes and 10 minutes for grassed areas.

In this case the Kirpich Method would be appropriate because it is used for small drainage basins that are dominated by channel flow.

$$t_c = 0.0195 \left(\frac{L^{0.77}}{S^{0.385}} \right)$$

$t_c - concentration\ time\ (\mathrm{min})$
$L - stream\ length\ (\mathrm{m})$
$S - slope$

3.2.2 Rainfall Intensity

The rainfall intensity (I) is the average rainfall rate mm/h for a duration equal to the time of concentration for a selected return period. Once a particular return period has been selected for design and a time of concentration calculated for the drainage area, the rainfall intensity can be determined from Rainfall-Intensity-Duration curves.

The maximum rainfall data will be represented by VLORA A meteorological station. Maximum rainfall intensity with different probability is taken from this station in: "THE MANUAL OF MAXIMUM RAINFALL WITH DIFFERENT PROBABILITIES" published by the "Albanian Hydrometeorology Institute" 1985. Below are given the IDF curves.

Table 3-1 Summary table of maximum with different probabilities (VLORA A station)

Duratin [hours]	Probability [%]					
	1	2	5	10	20	50
	Intensity [mm/h]					
24	7.5	6.8	5.9	5.2	4.4	3.3
12	13.8	12.4	10.5	9.1	7.6	5.3
6	25.5	22.7	19.0	16.2	13.2	8.7
2	49.0	43.5	36.5	31.0	25.5	17.0
1	76.0	68.0	57.0	49.0	40.0	27.0
0.50 (30 minutes)	102.0	92.0	78.0	66.0	56.0	38.0
0.33 (20 minutes)	130.3	118.2	100.0	84.8	72.7	48.5
0.1667 (10 minutes)	216.0	198.0	174.0	108.0	90.0	66.0

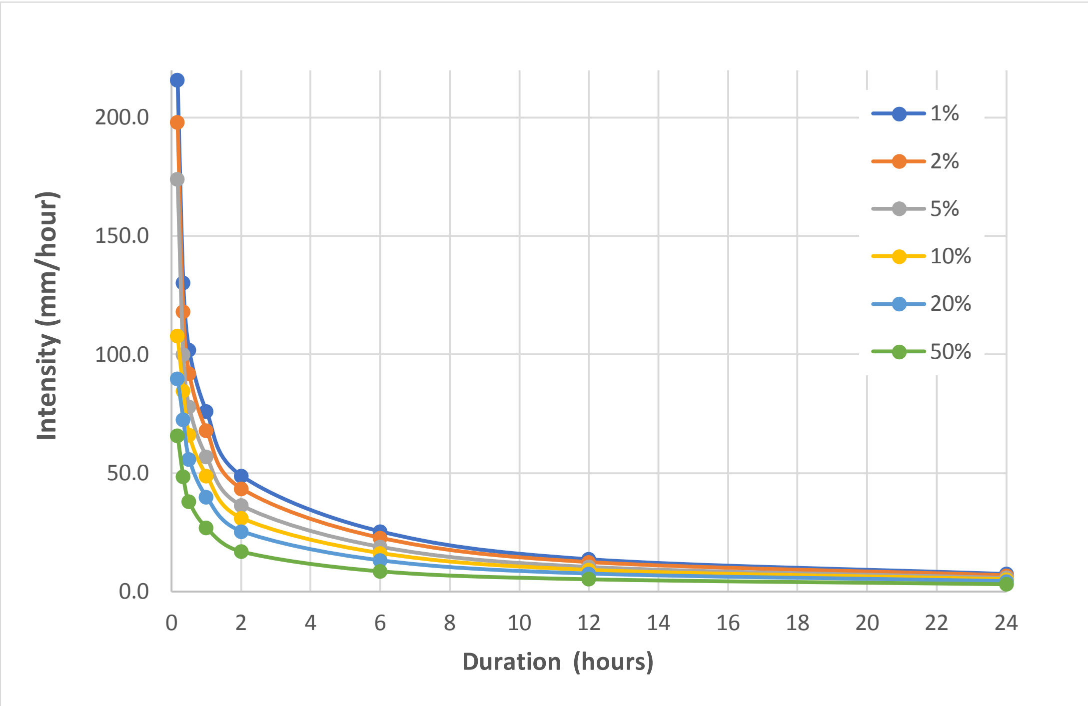

Figure 3-1 IDF curves VLORA A station

3.2.3 Runoff Coefficient

The runoff coefficient C is the variable of the rational method least susceptible to precise determination and requires judgment and understanding on the part of the designer. While engineering judgment will always be required in the selection of runoff coefficients, a typical coefficient represents the integrated effects of many drainage basin parameters, the following discussion considers only the effects of soil groups, land use and average landslope.

Methods for determining the runoff coefficient are presented based on hydrologic soil groups and land slope, land use and a composite coefficient for complex watersheds.

Table below gives the recommended coefficient C of runoff for pervious surfaces by selected hydrologic soil groupings and slope ranges. From this table the C values for non-urban areas such as forest land, agricultural land, and open space can be determined. Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Infiltration is the movement of water through the soil surface into the soil.

Based on infiltration rates soils have been divided into four hydrologic soil groups as follows:

Group A. Soils having a low runoff potential due to high infiltration rates. These soils consist primarily of deep, well drained sands and gravels.

Group B. Soils having a moderately low runoff potential due to moderate infiltration rates. These soils consist primarily of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.

Group C. Soils having a moderately high runoff potential due to slow infiltration rates. These soils consist primarily of soils in which a layer exists near the surface that impedes the downward movement of water or soils with moderately fine to fine texture.

Group D. Soils having a high runoff potential due to very slow infiltration rates. These soils consist primarily of clays with high swelling potential, soils with permanently high-water tables, soils with a claypan or clay layer at or near the surface and shallow soils over nearly impervious parent material.

Table 3-2 Recommended coefficient of runoff for pervious surfaces by selected Hydrologic soil Groupings and slope ranges

Slope ranges	A	B	C	D
Flat (0 - 1%)	0.04-0.09	0.07-0.12	0.11-0.16	0.15-0.20
Average (2 - 6%)	0.09-0.14	0.12-0.17	0.16-0.21	0.20-0.25
Steep (over 6%)	0.13-0.18	0.18-0.24	0.23-0.31	0.28-0.38

Considering the site condition, the hydrologyc soil type fits Group B and slope ranges are more than 6%, the runoff can be **accepted C = 0.28-0.3.**

4. CULVERTS

4.1 Culvert identification

Culvert identification process is done in two steps. First it is checked the topographical survey for the existing culverts. Considering the existing culvert alignment are placed also the new ones. After that, GIS tool is used to identify the stream that passes through them and its watershed. Additional culverts are added during the road design process in order to drain various parts of the intersections. Also, additional culverts are assigned at regular intervals in order to drain the road platform. The evaluation list of culverts was then analyzed with HY-8 software.

4.2 HY-8 software

HY-8 automates culvert hydraulic computations. As a result, a number of essential features that make culvert analysis and design easier.
HY-8 enables users to analyze:
• The performance of culverts
• Multiple culvert barrels at a single crossing as well as multiple crossings
• Roadway overtopping at the crossing and
• Develop report documentation in the form of performance tables, graphs, and key information regarding the input Variables.

HY-8 performs culvert hydraulic calculations based on the input minimum, design, and maximum discharge values. Calculations comprising the performance curve are made for ten equal discharge intervals between the minimum and maximum values. A user may input a narrower range of discharges in order to examine culvert performance for a discharge interval of special interest.
MINIMUM DISCHARGE
Lower limit used for the culvert performance curve. Can be edited to a number greater than '0'.
DESIGN DISCHARGE
Discharge for which the culvert will be designed. Always included as one of the points on the performance curve.
MAXIMUM DISCHARGE
Upper limit used for the culvert performance curve.

When defining the roadway data for the culvert, the following parameters are required:
• Roadway Profile
• Roadway Station
• Crest Length
• Crest Elevation
• Roadway Surface
• Top Width

The roadway elevation can be either a constant or vary with station. An initial roadway station may be defined by the user or left at the default of 0.0. The stationing is used to position culverts along the length of the roadway profile when choosing the Front View option.
The roadway surface may be paved or gravel, or an overtopping discharge coefficient in the weir equation may be entered. The user may select a paved roadway surface or a gravel roadway surface from which the program uses a default weir coefficient value. If input discharge coefficient is selected, the user will enter a discharge coefficient between 2.5 and 3.095.

The values entered for the crest length and top width of the roadway have no effect on the hydraulic computations unless overtopping occurs.

Tailwater Data

HY-8 provides the following options for calculating the tailwater rating curve downstream from a culvert crossing:
• Channel Shape
• Irregular Channel
• Rating Curve
• Constant Tailwater Elevation
Uniform depth is used to represent tailwater elevations for both a defined channel shape and an irregular channel. The cross section representing these two options should be located downstream from the culvert where normal flow is assumed to occur (downstream from channel transitions, for example). The calculated water surface elevations are assumed to apply at the culvert outlet.

Channel Shape

There are three available channel shapes to define the downstream tailwater channel: rectangular, trapezoidal, and triangular. When selecting a channel shape the input window adjusts to display only those parameters required for the defined shape. When defining a channel shape, the following channel properties are required for analysis: Bottom Width — Width of channel at downstream section, shown in drawing below. Side Slope (H:V) (_:1) — This item applies only for trapezoidal and triangular channels. The user defines the ratio of Horizontal/Vertical by entering the number of horizontal units for one unit of vertical change. Channel Slope — Slope of channel in m/m or ft/ft. If a zero slope is entered, an error message appears upon exiting the input data window. The user must enter a slope greater than zero before the crossing may be analyzed. Manning's n — User defined MANNING'S roughness coefficient for the channel. Channel Invert Elevation — User must enter elevation. Program will show actual barrel #1 outlet invert elevation.

4.3 Hydraulic conditions

4.3.1 Types of flow control

Inlet and outlet control are the two basic types of flow control defined in the research conducted by the NBS and the FHWA (formerly BPR). The basis for the classification system was the location of the control section. The characterization of pressure, subcritical, and supercritical flow regimes played an important role in determining the location of the control section and thus the type of control. The hydraulic capacity of a culvert depends upon a different combination of factors for each type of control.

a. Inlet Control. Inlet control occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. The control section of a culvert operating under inlet control is located just inside the entrance. Critical depth occurs at or near this location, and the flow regime immediately downstream is supercritical. Figure below shows one typical inlet control flow condition. Hydraulic characteristics downstream of the inlet control section do not affect the culvert capacity. The upstream water surface elevation and the inlet geometry represent the major flow controls. The inlet geometry includes the inlet shape, inlet cross-sectional area, and the inlet configuration.
   For inlet control, the control section is at the upstream end of the barrel (the inlet). The flow passes through critical depth near the inlet and becomes shallow, high velocity (supercritical) flow in the culvert barrel. Depending on the tailwater, a hydraulic jump may occur downstream of the inlet.

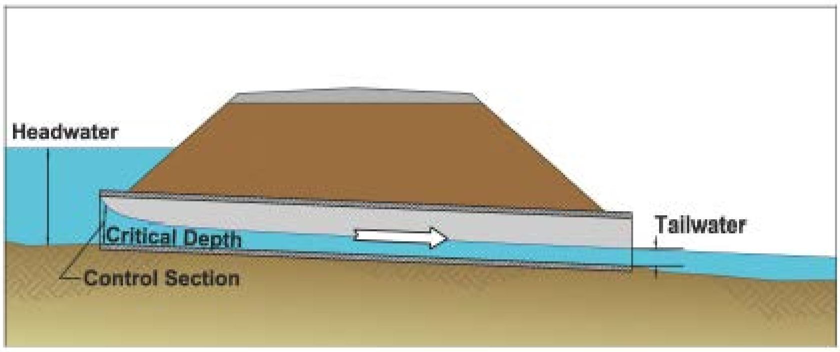

Figure 4-1 Inlet control conditions

b. Outlet Control. Outlet control flow occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section for outlet control flow in a culvert is located at the barrel exit or further downstream. Either subcritical or pressure flow exists in the culvert barrel under these conditions. Figure below shows two typical outlet control flow conditions. All of the geometric and hydraulic characteristics of the culvert play a role in determining its capacity. These characteristics include all of the factors governing inlet control, the water surface elevation at the outlet, and the barrel characteristics

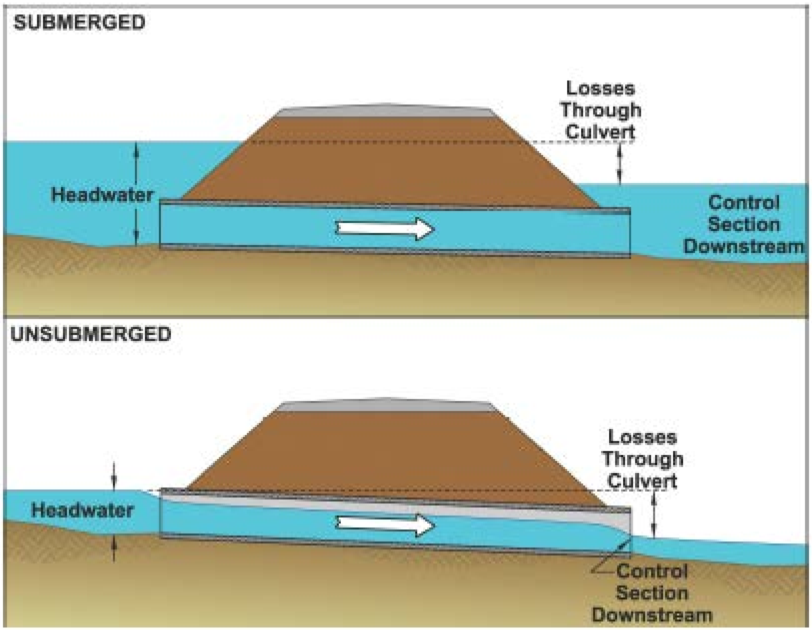

Figure 4-2 Outlet control conditions

The factors distinguish between the geometric properties of the inlet versus the barrel to account for the effect of tapered inlets used on some culverts. For a culvert without a taper the inlet area and shape would be equal to the barrel area and shape. The slope of the culvert is called barrel slope to distinguish it from other slope parameters that may exist at the entrance, such as when a depressed inlet is used. Barrel slope is the primary factor influencing whether or not a culvert will be in inlet or outlet control. In the case of a mitered culvert, the length of the barrel is based on where the crown intersects the fill slope.

Headwater

Energy is required to force flow through a culvert. This energy takes the form of an increased water surface elevation on the upstream side of the culvert. The depth of the upstream water surface measured from the invert at the culvert entrance is generally referred to as headwater depth.

A considerable volume of water may be pounded upstream of a culvert under high fills or in areas with flat ground slopes. The pond which is created may attenuate flood peaks under such conditions, similar to the attenuation caused by a reservoir or lake. Analysis of flood peak attenuation is based on storage routing. Although this decrease in peak discharge may justify a reduction in the required culvert size, this is not a widely used practice in culvert design.

Headwater Factors

• Headwater depth is measured from inlet invert of the inlet control section to the surface of the upstream pool.
• Inlet area is the cross-sectional area of the face of the culvert. Generally, the inlet face area is the same as the barrel area.
• Inlet edge configuration describes the entrance type. Some typical inlet edge configurations are thin edge projecting, mitered, square edges in a headwall, and beveled edge.
• Inlet shape is usually the same as the shape of the culvert barrel. Typical shapes are rectangular, circular, elliptical, and arch. Check for an additional control section if different than the barrel.

Tailwater

Tailwater is defined as the depth of water downstream of the culvert measured from the outlet invert. It is an important factor in determining culvert capacity under outlet control conditions. The amount of tailwater is based on the characteristics of the downstream channel at the given design discharge and is evaluated based on traditional open channel flow calculations, often using normal depth approximations. Increased tailwater may be caused by an obstruction in the downstream channel, such as another highway crossing with a bridge or culvert, the confluence with another channel, the existence of a reservoir or beaver dam, etc. In such cases, backwater calculations from the downstream control point are required to precisely define tailwater. High tailwater alone is capable of making a culvert operate under outlet control, when it would otherwise be under inlet control.

Outlet Velocity

Since a culvert often constricts the available channel area, flow velocities in the culvert may be higher than in the channel. These increased velocities can cause streambed scour and bank erosion in the vicinity of the culvert outlet. Minor problems can occasionally be avoided by increasing the barrel roughness. Energy dissipaters and outlet protection devices are sometimes required to avoid excessive scour at the culvert outlet. When a culvert is operating under inlet control and the culvert barrel is not operating at capacity, it is often beneficial to flatten the barrel slope or add a roughened section to reduce outlet velocities.

4.4 Culvert size assignment

A complete list is given below, which contains:

- Culvert ID and progressive number
- Culvert position along the motorway (PK)
- Area of the drainage basin and 50 and 100 return period discharge
- Section type and size
- Inlet and outlet elevation
- Length and slope

Table 4-1 Main parameters of hydraulic structures

Watershed	Station	ID	Concentration time	Area(m2)	Return period (years)	Intensity (mm/h)	Discharge (m3/s)	Diameter	CUE (m)	CDE (m)	SLOPE %	LENGTH (m)
W02	0+142.3	CN/1	10 min	60000	25	105	0.49	Ø1000	396.64	393.89	8.57	32.1
N/A	0+026 (NJ)	CN/2	10 min	3500	25	100	0.03	Ø600	395.64	393.14	11.57	21.6
W03	0+314	CN/3-1	30 min	300000	25	100	2.33	Ø1000	407.62	404.64	14.61	20.4
W03	0+314	CN/3-2	30 min	300000	25	100	2.33	Ø1000	404.44	404.32	1.00	12
N/A	0+360	CN/4	10min	3500	25	100	0.03	Ø600	410.25	408.3	10.54	18.5
N/A	0+235 (NJ)	CN/5	10 min	3500	25	100	0.03	Ø600	408.3	408.2	1.06	9.4
W05	N/A	Bridge 1	60 min	2950000	50	68	15.60	N/A	N/A	N/A	N/A	N/A
N/A	6+708	CS/1	10 min	50000	25	100	0.39	Ø600	369.12	368.93	1.09	17.5
N/A	6+868	CS/2	10 min	50000	25	100	0.39	Ø600	364.28	364.13	0.97	15.5
N/A	6+993	CS/3	10 min	50000	25	100	0.39	Ø600	356.61	356.41	1.03	19.4
N/A	7+087	CS/4	10 min	50000	25	100	0.39	Ø600	351.05	348.09	8.92	33.2
W06	N/A	Bridge 2	30 min	350000	25	100	2.72	N/A	71.41	70.75	2.70	24.4

**CUE: Culvert upstream elevation*

**CDE: Culvert downstream elevation*

5. BRIDGES

The project includes two considerably large single spam bridges 110 m and 40m. Most importantly on the hydraulic point of view is that the elevation of bridges relative to the bed level of the streams is very **high 37.7m and 15.8m.** Considering these factors, it is sure that the flow is not affected by the briges or viceversa. To show this, it is performed a classic Manning open channel flow analysis.

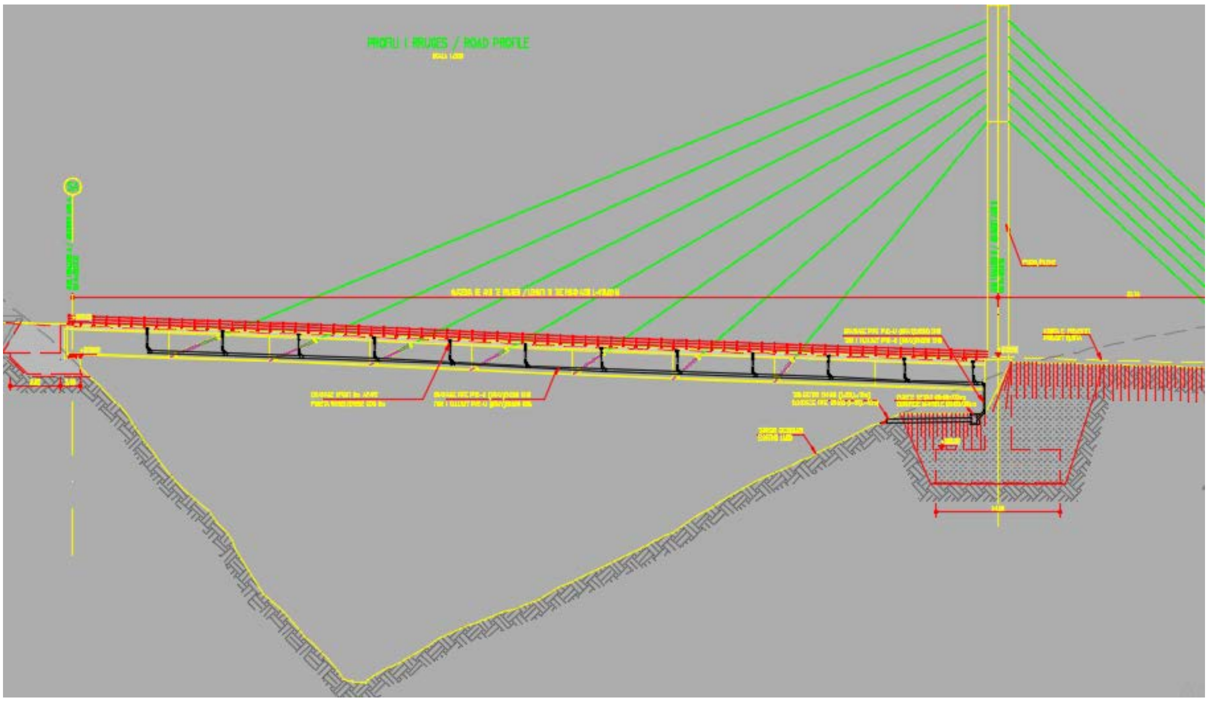

Figure 5-1 Bridge 1

Table 5-1 Water depth at the stream Bridge 1 for return period RP=50 years

Parameter	Vlera	Unit
Side slope bank Z_1, (H:V)	0.8	
Side slope bank Z_2, (H:V)	2	
Bottom width, b	2.75	m
Water depth, y	**0.88**	**m**
Manning roughness, n	0.05	
Channel slope, S	0.1	
Top width, T	5.214	m
Wetted perimeter, P	5.84	m
Area, A	**3.50416**	m^2
Hydraulic radius, R	**0.60**	m
Flow velocity V	**4.50**	m/s
Discharge	**15.76**	$\mathbf{m^3/s}$

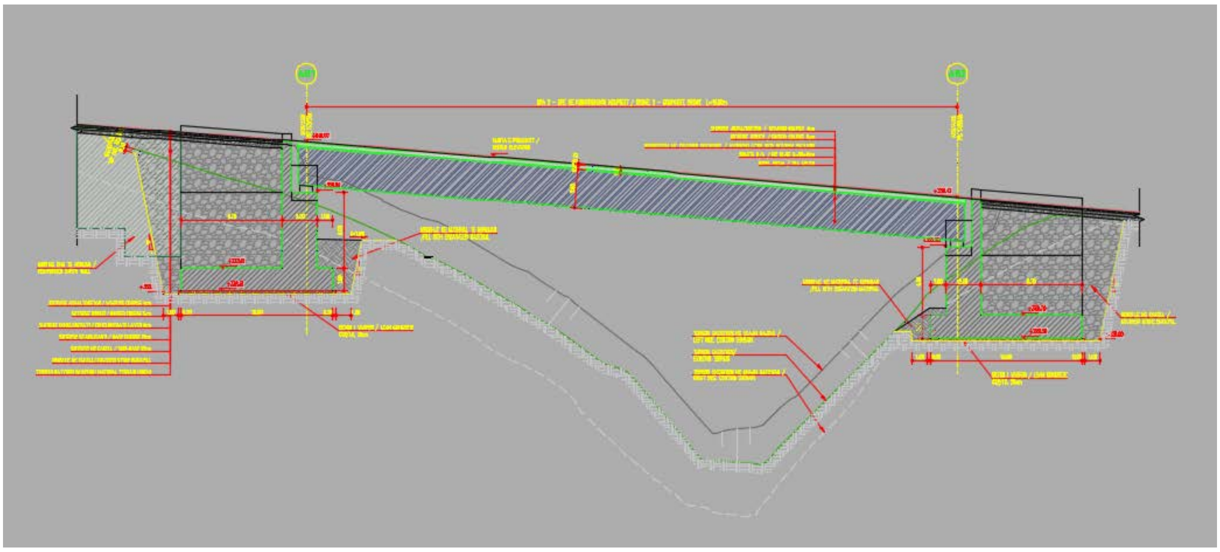

Figure 5-2 Bridge 2

Table 5-2 Water depth at the stream Bridge 2 for return period RP=50 years

Parameter	Vlera	Unit
Side slope bank Z_1, (H:V)	0.8	
Side slope bank Z_2, (H:V)	1	
Bottom width, b	4.1	m
Water depth, y	**0.25**	**m**
Manning roughness, n	0.05	
Channel slope, S	0.14	
Top width, T	4.55	m
Wetted perimeter, P	4.77	m
Area, A	**1.08125**	m^2
Hydraulic radius, R	**0.23**	m
Flow velocity V	**2.78**	m/s
Discharge	**3.01**	**m^3/s**

6. PLATFORM DRAINAGE

6.1 Gutter flow

The main structures for platform drainage are the lateral gutters. They discharge on culvert placed on regular distances. The concentration time for gutter flow calculation is accepted 10 min and runoff coefficient C=0.95. The return period for the design flow is accepted 10 years. The longest section is about 160m (6+700 - 6+860). The return period for

The ponded width is a geometric function of the depth of the water (y) in the curb and gutter section. The spread is usually referred to as ponded width (T), as shown in figure below.

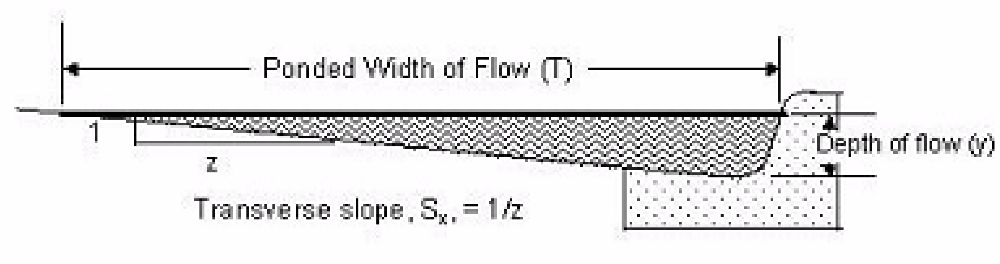

Figure 6-1 Gutter Flow Cross Section Definition of Terms

Using Manning's Equation for Depth of Flow as a basis, the depth of flow in a curb and gutter section with a longitudinal slope (S) is taken as the uniform (normal) depth of flow. For equation below, the portion of wetted perimeter represented by the vertical (or near-vertical) face of the curb is ignored. This justifiable expedient does not appreciably alter the resulting estimate of depth of flow in the gutter section.

$$y = z(\frac{QnS_x}{S^{1/2}})^{3/8}$$

where:

y = depth of water in the curb and gutter cross section (ft. or m)

Q = gutter flow rate (cfs or m3/s)

n = Manning's roughness coefficient

S = longitudinal slope (ft./ft. or m/m)

S_x = pavement cross slope = 1/x (ft./ft. or m/m)

z = 1.24 for English measurements or 1.443 for metric.

Table 6-1 Road gutter flow calculation

Carriageway width (m)	L	13
10 min intensity, RP=10 years(mm)	i	129
Flof for 160m segment length (m)	Q (cms)	0.016304
Manning's coefficient	n	0.014
Transverse slope Sx	Sx	0.08
Longitudinal slope	S	0.04
Water depth (cm)	y (cm)	4.410104

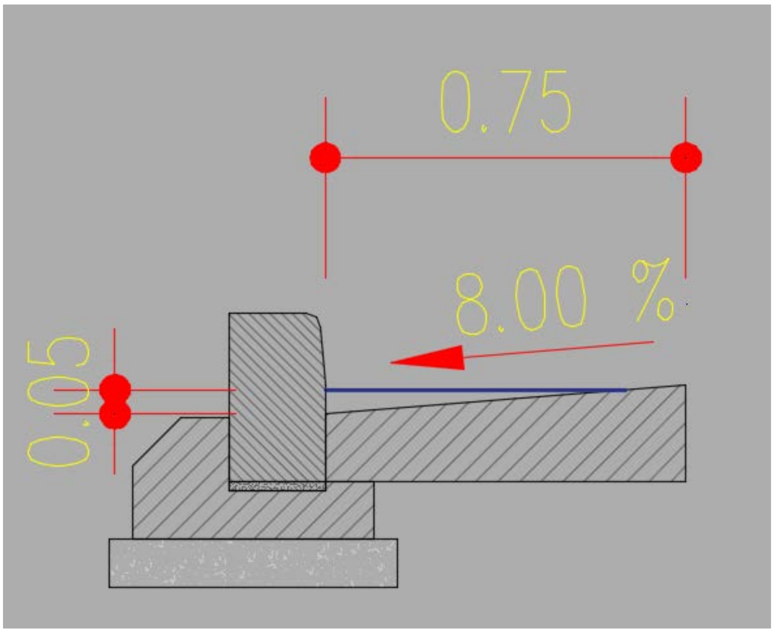

Figure 6-2 Water depth in road gutter

6.2 Bridge drainage

Bridges are drained though grated manholes located at regular spacing along the deck. Control over discharge flowing from the deck is exerted at two different stage for each draining point:

- The flow is conveyad from the deck to the grated manhole
- The flow enters the pipeline under the deck

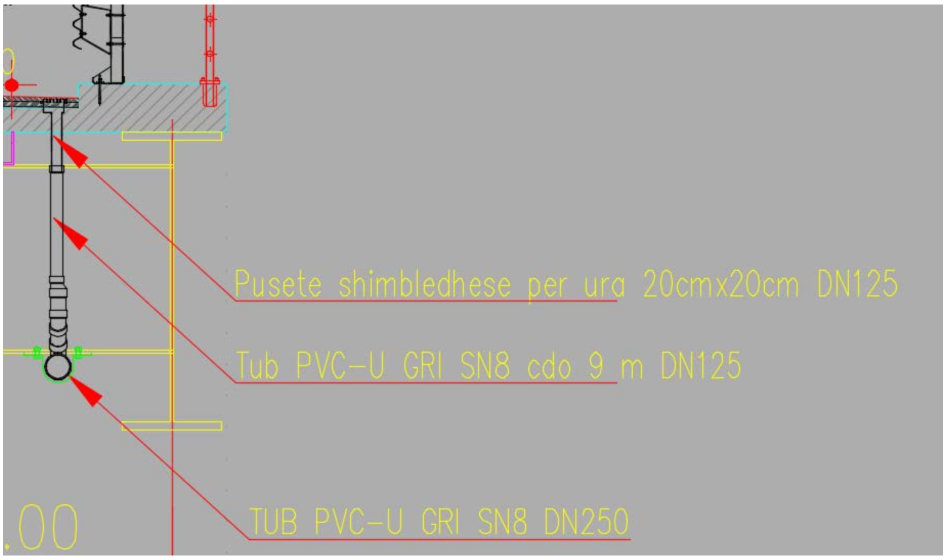

Figure 6-3 Bridge 1 drainage system

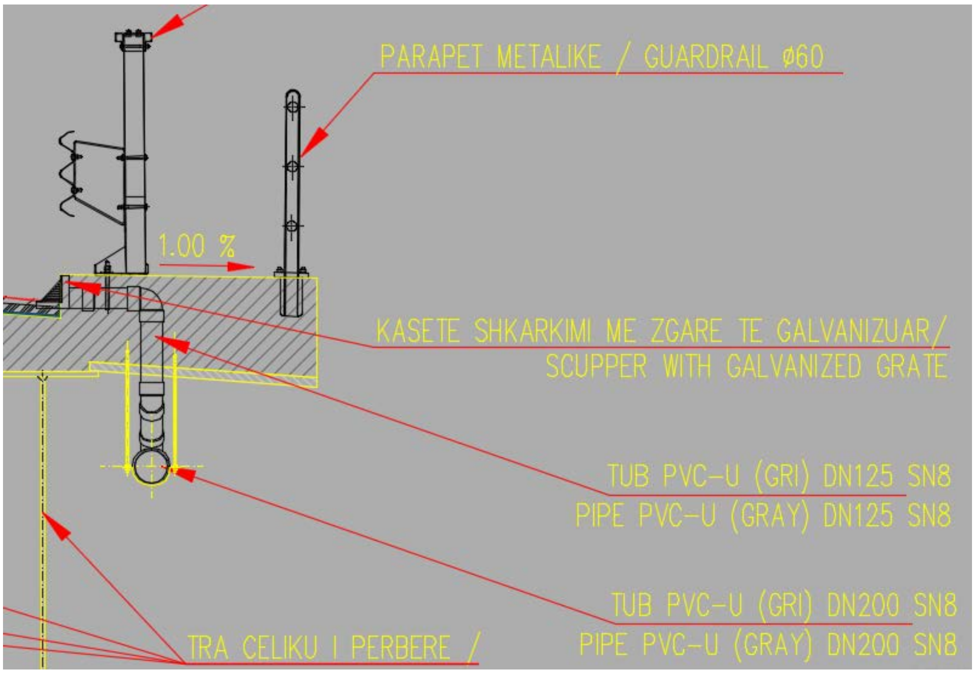

Figure 6-4 Bridge 2 drainage system

The distance between grated manholes is the main factor to consider for bridge deck drainage. The inlet capacity of the manhole is compared against the storm runoff. The time of concentration is accepted 5min and the reurn period 20 yerar. Under these parameters from Dhermi meteorological station the rainfall intensity is i=145.5 mm/h. Considering the runoff coefficient of the deck C=1 and the width of the bridge 15m, the discharge from the storm is q = 0.604 l/s/m.

The capacity of grate inlets operatings as weirs is:

$$Q_w = C_w \cdot P_a \cdot d^{1.5}$$

$Q_w - discharge\ capacity\ (\frac{m^3}{s})$

$C_w = 1.66\ \ weir\ ceofficient$

$P_d = grate\ perimeter\ (m)$

$d = flow\ depth\ (m)$

In order for the flow to spread just 1 meter (at least 2m of free lane remaining) the water depth on the grate inlet should be 5cm. Considering 5 cm depth over the grate inlet and a perimeter of 60 cm (the sidewalk side of the manhole is not considered)

$$Q_w = 1.66 \cdot 0.6 \cdot 0.07^{1.5} = 0.011\ m3/s\ (11\frac{l}{s})$$

Distance between the manholes should be D = 18.3m. By taking in account 50% efficiency of the grate inlet due to debries it is decided for the manholes to be placed each 9m in the Bridge 1 deck.

For Bridge 2 it is decided to use scooper drains because the steel beam of the deck doesn't allow the vertical drainage. Dimmensions of the scooper are 18cm x13 cm. Following a similar analisys as in the Bridge 1 the distance between the scoopers is evaluated 6m.

Pipeline drainage under Bridge 1 deck has the following characteristics

Pipe PVC-U (gray) DN250 SN 8

Total discharge (RP=20 years, tc = 5 min) Q = 66 l/s

Manning's coefficient n = 0.011

Longitudinal slope i = 4%

Water depth h = 0.13 m

Velocity V = 2.85 m/s

Pipeline drainage under Bridge 2 deck has the following characteristics

Pipe PVC-U (gray) DN200 SN 8

Total discharge (RP=20 years, tc = 5 min) Q = 26 l/s

Manning's coefficient n = 0.011

Longitudinal slope i = 8%

Water depth h = 0.07 m

Velocity V = 2.89 m/s

7. PORTAL DRAINAGE SYSTEMS

7.1 South Portal drainage system

The drainage system at the nord portal is composed by three components.

1. Conveyance of the groundwater captured by the groundwater system of the tunnel
2. Conveyance of the tunnel road drainage system which drains the water used during the road cleaning process. This system includes also an Oil-Water separator before the discharge point
3. Stormwater drainage system

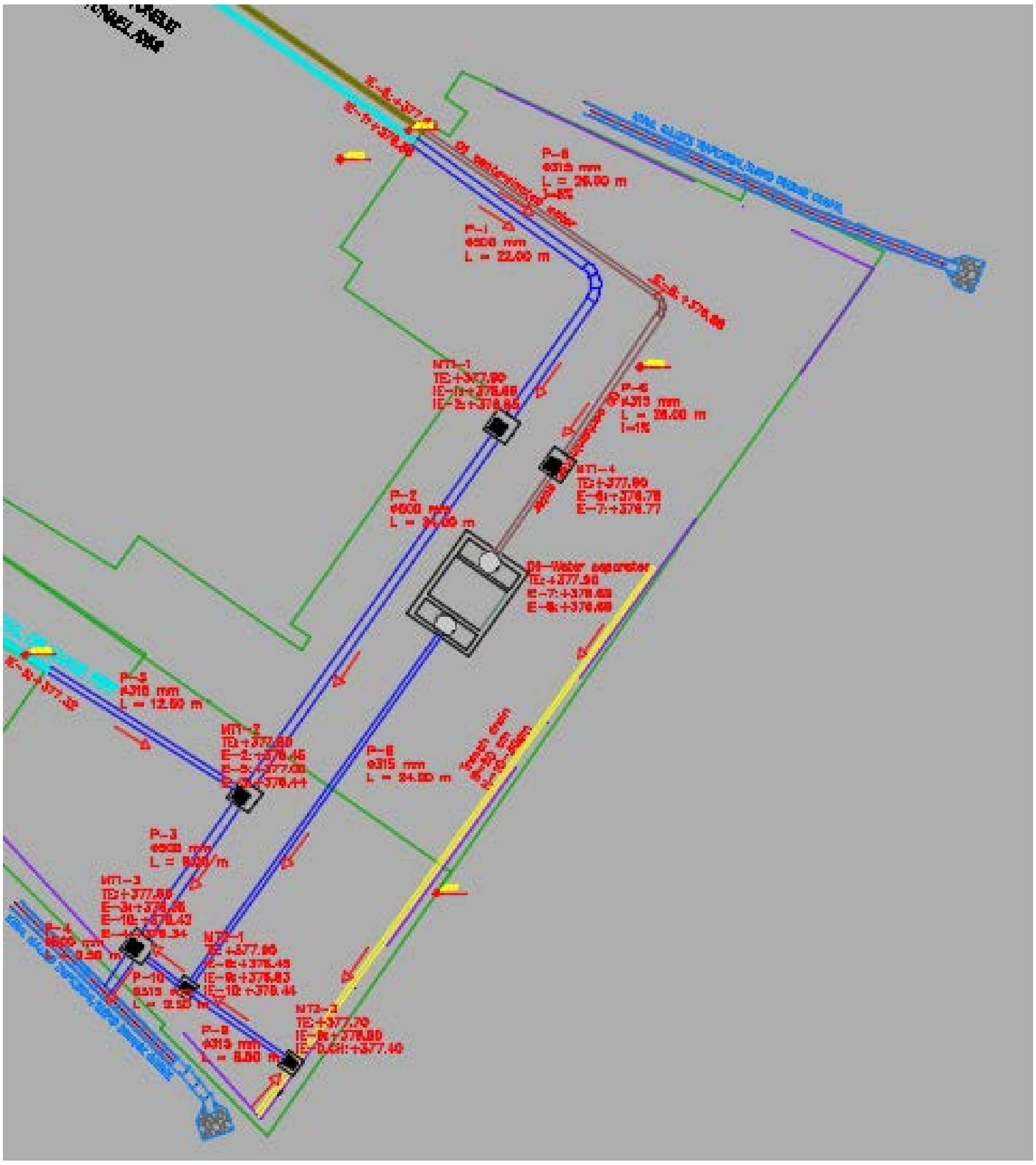

Figure 7-1 Drainage systems South Portal

7.1.1 Conveyance of the groundwater drainage system of the tunnel

This system conveys the groundwater collected along the tnnel to a discharge point. The design flow of the of this system is 0.32 m3/s as calculated in the hydrogeological report and the groundwater collection pipe is DN500mm. Summary table of hydraulic calculations is given below.

Table 7-1 Summary hydraulic calculations of tunnel groundwater groundwater drainage system

Manhole	Connection point	MT1-1	MT1-2	MT1-3	Discharge point
Station	0	22	46.74	56.86	59.93
Project elevation	378.12	377.9	377.9	377.9	377.9
Incoming pipe elevation	376.86	376.66	376.45	376.35	376.32
Outcoming pipe elevation	376.86	376.66	376.45	376.35	376.32
Manhole depth	1.26	1.24	1.45	1.55	
Outcoming pipe slope	0.0091	0.0085	0.0099	0.0098	
Outcoming pipe diameter(m)	0.46	0.46	0.46	0.46	
Optimal depth (m)	0.437	0.437	0.437	0.437	
Depth (m)	**0.3300**	**0.3300**	**0.3300**	**0.3600**	
Manning coefficient	0.01	0.01	0.01	0.01	
Outcoming pipe slope (m/m)	0.009091	0.008488	0.009881	0.009772	
Angle θ (rad)	4.04	4.04	4.04	4.34	
Area (m2)	0.13	0.13	0.13	0.14	
Hydraulic radius (m)	0.14	0.14	0.14	0.14	
Velocity(m/s)	2.54	2.45	2.65	2.66	
Flow(l/s)	**323.8**	**312.9**	**337.6**	**371.4**	
Partial drainage area (m^2)	2651.00	2212.00	2216.00	2216.00	
Total drainage area (m^2)	7079.00	4428.00	2216.00	2216.00	
Concentration time	5 min	5 min	5 min	5 min	
Rainfall intensity (mm/h)	162.00	162.00	162.00	162.00	
Runoff coefficient	0.90	0.90	0.90	0.90	
Flow rate from the storm for each pipe(l/s)	286.6995	179.334	89.748	89.748	

Note that the flow at manhole MT1-3 is increased at 370 l/s because of the additional flow from the tunnel road drainage system and South portal drainage system. The respective design flow of each of them will be discussed in the following chapters.

7.1.2 Conveyance of the tunnel road drainage system

It is supposed that the cleaning process of the tunnel road will be carried out by a fire truck veichle or a similar one. In general, the flow capacity of a fire truck is about 1000 – 1500 l/min (17 – 25 l/s). For this reason, the design flow of the the tunnel road drainage system is accepted 20 l/s. Summary table of hydraulic calculations is given below.

Table 7-2 Summary hydraulic calculations of tunnel road drainage system

Manhole	Connection point	Bend section	MT1-4	O/W separator	MT2-1
Station	0	15.46	26.68	32.44	56.1
Project elevation	378.12	377.9	377.9	377.9	377.9
Incoming pipe elevation	377.63	376.88	376.77	376.69	376.44
Outcoming pipe elevation	377.63	376.88	376.77	376.69	376.44
Manhole depth	0.49	1.02	1.13	1.21	1.46
Thellesia e pusetes dalje	0.49	1.02	1.13	1.21	1.46
Outcoming pipe slope	0.0485	0.0098	0.0139	0.0106	
Outcoming pipe diameter(m)	0.3	0.3	0.3	0.3	
Optimal depth (m)	0.285	0.285	0.285	0.285	
Depth (m)	**0.0550**	**0.0820**	**0.0750**	**0.0800**	
Manning coefficient	0.01	0.01	0.01	0.01	
Outcoming pipe slope (m/m)	0.048512	0.009804	0.013889	0.010566	
Angle θ (rad)	1.77	2.20	2.09	2.17	
Area (m2)	0.01	0.02	0.01	0.02	
Hydraulic radius (m)	0.03	0.05	0.04	0.05	
Velocity(m/s)	2.29	1.30	1.47	1.33	
Flow(l/s)	**20.3**	**20.3**	**20.3**	**20.1**	
Partial drainage area (m^2)	2651.00	2212.00	2216.00	2216.00	
Total drainage area (m^2)	7079.00	4428.00	2216.00	2216.00	
Concentration time	5 min	5 min	5 min	5 min	
Rainfall intensity (mm/h)	162.00	162.00	162.00	162.00	
Runoff coefficient	0.90	0.90	0.90	0.90	
Flow rate from the storm for each pipe(l/s)	286.6995	179.334	89.748	89.748	

7.1.2.1 Oil – Water separator

The pourpose of the oil - water separator in this case is to remove the oily substances coming from the tunnel road cleaning process before discharging the flow to the respective stream. The design process and sizing of the oil-water separator is described below.

7.1.2.1.1 OIL SEPARATOR DESIGN PROCESS

Stocks Law

Gravity separation occurs due to the difference in specific gravity between oil and water. The rate of this separation is calculated using a formula known as Stoke's Law. The formula predicts how fast an oil droplet will rise through water based on the droplet density, size and distance it must travel. Coalescing media improves the efficiency of oil/water separators by reducing the distance oil droplets need to rise before joining other oil droplets and rising to the surface. Once the oil comes in contact with the media, the oleophilic polypropylene material effectively removes it from the flow stream. Oil accumulates on the media surface, forming larger and more buoyant droplets that eventually break away from the media, rise to the water surface, and are trapped and isolated from the system outlet.

Surface Loading Rate and Rise Rate

In order to properly design the VortClarex system, the surface loading rate must be compared to the critical rise rate, also known as the terminal velocity. Surface loading rate equation:

$$Surface\ load\ Rate = \frac{Design\ flow}{Horizontal\ surface\ of\ the\ separator}$$

Next determine the rise rate of the oil droplet:

$$V_t = \left(\frac{g}{18\mu}\right) x (p_w - p_o) x D^2$$

$V_t - rise\ rate\ of\ the\ oil\ droplet$

$g - acceleration\ due\ to\ gravity$

$\mu - viscosity\ of\ the\ wastewater$

$p_w - water\ density$

$p_o - oil\ density$

$D - oil\ droplet\ diameter\ (commonly\ accepted\ 150\ micron)$

According to the equation, larger diameter droplets of oil will rise to the water's surface faster. If rise rate is greater than the surface loading rate, a majority of the oil droplets will reach the water surface and be trapped.

Treatment Capacity

Maintaining non-turbulent flow throughout the system allows for the maximum separation possible. Turbulent flow will disrupt the coalescing process, causing the system to perform inefficiently. The Reynolds Number, Re, is used to determine flow conditions:

$$R_e = \frac{pVL}{\mu}$$

$p - particle\ density\ of\ the\ media\ (wastewater)$

$V - flow\ velocity$

$L - hydraulic\ radius$

$\mu - viscosity\ of\ the\ wastewater$

To maintain laminar flow conditions, the Reynolds Number should be kept below 2300.

The system should also conform with the additional following hydraulic conditions and parameters:

- Hydraulic distribution of the influent flow must fully utilize the cross-sectional area of the media.
- Flow control and direction must be determined to prevent hydraulic short-circuiting around, under or over the media pack.
- Horizontal flow-through velocities in the separator must not exceed 4 ft/min (1.2 m/min) or 15 times the rate of rise of the droplets, whichever is smaller.

Table 7-3 Summary hydraulic calculations of oil-water separator

Q(l/s)	**20**
L(m)	**4**
B(m)	**3.5**
H (m)	**2.5**
V (m/s) < 15Vt	**0.002285714**
Surface loading rate	**0.001428571**
Oil droplet rise rate (Vt)	
g (m/s2)	9.81
µ (m2/s)	0.0011
pw (kg/m3)	1000
po (kg/m3)	850
D (m)	0.00015
Vt (m/s)	**0.001672159**
Reynolds number calculation	
p (kg/m3)	1000
V (m/s)	0.002285714
L(m)	1.029411765
µ (m2/s)	0.0011
Re < 2300	**2139.04**

7.1.3 South Portal storm water system.

The area of the south portal is sloped towards the retaining wal located at its edge. For this reason, parallel to the with the retaining wall is located a trench drain for collecting the stormwater. The hydrological conditions taken in account for evaluating the design flow of this part are as follow:

Drainage area A = 635 m2

Rainfall return period RP = 10 years, concentration time tc = 5min

Rainfall intensity i = 145.5 mm/h (Dhermi station)

Runoff coefficient C = 0.95

Q = 25.7 l/s

Table 7-4 Trench drain hydraulic calculation

Parameter	Value	Unit
Side slope bank Z_1, (H:V)	0	
Side slope bank Z_2, (H:V)	0	
Bottom width, b	0.2	m
Water depth, y	**0.17**	**m**
Manning roughness, n	0.014	
Channel slope, S	0.005	
Top width, T	0.2	m
Wetted perimeter, P	0.54	m
Area, A	**0.034**	m^2
Hydraulic radius, R	**0.06**	m
Flow velocity V	**0.80**	m/s
Discharge	**0.027**	$\mathbf{m^3/s}$

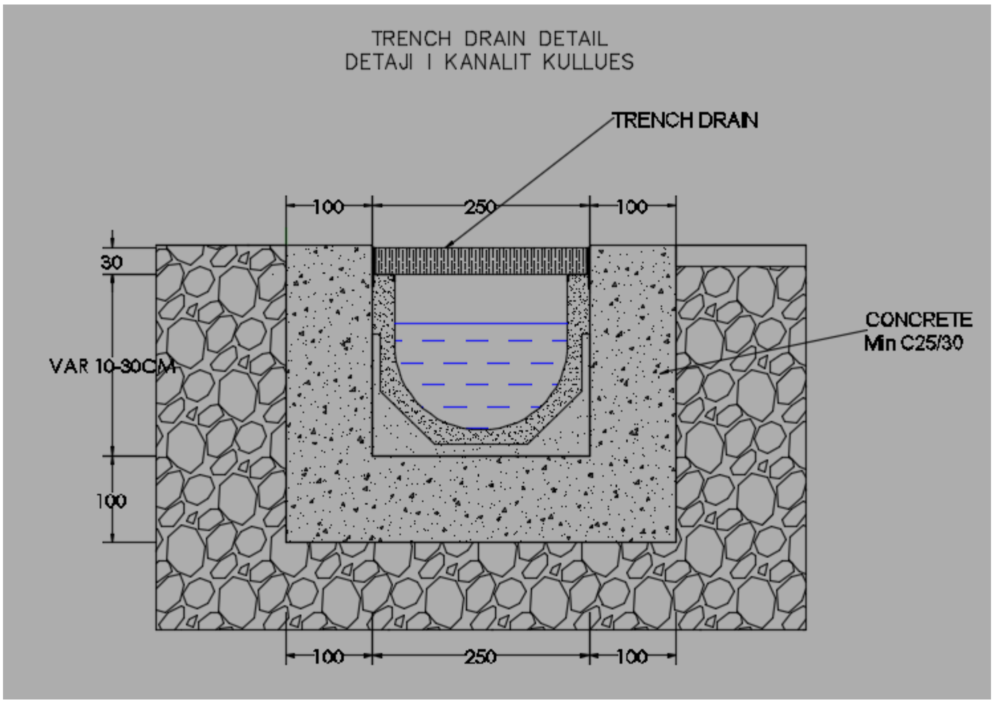

Figure 7-2 Trench drain cross section

The terrain elevation of the along the trench drain is constant. In order to carry the appropriate design flow, a slope of 0.5% is given to the bottom of the channel. For this reason, the depth of the trench drain varies from 10 cm to 30 cm.

7.2 North Portal drainage system

The drainage system of the north portal is composed by a single pipeline and five rainwater manholes. This system serves to drain the water of the area at the service building located on the left of the north portal. The system discharges the outlet of culvert CN-5. The design parameters of this system are as follow:

Drainage area A = 1800 m2

Rainfall return period RP = 10 years, concentration time tc = 10min

Rainfall intensity i = 108 mm/h (Vlora station)

Runoff coefficient C = 0.9

Q = 48 l/s

Draiange Pipe: Corrugated HDPE DN315 SN8

7.3 Other utilities at the service bulding

Other utilities located at the service building manhole are:

1. Water supply connection with the local water distribution system DN32 mm. The water supply connection includes also a watermeter manhole as showed in the respective drawings.
2. Septic tank for the wastewater which is located in front of the service building at a distance 2m from the building entrance. The outlet pipe passes under the road and discharges to the filtration field pipe network wich is placed at e depth approximately 1.5m from the ground.

For sizing the septic tank, it is used the British standard wich takes in consideration the following formula:

$$C = A + P(RxQ + NxS)$$

$C - capacity\ in\ liters$

$P - number\ of\ people$

$A - 2000\ liters\ as\ a\ constant$

$R - retention\ period\ of\ sewage\ in\ days$

$Q - sewage\ flow\ in\ liter/day$

$N - number\ of\ years\ between\ cleanings$

$S - sludge\ accumulation\ in\ liters\ per\ person/year$

$$C = 2000 + 10(1x1000 + 1x30) = 12300\ liters\ (12.3m3)$$

8. FIRE FIGHTING WATER SUPPLY SYSTEM

The fire fighting system of the tunnel will be supplied by a water tank located above the North Portal at elevation 501.00 masl. The tank has a volume of 200m3 and will be filled by fire trucks veichles. The flow requirement from the reservoir is 20 l/s.

The water supply pipeline is a DCI DN200mm that runs along the access road of the tank and connects with the fire fighting system of the tunnel at the North Portal at the elevation 413.04 masl. The length of the pipeline is approximately 950m. The pipe is divided in two pressure rating parts PN10 until station 0+600 and PN16 until the connection point.

8.1 Head losses

For friction head loss calculation, it is used the Hazen-Williams equation:

$$H_w = \frac{10.68LQ^{1.852}}{C^{1.852}D^{4.87}} = \frac{10.68 * 950 * 0.02^{1.852}}{140^{1.852}0.2^{4.87}} = 0.3m$$

$L - pipe\ length\ (m)$

$Q - flow\ rate\ \left(\frac{m^3}{s}\right)$

$D - pipe\ iner\ diameter\ (m)$

$C - Hazen - Wiliams\ roughnes\ coefficent, C = 140\ for\ Ductile\ Iron\ pipes$

$g - gravitational\ acceleration, 9.81\ m/s2$

Local head losses are accepted 15% of the friction head losses. The total head losses are:

$$H_T = H_w + 0.15H_w = 0.3 + 0.15 * 0.3 = 0.35\ m$$

The minimum available head at the connection point with the fire fighting system wil be

$$Tank\ elev. - Connection\ elev. - H_T = 501.00 - 413.04 - 0.35 = \mathbf{87.61\ m}$$

8.2 Water hammer evaluation

Instantaneous closure of the valve

The water hammer instantaneous valve closure calculation predicts the maximum increase in pressure that will occur due to a sudden valve closure. The valve closure time in water hammer is considered to be instantaneous if the valve closes faster than (or equal to) the time required for a pressure wave to travel two pipe lengths (i.e. the time for the wave to travel downstream from the valve, reflect off the upstream boundary and return to the valve). The pressure predicted by the water hammer instantaneous valve closure calculation provides the engineer with the expected maximum pressure increase. The water hammer calculation can also be used in reverse - to compute the pipe velocity - if a maximum pressure rise due to water hammer is input.

One-dimensional momentum conservation for frictionless flow is used to derive the Joukowski equation for water hammer. The equation was developed for a liquid flowing steadily through a pipe and then instantly the velocity drops to zero due to a sudden valve closure in a water hammer event. The water hammer equation assumes that liquid compression and pipe friction are negligible. Though the Joukowski equation's primary applicability is for a liquid velocity that drops to zero upon contacting a closed valve causing water hammer, the equation is valid for any instantaneous drop in velocity, not necessarily a drop to zero velocity. The Joukowski equation is seen with and without a negative sign on the right-hand side depending on whether the pressure wave is traveling upstream or downstream in the water hammer event.

$$\Delta P = \rho c \Delta V$$

$\Delta P - maximum\ pipe\ pressure\ increase\ in\ water\ hammer\ event\ due\ to\ sudden\ valve\ closure$

$\rho - fluid\ density$

$\Delta V - change\ in\ velocity\ in\ water\ hammer$

$c - celerity\ (wave\ speed)$

The equation for wave speed, c, during water hammer is based on mass conservation and allows the pipe wall material to expand

$$c = \sqrt{\frac{E}{\rho}}$$

$$\frac{1}{E} = \frac{1}{E_f} + \frac{D}{w * E_p}$$

$E - composite\ elastic\ modulus$

$E_f - elastic\ modulus\ of\ fluid$

$E_p - elastic\ modulus\ of\ pipe\ material$

$D - inside\ pipe\ diameter$

$w - pipe\ wall\ thickness$

Instantaneous valve closure due to water hammer is defined to occur if the valve is closed faster than the wave travel time

$$t_w = \frac{2L}{c}$$

$t_w - \ wave\ travel\ time$

$L - pipe\ length$

In water hammer, the wave travel time, tw, is the time for a pressure wave to propagate from the valve, upstream to the reservoir, and back down to the valve.

Table 8-1 Water hammer calculation for instant valve closure

D (m)	0.2034
Ef (N/m2)	2150000000
Ep (N/m2)	210000000000
L (m)	950
w (m)	0.00640
ΔV (m/s)	1
ρ (kg/m3)	1000
E (N/m2)	1622176937
c (m/s)	1274
tw(s)	1
ΔP (m) (tc=tw)	**79**

Time of closure > tw

If the time of closure tc > tw then the closure is said to be gradual and the increased pressure is

$$\Delta P = \frac{\rho L \Delta V}{t_c}$$

For the gradual closure the time of closure is accepted tc = 4s then the pressure increase is:

$$\Delta P = \frac{1000x\ 950x\ 0.61}{4} = 144875\ (14.76m)$$

8.3 Hydrological Conditions and Tunnel Drainage

8.3.1 Expected Hydrological Conditions

Conclusions of the hydrogeological report

- The Llogara Tunnel will be built on the carbonate rocks of the Mount Kanali.

- In these rocks, various factors developed over the geological time of millions of years have developed karstic phenomena and formed a well-developed hydrographic network of groundwater.

- The expected dynamic groundwater flow in the tunnel is expected to be between 0.200m3/s and 0.300m3/s.

- The expected maximum dynamic groundwater flow in the tunnel is expected to be around 0.332m3 / s.

- In addition to the expected maximum dynamic flow, in the tunnel can be found unpredicted amounts of groundwater in the form of eruptions, which can reach significant quantities and last up to several weeks.

- Determining the location of different caverns that may meet during the advancement of the tunnel, or that may be near it, at this stage of the study is not possible.

- The permeability of the geological environment where the tunnel will pass is related with cracks and caverns created by karst processes. The given hydraulic indicators rely on analogies with similar aquifers.

- The qualitative properties of groundwater that is expected to be encountered in the tunnel are good, they are not aggressive towards metal and concrete structures.

Recommendations

- For the identification of water eruptions in the tunnel or the presence of various caverns, we firstly recommend drilling horizontal wells, 20-25 m long, or performing periodic geophysical works, then to advance with the front of the tunnel.

- At the end of the tunnel opening workings, we recommend performing geophysical works on the floor of the tunnel to ensure that under the floor there are no open (empty) karst caverns that could endanger the safety of passengers and vehicles.

Tunnel design aspects

It is understood that the assessment of the ground water table and inflow to the tunnel made in the hydrogeological study is subject to considerable uncertainty due to lack of enough specific factual data to calibrate the assumed model and properties. The current prediction of water inflow to the tunnel with more than 300 l/sec in exceptional circumstances is very high. A 3D assessment of the potential water table has been made in order to evaluate better the water table.

Aspects for tunnel excavation phase:

It is understood there are no particular hydraulic compartments within the alignment of Llogara tunnel. The karstified limestone rock mass is generally considered permeable without expected aquiclude elements, like extended cataclastic fault zones with a clay matrix (fault gauge) or flysch type rocks. The whole mountain can be understood as a rock mass perforated by vertical and bedding parallel karst pipes, through which the rainwater drains. The drainage level can approximately be assumed at the sea level. As long as the rainfall is less than the drainage capacity of the underground drainage system (which is supposed to be the case during the dry summer season), the water level within the karst channels is expected not higher than 350 m above sea level. In the rainy (winter) season, the amount of rainfall can be temporarily higher than the drainage capacity of the karst system, and therefore the water table will build up to an estimated elevation of 500 m above sea level.

Therefore, with the advance of the tunnel during winter season, the water table (where existing) will be drawn down in front of the tunnel with the advance of the excavation. However, the excavation may encounter karst channels where locally very high-water ingress occurs.

For the North tunnel drive which is a predominantly downhill excavation with a gentle slope of 0.6% there is need to provide pumping capacity in the construction arrangements. However, the elevation between the expected lowest point of the North drive in relation to the highest point is less than 20m (3000m x 0,006). The amount of pumping capacity should, however, for practical reasons be limited to 50 l/sec. Therefore, in the event of a much larger temporary inflow of water, the tunnel excavation needs to be interrupted until the inflow is reduced by draw-down of the water head and subsequent pumping of the water from the tunnel front. Since the karstic features are expected to have a high permeability, the draw-down should be a matter of days.

For the South tunnel drive which is an uphill excavation, water inflows can be drained gravitationally to the South tunnel portal.

Aspects for the permanent drainage

It is understood that there is a seasonal variation of the ground water table likely. In winter time, there may be a build-up of a water table above tunnel level, in summer time the rock mass in tunnel level is very likely dry.

The current design includes a very large main drainage pipe dia. 500mm. Since the lateral drainage is connected to the main drainage in short intervals, the lateral drainage is not critical. The main drainage pipe with dia. 500mm and inclination of 0.6% can – according to preliminary calculations - deliver some 300 l/sec. Therefore, the foreseen drainage system is practically capable to dealing with a kind of worst-case scenario as stipulated in the hydrogeological study. Nevertheless, this concept shall be verified by collecting experience during excavating the tunnel at least during two wet seasons. Even such late experience could be considered in the final solution of the drainage during construction.

8.3.2 Tunnel Waterproofing System

The tunnel is designed as a drained tunnel with a waterproofing and drainage layer on the arch of the tunnel and drainage in the sides at the level of the carriageway structure. In this way, a water pressure on the lining is avoided, and on the other hand, it is necessary to plan long-term maintenance work on the drainage system.

The waterproofing system itself consists of the following layers:

1. Basis or sprayed concrete support for the waterproofing layer with a minimum thickness of 3cm
3. Geotextile drainage and waterproofing protective layer
4. Waterproofing layer with a minimum thickness of 2.0 mm

Waterproofing Basis Shotcrete

For the needs of leveling all supporting elements and shotcrete of the primary lining, it is necessary to treat the substrate with shotcrete C20 / 25 and grain size 0-4 mm. The mix design of the shotcrete must ensure a low potential for calcium leakage in a way that meets the requirements of Table 4.1 of the ÖVBB Richtlinie 'Tunnelentwässerung' Issue April 2010.

Protective and drainage layer

As a rule, a protective and drainage layer is made of a high-permeability composite of 3 dimensional polypropylene geotextiles laminated on a drainage layer of monofiber polypropylene geotextiles with a thickness of min. 5 mm.

PVC fastener

PVC fastener consists of a PVC disc for temporary attachment of the protective and drainage layer to the shotcrete. The element must not cause the protective and drainage layer to break.

PVC membrane

PVC membrane thickness min. 2 mm (without downward tolerance) must comply with the requirements of Table 4.6 ÖBV 'Richtlinie Tunnelabdichtung Issue Dezember 2012.

Drainage connecting element

The Contractor shall use a prefabricated connection element for drainage on the parts of the tunnel, in order to facilitate the execution of works in terms of laying PVC membrane and drainage backfill.

Figure 11-1 Drainage pipe, drainage gravel and prefabricated element to facilitate connection to waterproofing membrane

Figure 11-2 Installed water proofing membrane

8.3.3 Tunnel Drainage System

The drainage tunnel system consists of the following elements:

- Drainage backfill for drainage pipe
- Side drainage pipes DN 315
- Drainage layer under the pavement structure
- Collective drainage pipes DN 500, which collects water from side drains and drainage layer

The concept of the drainage tunnel is based on two side drainage pipes at the height of the tunnel foundation and a collecting drainage pipe, which drains the water under the road to the portal area. There are no shafts in the carriageway on the collecting drainage pipe, but the pipe is directed in each inspection niche on the eastern wall of the tunnel. Such a system ensures a longer carriageway life and eliminates problems with manhole covers.

The drainage system with a main pipe dia.500mm and a tunnel gradient of 0.6% has a capacity in the order of 300 l/sec.

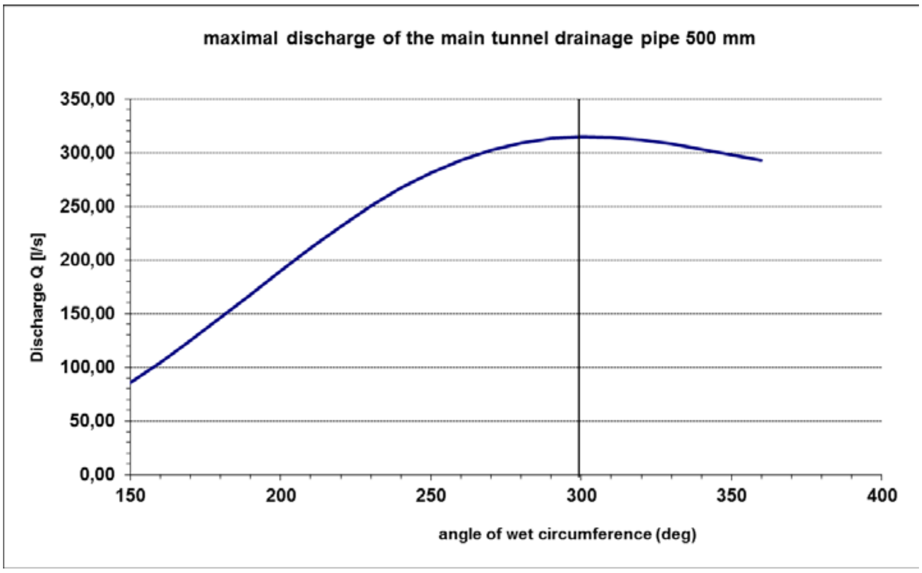

Figure 11-3 Preliminary calculation of main drainage pipe capacity at 0.6% inclination

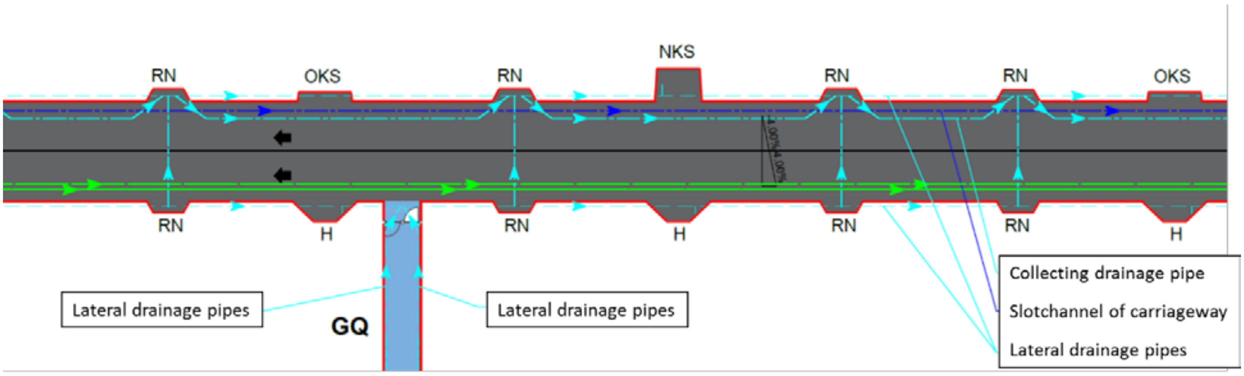

Figure 11-4 Principle of tunnel drainage arrangement

8.3.3.1 INSPECTION AND MAINTENANCE NICHES

Drainage inspection and maintenance niches are located in pairs every 60 m, with the inspection niche east being larger, as it also allows cleaning of the collecting drainage pipe. The inspection niche west is smaller and allows a transverse connection to the inspection niche east, being designed in such a way that at normal water flows flows into the collecting pipe every 4 inspection niches. With increased flows, excess water is drained through the overflow. In this way, a sufficiently large flow is maintained to prevent excessive clogging.

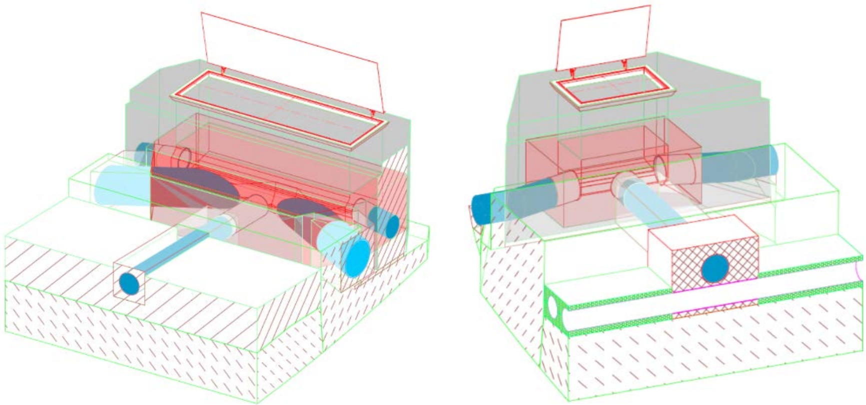

Figure 11-5 3D image of large (left) and small drainage inspection niche

The bottom of the inspection niche is made of prefabricated fiber reinforced concrete, to which the side and collecting drainage pipes are connected watertight through seals.

8.3.3.2 LATERAL DRAINAGE PIPES

Ribbed drainage pipes must not be installed in the tunnel. All drainage pipes are made of polypropylene of stiffness class SN8 with a nominal diameter of 315mm. The pipes shall have drainage notches around the circumference of 220 deg. The main drainage pipes with a nominal diameter of 500 mm shall have drainage notches around the circumference of 180 deg. The transverse drainage pipes with a nominal diameter of 315 mm are without notches. Drainage notches must be at least 10 mm wide and the total size of the openings must be at least 150 cm2 / m1. The individual notch should not be longer than 1/8 of the pipe circumference.

Figure 11-6 Lateral drainage pipe

8.3.3.3 DRAINAGE GRAVEL AROUND PIPE

The drainage material around the lateral drainage is extremely important for limiting the formation of calcite clogging in the drainage system. If porous concrete is used, it shall

- Use CEM III / A cements with a content of at least 40% fly ash or similar
- Using polymeric chemical binders without affecting the environment
- Using fasteners and backfill without binder
- Grain size of 16/32 mm without fine particles is allowed
- The aggregate must not be of carbonate origin