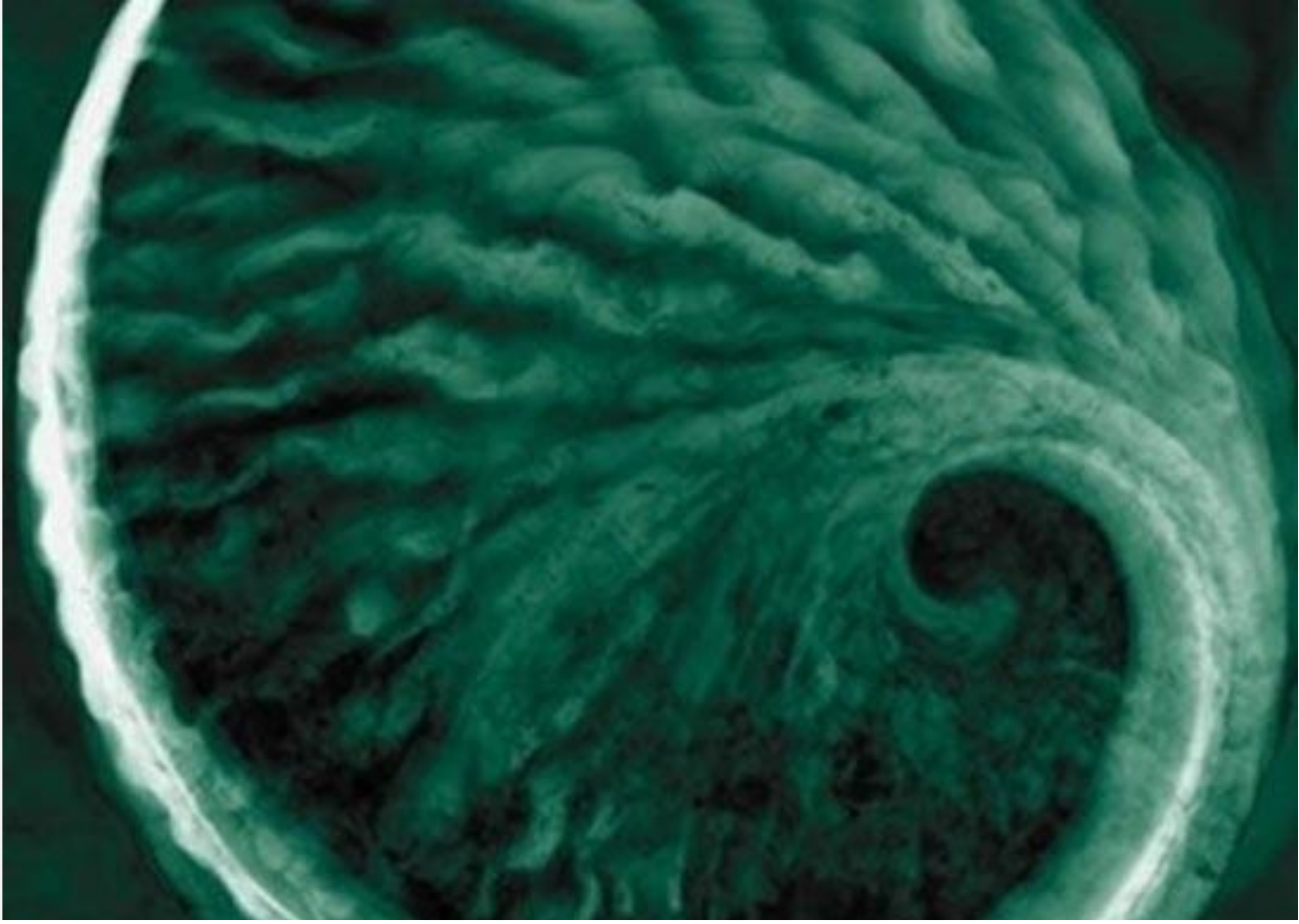

Flood Risk Assessment Report

Nakkaş Başakşehir Motorway, Turkey

Nakkaş Otoyol Yatırım ve İşletme A.Ş

05 December 2022

Project No.: 0580559

The business of sustainability

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05 December 2022

Flood Risk Assessment Report

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1. INTRODUCTION

Nakkaş Otoyol Yatırım ve İşletme A.Ş. (Nakkaş Otoyol A.Ş) a Special Purpose Vehicle (SPV) signed a contract with the Turkish Ministry of Transport, General Directorate for Highways (KGM) to build, operate and transfer a 4-lane dual toll road with a total length of 30,64 km including connection road and 1,619 m long Sazlıdere Cable Stayed Bridge.

The European Bank for Reconstruction and Development (EBRD), the Asian Infrastructure Investment Bank (AIIB), Atradius, Swiss Export Risk Insurance (SERV), Standard Chartered Bank (SCD), DZ Bank, Bank of China, Credit Suisse, Deutsche Bank, Islamic Corporation for Development of Private Sector (ICD), and Vakifbank are considering financing the Project. As a major, long-term infrastructure Project, "Section 8 - Nakkas-Basaksehir Motorway" is considered as **Category A** and it is subject to full ESIA assessment including a Resettlement Action Plan (RAP). Therefore, Nakkaş Otoyol A.Ş appointed ERM GmbH (ERM) to conduct the ESIA studies and appointed GEM Sustainability Services and Consultancy Inc. (GEM) to conduct the studies to develop RAP in line with Lenders standards.

During discussions of the draft ESIA Package with lenders in April 2022, the lenders requested that a more specific assessment be conducted on the flood risks that may be triggered by the Project development. This document in hand is the requested Flood Risk Assessment Report.

1.1 Scope

The ESIA studies for the Project indicated that the climate change hazards material in the Project region is potential wildfire, landslide, flash flooding, and water scarcity.

Some sections of the Motorway are located in close proximity (less than 1km) of water courses. For instance, the RoW runs in certain sections along the Küçükçekmece and Sazlıdere dam lake. Nevertheless, in the Project region the risk of flash flooding is considered medium, whilst river flood and urban flood are considered low. And given the distance of several kilometres to the Marmara Sea, the risk of coastal flooding is very low/negligible. The Istanbul area has potentially rainfall patterns, terrain slope, geology, soil, land cover and earthquakes that make localized landslides (and resulting flash-flooding) an infrequent hazard phenomenon (considered as a medium hazard) as defined in Table 1-1-1 Relevant Natural Hazards in the Project Area.

Table 1-1-1 Relevant Natural Hazards in the Project Area

Hazard	Hazard Level Valuation
Extreme heat	**Medium**
Wildfire	**High**
River flood	**Low**
Urban flood	**Low**
Coastal Flood	**Very low**
Landslides/Flash Flooding	**Medium**
Water scarcity	**Medium**

Source: Think Hazard, 2021[1]

[1] The Global Facility for Disaster Reduction and Recovery (GFDRR) Think Hazard. Available at: https://thinkhazard.org/en/report/3056-turkey-İstanbul [18.06.2021]

The Project structures can result in the removal of flood storage capacity, causing an increased risk of flooding elsewhere, and the hydraulic structures such as bridges, culverts, and diversion channels can also impede flow during times of flood, thus causing water levels upstream of structures to be raised above what would occur in the absence of the structure. Undersized sub-structures crossing rivers, streams, and drainage systems in urban highways may also prevent the floodwaters from flowing in extreme conditions. The backwater from the drainage sub-structures may inundate and increase the risk of loss of life and properties in the urban area.

Considering these potential risks, this flood risk assessment was prepared by ERM and ACE as a part of the ESIA studies to identify the risks and assess these risks during the design of the hydraulic structures. The scope of the study was defined as:

- Calculations of peak flow and hydrographs for 100-, 200- and 500-Years extreme events for the catchments area where the Project is crossing
- Viaduct hydraulics examination and flood risk assessment
- Culverts hydraulics examination and flood risk assessment

1.2 Study Area

The Project Right of Way (RoW) crosses the Ayamama, Nakkaş, Sazlıdere Rivers and various small streams flowing north to the south. There are five viaducts and 20 culverts crossing the streams and rivers on the alignment of the project, as shown in Figure 1-1.

Figure 1-1 Motorway Section and Main Crossing River

The study area for this Floor Risk Assessment includes the catchments of the crossings within the Project area as shown in Figure 1-2.

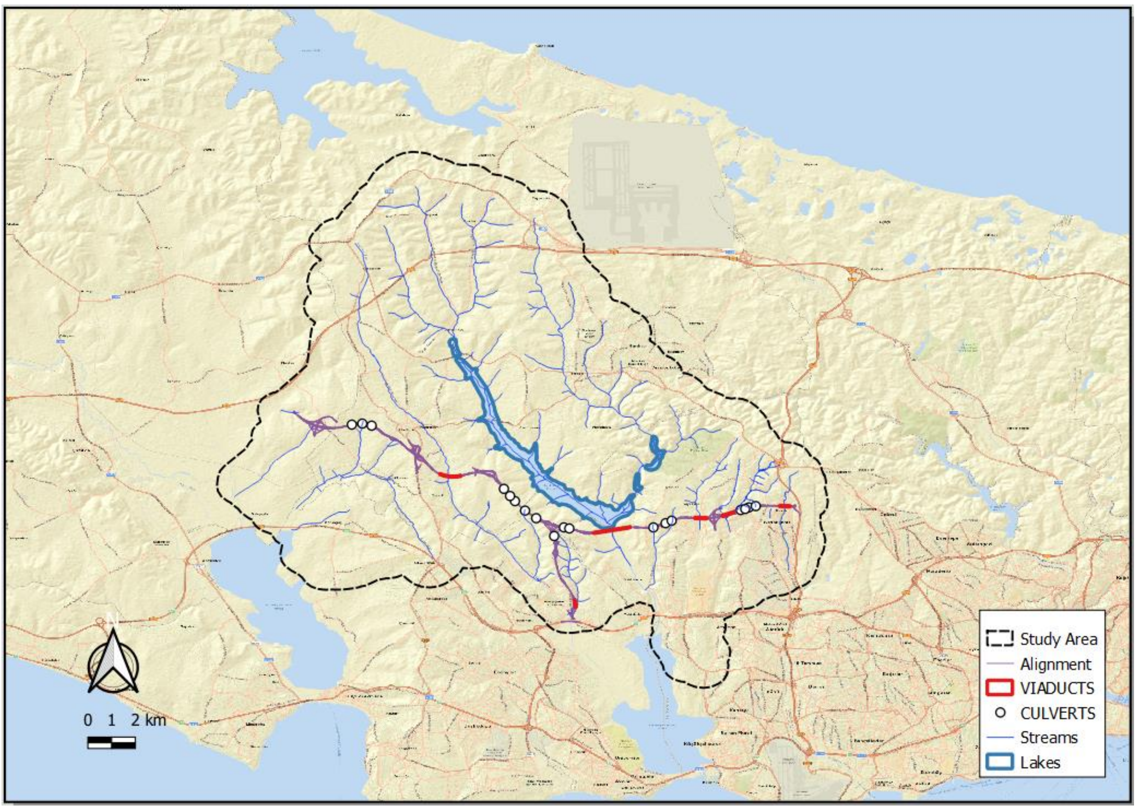


Figure 1-2 Flood Risk Assessment Study Area

1.3 Highway Drainage Structures and River Crossings

The Project includes a number of elements currently identified as shown in the following Table 1-2.

Table 1-2 Project Key Elements

Component	Details
Length of main road	24,17 km
Length of connecting roads	6,47 km
Cross Sections	2x4 lanes for main Motorway and 2x2 for connecting roads
Interchanges	10
Cable Stayed Bridge	1619 m (Length) x 46 m (Width) and Tower Height of 196 m
Overpasses	18
Underpasses	18
Viaducts	5
Culverts	55
Toll Booth	The number of toll booths has not been specified at the current stage. Free flow systems and tollgate toll collection systems will be incorporated in the Project, similar to the other segments of the NMM.

Lighting	Will be provided at intersections, toll booths and service areas.
Service Stations/Rest Areas	The type and number of Service Stations/Rest Areas have not been specified at the current stage.
O&M Facilities	There are two O&M facilities planned at KM 36+300 and at 49+200 specific for Sazlıdere Cable Stayed Bridge.These O&M facilities will also serve as Disaster Recovery centers.

The list crossing elements in the above table are shown in Table Table 1-3 and Table 1-4, respectively .

Table 1-3 List of Viaducts and the River Crossing

Name	Start KM	End KM	Length
Viaduct 01	42+841	43+809	968
Viaduct 02	55+129	55+669	540
Sazlıdere Cable Stayed Bridge	50+730	52+360	1630
Viaduct 03	56318	56+888	570
Viaduct 04	3+874	4+399	525
Viaduct 05	58+820	59+290	470

Table 1-4 List of Culverts and Properties

Name	KM	Width (m)	Height(m)
M02	38+032	4.0	2.5
M03	38+480	5.0	2.5
M04	38+895	4.0	2.5
M08	46+022	2.0	2.0
M09	46+847	2.0	2.0
M10	47+517	2.0	2.0
M11	49+448	3.0	2.0
M12	49+684	2.0	2.0
M13	53+296	3.0	2.5
M14	53+863	2.0	2.0
M15	54+148	4.0	2.0
M16	57+160	2.0	2.0
M17	57+366	2.0	2.0
M18	57+532	2.0	2.0
M19	57+833	2.0	2.0
M23	46+495	2.0	2.0
M43	00+607	2.0	2.0
M47	00+281	2.0	2.0
M53	48+152	2.0	2.0

1.4 Study Methodology

The flood risk assessment study methodology consisted of the following components

- Identification of the watershed and its features for the Project crossings
- Digital elevation modelling of the area
- Collection and assessment of observed extreme storms and peak flow of floods
- Water surface profile calculation for viaducts and culverts under extreme storm events.

A 5-meter Digital Elevation Model (DEM) of the study area from the Surveying General Administration (HGM) was prepared and used for catchment delineation for streams and rivers crossing with substructures. The DEM data is used to extract the physiographic characteristic of the catchments (Catchments area, streams length, and slope) used to extract the critical duration of extreme rainfall and calculate extreme floods with 100 and 500 years return periods. The location of viaducts and culverts is specified for this study, and their catchment area is delineated using the digital elevation model data.

For extreme floods calculation, observed extreme storms and peak flow of floods are collected and analyzed by statistical methods. The peak flow analysis is used for regional flood frequency analysis; however, stormwater analysis is used for rainfall-based floods calculation. The rational method is used for flood peak calculation in small catchments (with less than 1 km² area); however, the Synthetic Unit Hydrograph method is used for greater catchments. As a guide, the prepared standards by DSI (State Hydraulic Works), ISKI (Istanbul Water and Sewerage Administration), and

KGM (General Directorate for Highways) are considered in the calculation. The calculation is aimed to achieve the 100 years flood peaks and hydrograph. For the special condition of the Sazlıdere Cable Stayed Bridge the calculated flood hydrograph in the inflow to the Sazlidere Dam reservoir is routed for achieving the flood hydrograph in the crossing location.

The water surface profile calculation for viaducts and culverts are planned separately. The River Analysis System (RAS) was used for water surface modeling in this study. This model is a public domain software from the Hydrologic Engineering Centre (HEC) of the United States Army Corps HEC-RAS [2]uses several input parameters for hydraulic analysis of the stream channel geometry and water flow on sub-structures. These parameters are used to establish a series of cross-sections along the stream. In each cross-section, the locations of the stream banks are identified and used to divide into segments of the left floodway, main channel, and right floodway. At each cross-section, HEC-RAS uses several input parameters to describe the shape, elevation, and relative location along the stream, such as:

- River station (cross-section) number.
- Left and right bank coordinates
- Reach lengths between the left floodway, stream centerline, and right floodway of adjacent cross-sections.
- Manning's roughness coefficients for the main channel and left and right floodplains
- Channel contraction and expansion coefficients
- Geometric description of any hydraulic structures, such as bridges, culverts, and weirs

The geometry of the bridge and culvert is defined separately in the HEC-RAS model. Generally, the water surface profile is computed upstream and downstream of the crossing direction, as shown in Figure 1-3. Skew Bridge/Culvert option is available from the bridge/culvert editor, as shown in Figure 1-4. The skew angle compares the flow angle through the bridge with a line perpendicular to the cross-sections bounding the bridge.

Skewed bridge crossings are generally handled by adjusting the bridge dimensions to define an equivalent cross-section perpendicular to the flow lines. The bridge information, and cross-sections that bound the bridge, can be revised from the bridge editor. The detail of culver and bridge modeling is presented in the hydraulic reference of the HEC-RAS model.

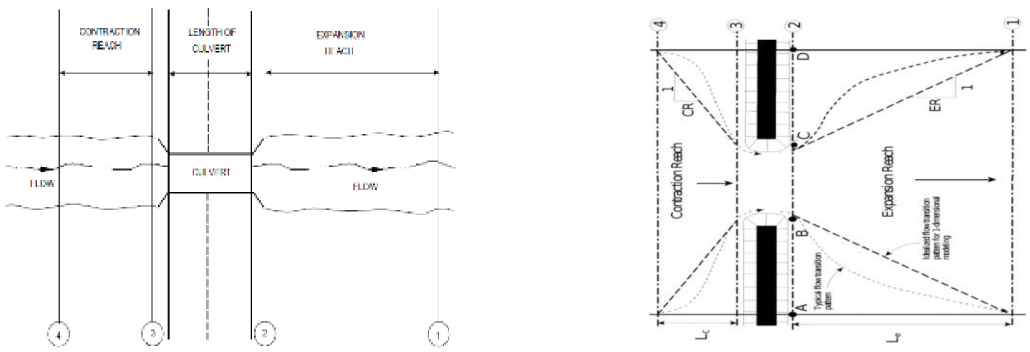


Figure 1-3 Location of the Cross-Section for Bridges and Culvert Hydraulic Modeling (Source https://www.hec.usace.army.mil/software/hec-ras/documentation/HEC-RAS%205.0%20Reference%20Manual.pdf)

[2] Hydrologic Engineering Center's (HEC) River Analysis System (HEC-RAS) software https://www.hec.usace.army.mil/

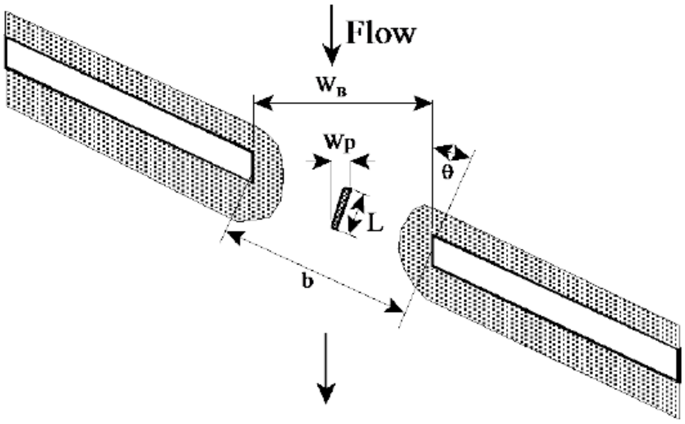


Figure 1-4 Example Bridge on a Skew condition (Source https://www.hec.usace.army.mil/software/hec-ras/documentation/HEC-RAS%205.0%20Reference%20Manual.pdf)

HEC-RAS assumes that the energy head is constant across the cross-section, and the velocity vector is perpendicular to the cross-section. For modeling the river's hydraulics, the Saint-Venant equation for unsteady flow and energy equation in steady flow is solved in sequential sections (Hicks and Peacock 2005, Johnson et al. 1999).

The constructed model is run for 100-years floods, and floods maps for the discharge were extracted in Mapper of the HEC-RAS model and represents the basis for the flood risk assessment methodology. In addition, flood risk for 200 years and 500 years were also investigated for sensitivity analysis of the crossings. The risk of floods around sub-structures is evaluated based on the flooding depth and velocity. RAS Mapper respectively automatically generates the flood maps. The details of methodology and approaches are presented in the relevant chapters.

The following limitations are present in the study:

- No field studies were done
- Project design assumed per February 2022 date
- In case any subsequent design changes made to the relevant Project components, the outcomes of this flood risk assessment may no longer be relevant
- Information on soil cover obtained from land use map of CORINE 2018. This may not be representative of the actual conditions

1.5 Collected Data

Collected data based on the described methodology are DEM, extreme storms and floods, and land cover data. Land cover and soil properties are prepared from open-source data.

1.5.1 Digital Elevation Model

As mentioned in the methodology, a 5-meter DEM of the study area from the Surveying General Administration (HGM) was prepared and used for catchment delineation. The DEM is used in TUREF/TM30 projection (EPSG: 5254). The alignment and extension of the DEM are shown in Figure 1-5.

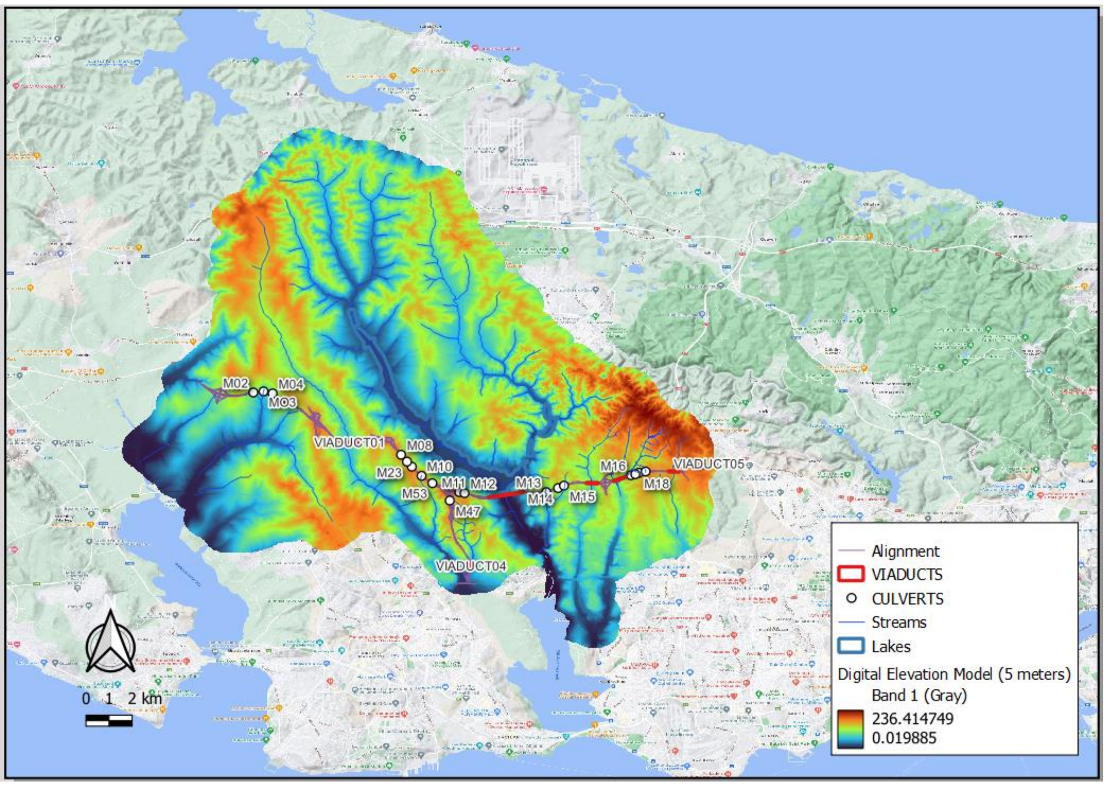


Figure 1-5 The Digital Elevation Model (DEM) prepared for the Study Area

1.5.2 Storms and Floods Data

The locations of meteorology (MGI) and hydrometric (AGI) stations are presented in Figure 1-6. The meteorology stations belong to Istanbul Greater Municipality Disaster Coordination General Directorate (Afet Koordinasyon Merkezi Mudurlugu AKOM), which measures the rainfall with 1 minute time interval, and the pluviograph stations of MGM record standard durations (5,10,30,45,60,120, 180, and …) are considered in this collection. The list of prepared stations is presented in Table 1-5. In addition, collected annual peak flow data of hydrometric stations near the study area is presented in Table 1-6.

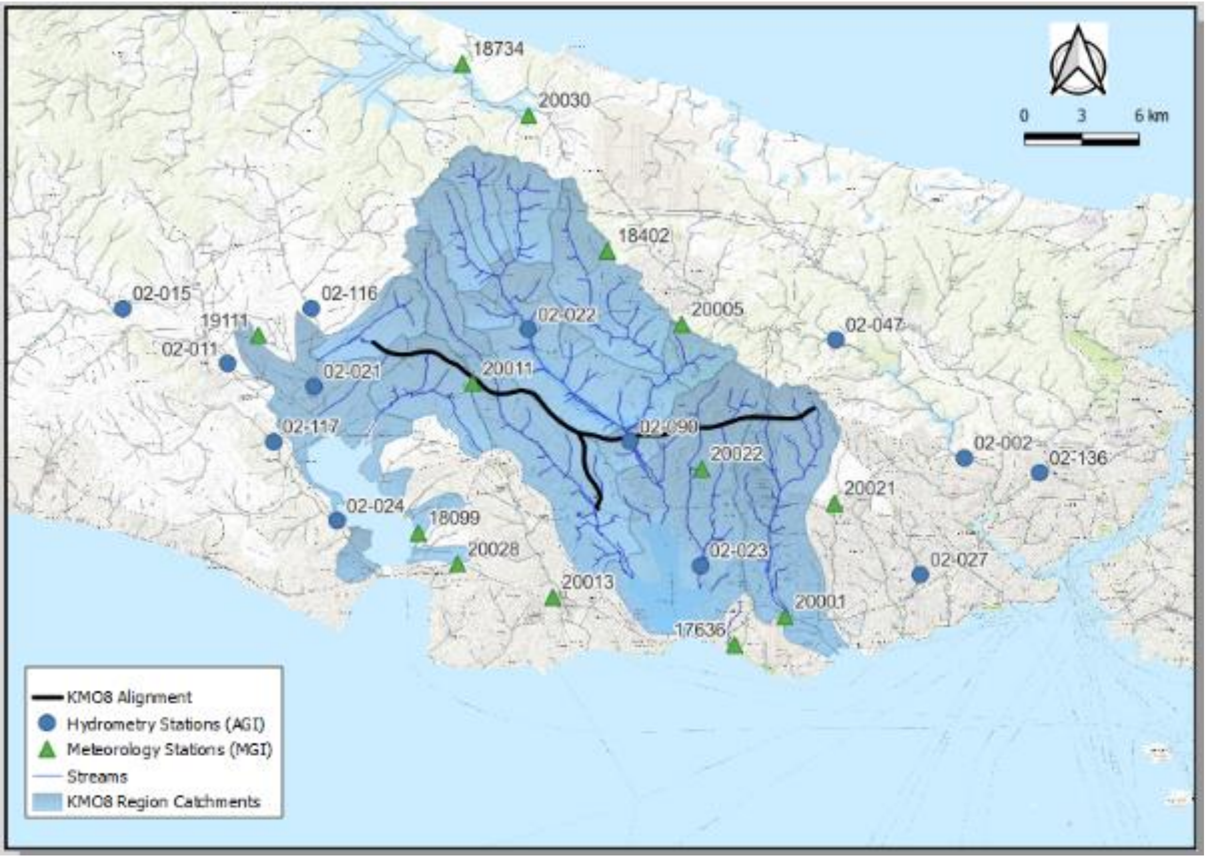


Figure 1-6 The Existing Meteorology and Hydrometric Stations for the Project Study Area (https://data.ibb.gov.tr/en/dataset/meteorology-observation-station-data-set)

Table 1-5 List of the Existing Meteorology Stations for the Project Study Area

Station Code	Station Name	Latitude	longitude	Elevation (m)
20022	Olimpiyat AKOM	41.085	28.7661	100
20030	Terkos AKOM	41.3044	28.6586	4
20011	Hadimköy AKOM	41.1383	28.624	183
20028	Svirajlari AKOM	41.0262	28.6144	165
18099	Büyüküekmece	41.0453	28.59	20
18402	Arnavutköy	41.2203	28.7075	140
20005	Arnavutköy AKOM	41.1747	28.7536	169
20021	Mahmutbey AKOM	41.0636	28.8485	76
18734	Arnavutköy/Terkos BARAJI	41.3364	28.6175	16
19111	Çatalca	41.16804	28.49087	78
20001	Ataturk Havalimani AKOM	40.9933	28.8178	12
17636	Florya	40.9758	28.7865	37
20013	Haramidere AKOM	41.0055	28.6736	61

Table 1-6 List of the Existing Hydrometric Stations for the Project Study Area

Station Code	Stream Name	Station Name	Operation Status*	Beginning Date	Closing Date	Station Elevation (m)	Upstream Catchment Area (km²)
02-116	Sarısu Dere	İzzettin	O	2/17/1994		8	84
02-117	Çakıl Dere	Ahmetli	O	5/9/1994		30	66
02-136	Kağıthane Dere	Kağıthane	O	7/2/1997		3	183
02-015	Karasu	İnceğiz	O	9/16/1965		30	175
02-022	Sazlıdere	Bosna	O	5/19/1905	7/1/1981	12	84
02-023	Nakkaş D.	Halkalı	C	5/19/1905	2/18/1993	2	44
02-024	Çakıldere	Tepecik	C	5/19/1905	6/8/1905	3	96
02-047	Malava D.	Pirinçköy	O	3/15/1969		30	112
02-090	SazlıDere	Kayabaşı	O	1/10/1984		5	136
02-027	Çavuşbaşı D.	Şirinevler	C	7/1/1966	4/1/1969	3	22
02-002	Alibey D.	Albayın Çiftliği	C	6/9/1960	4/1/1969	5	170
02-021	Sarısu	Bahşayış DDY Köp.	C	7/8/1966	12/31/1972	5	143

** O: open and C: closed*

1.5.3 Land Cover Data

Land cover data is used to estimate loss rate parameters in the runoff calculation and includes the land use map of CORINE 2018 and the hydrologic group of soils prepared from the soil map prepared by TOB. The digital map of land use and soil is presented in Figure 1-8.

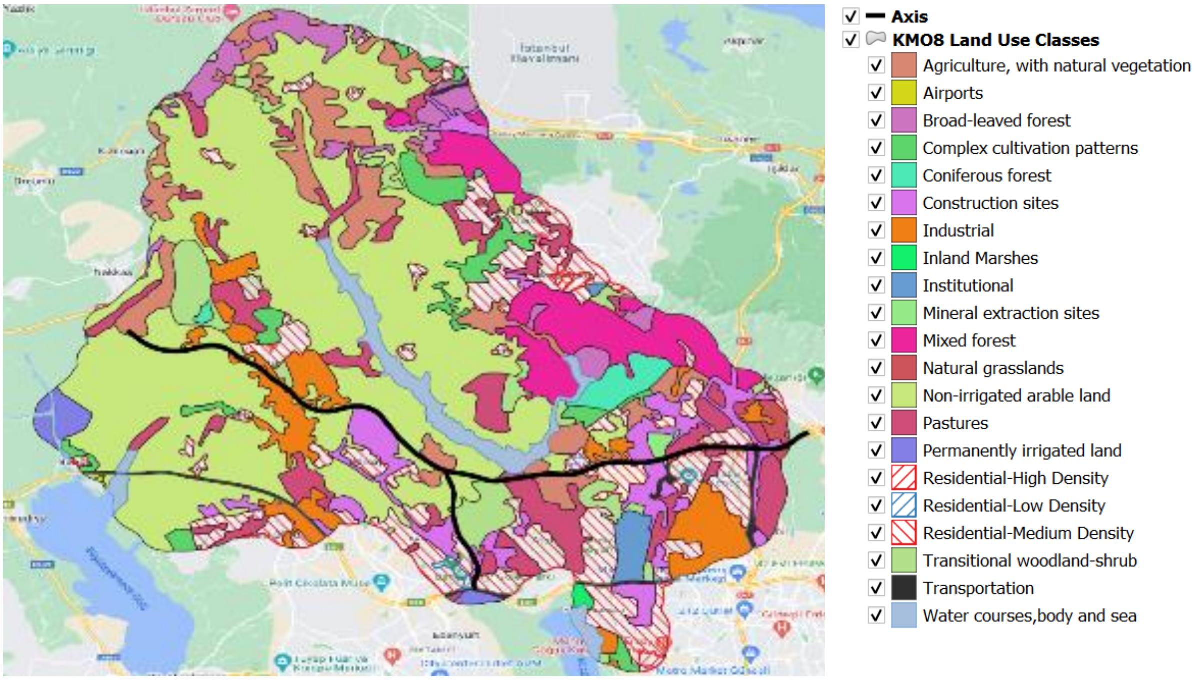


Figure 1-7 Digital Map of Land Use for the Project Study Area

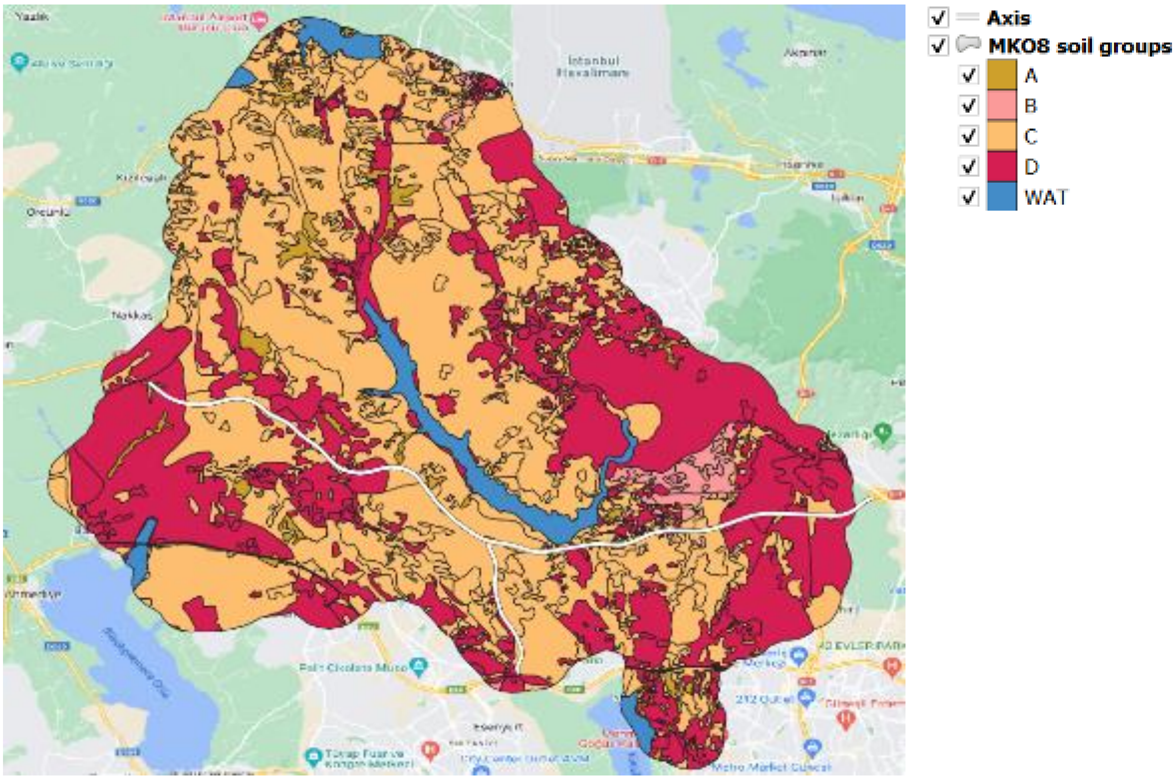


Figure 1-8 Digital Map of Soil for the Project Study Area

2. CATCHMENT ASSESSMENT FOR PEAK FLOW AND HYDROGRAPHS

2.1 Introduction

In this chapter, floods, peak flow, and hydrographs are calculated for the location of the sub-structures. The catchments area and characteristics are calculated initially, then the extreme storms on the catchments are extracted based on observed data of daily and hourly precipitation. Finally, the peak flow in the sub-structure location considering their catchment area is calculated by the rational or synthetic unit hydrograph method. These calculations are part of the inputs require to simulate water level elevations under extreme event conditions.

2.2 Catchments Delineation

Catchment delineation is one of the essential steps in hydrologic studies. The traditional manual catchment delineation method using topographic maps is time-consuming, and automated catchment delineation using Digital Elevation Model (DEM) is used in this study. Then the calculated catchment boundaries are controlled for urban area drainage and are modified limits based on the situation of roads and rainwater collection system. Automatic delineations are done by providing GIS tools in Hydrologic Modelling System (HMS) [3]for viaducts catchments and the QSWAT model for small culvert catchments. As mentioned in chapter one, a 5-meter Digital Elevation Model (DEM) of the study area from the Surveying General Administration (HGM) was prepared and used for catchment delineation (Figure 1-5). The catchment and stream networks' characteristics are also prepared using the tools. The result of catchments delineations for viaducts by the HMS model is shown in Figure 2-1. The calculation result after manual modification for urban areas for culverts is presented in Figure 2-3. In addition, the physiographic characteristic of the sub-structures is shown in Table 2-1.

[3] Model to simulate the complete hydrologic processes of dendritic watershed systems Hydrologic Modeling System (HEC-HMS) https://www.hec.usace.army.mil/software/hec-hms/

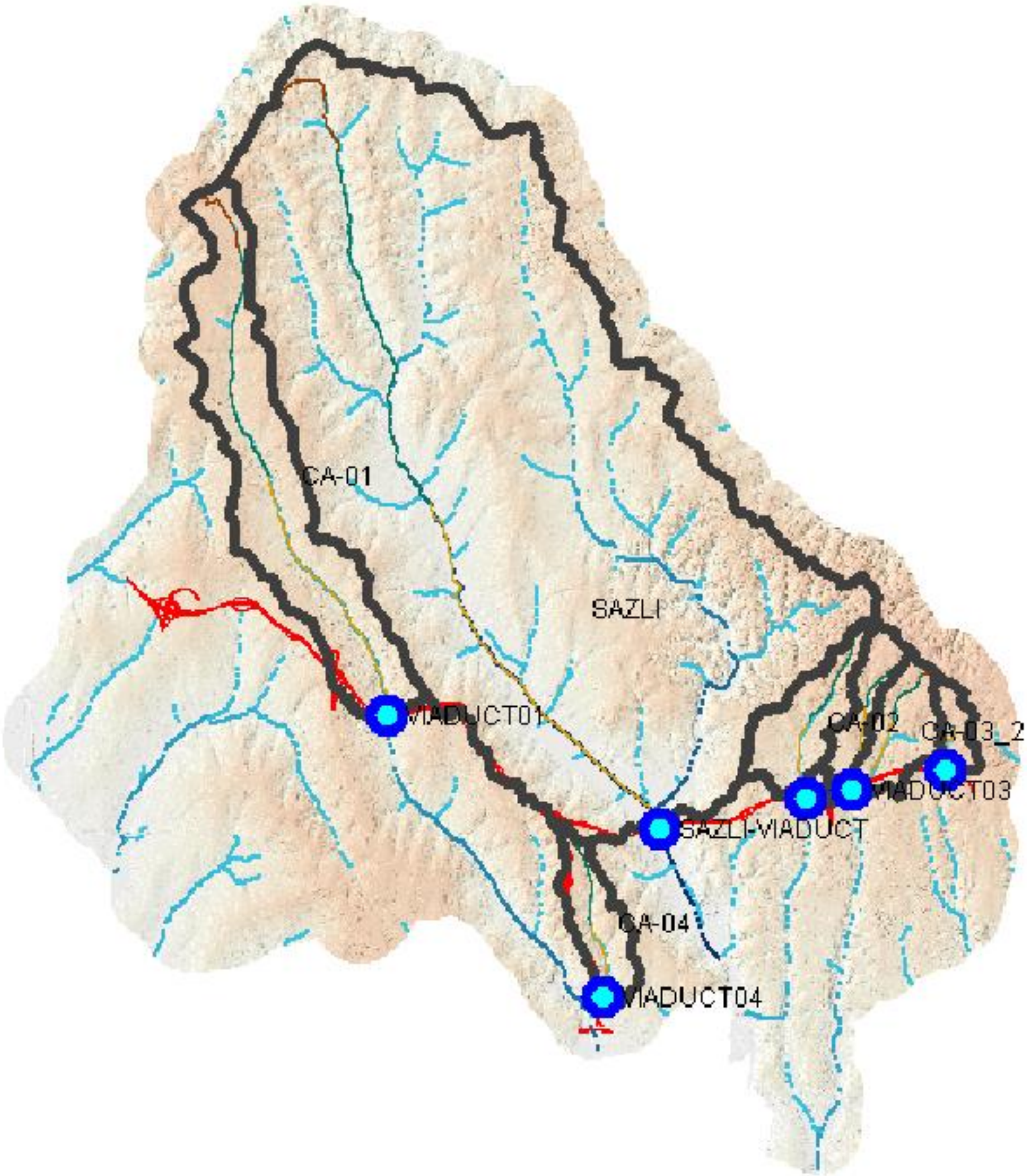
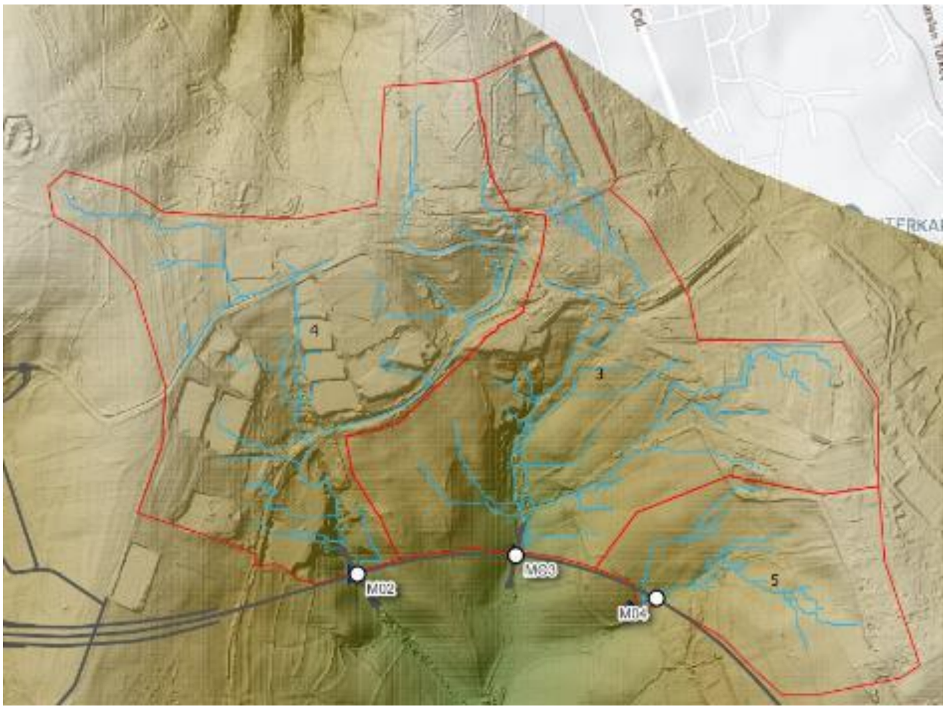


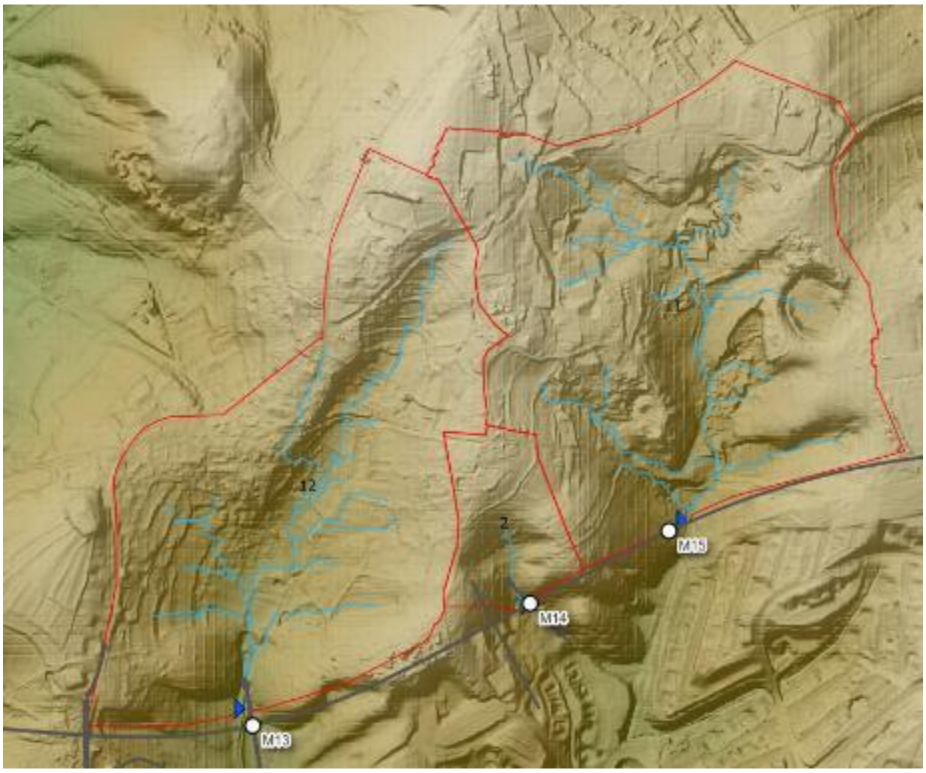
Figure 2-1 The Map of Delineated Catchment Boundary for Viaducts



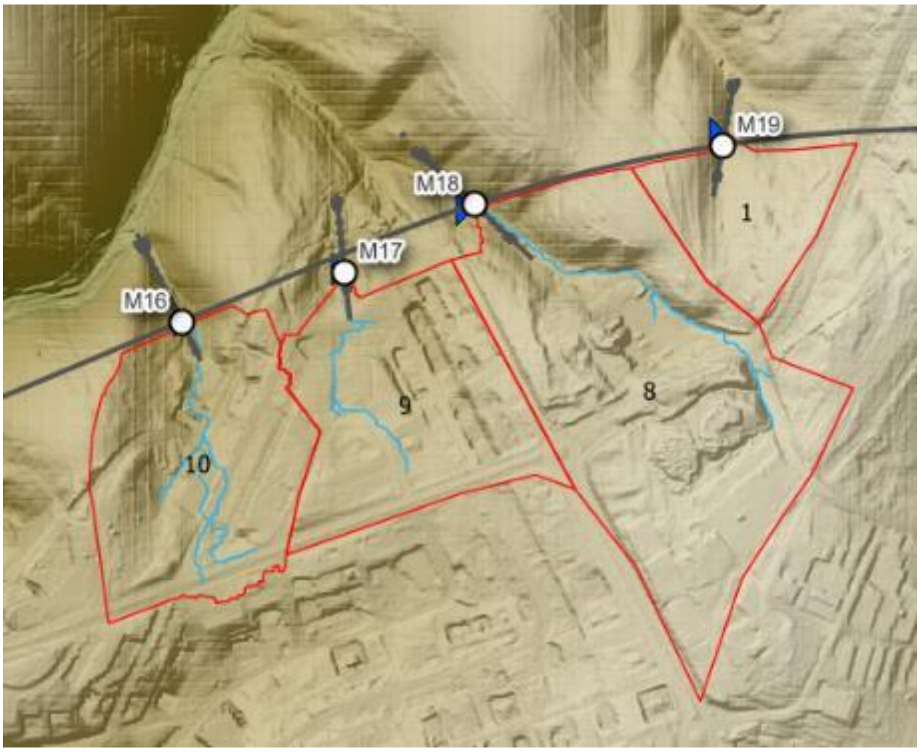
M02
M03
M04

M08
M23
M09
M10
M53
M47
M11
M12

Figure 2-2 The Delineated Catchment Boundary for Culverts



M13
M14
M15



M16
M17
M18
M19

Figure 2-3 The Delineated Catchment Boundary for Culverts (Continued)

Table 2-1 Calculated Physiographic Characteristics of the Catchments in the Crossing with Sub-Structures

Sub-structure	Catchment Area (km²)	Longest Flow Path Length (KM)	Longest Flow Path Slope (KM)	Centroid Flow Path Length (KM)	Catchment Relief (m)
Sazlıdere Cable Stayed Bridge	168.8400	25.9358	0.0063	11.4962	239
VIA-01	24.2440	16.6935	0.0085	7.6503	142
VIA-03_2	4.7547	4.3990	0.0317	2.0491	150
VIA-03_1	3.4251	5.4598	0.0265	2.5927	147
VIA-04	4.6000	5.4997	0.0282	2.1382	155
VIA-02	6.2533	5.8237	0.0251	2.1781	155
VIA-05	1.8927	3.3756	0.0238	1.3225	80
M02	0.877	2.2544	0.04658	-	105
M03	0.849	2.0457	0.05230	-	107
M04	0.28	0.8126	0.08984	-	73
M09	0.027	0.4670	0.05996	-	28
M10	0.035	0.3100	0.06129	-	19
M11	0.125	0.8276	0.08217	-	68
M12	0.11	0.5990	0.10017	-	60
M13	0.415	1.2984	0.05776	-	75
M14	0.039	0.3870	0.10078	-	39
M15	0.504	1.0889	0.05969	-	65
M16	0.086	0.4380	0.09132	-	40
M17	0.042	0.3461	0.08957	-	31
M18	0.095	0.5147	0.09132	-	47
M19	0.015	0.2500	0.09200	-	23

2.3 Storm Analysis

Extreme storms analysis is done for extracting storms height (mm) and intensity (mm/hours) for viaducts (bridges) and culverts in the study area. Before calculating the parameters, the concentration-time is considered for calculating the catchment storm's duration.

2.3.1 Storms Duration for Catchments

In this study, the parameter is calculated by extracted physiographic parameters based on the recommendation of the KGM standard for the time of concentration. Time of concentration can be divided into two parts: surface flowing time and channel flowing time.

$$t_c = t_s + t_{ch}$$

Where:

t_c: time of concentration for sub-structure (minutes)

t_s: time of storms flowing on the surface before receiving to channels (minutes)

t_{ch}: time of storms flowing in the channel before receiving to sub-structure (minutes)

The surface flowing time is considered by the length and velocity of water as the following equation:

$$t_s = \frac{LS}{60\,V}$$

Where:

LS: length of storm flowing on the surface (m)

V: velocity of surface flow (m/sec.)

Because of the effect of various surface roughness on the length and velocity of the surface flow, in this study, the surface runoff length assumes to be less than 150-300 meters, and the maximum velocity on the surface is assumed to be 0.5 m/sec, which the time assume 5 minutes for all catchments. Channel flowing time is calculated by the Kirpich method as the following equation:

$$t_{ch} = 0.0195(L^3/H)^{0.385}$$

Where:

L: length of storm flowing on the surface (m)

H: channel or catchment relief (m)

Based on mentioned assumptions and using the sub-structure catchments physiographic data, the time of concentration and storms duration were calculated for the catchments. For small catchments, 1.1 t_c is assumed as the duration of the storm. However, for viaducts catchment, it is assumed as $2\sqrt{t_c}$. The calculation result for the sub-structures catchments area is presented in Table 2-2.

2.3.2 Storms Height and Intensity

Storms height (mm) and intensity (mm/hours) are calculated based on observed data of short-term storms frequency analysis. In frequency analysis, the descriptive statistic parameters (mean, standard deviation, skewness, and kurtosis) are used for predicting the convenient distribution and storm amount for different return periods. Chow (1951) proposed using a frequency factor in hydrologic frequency analysis. In this procedure annual maximum storm of a station (P) is plotted to ascend, then the amount of storm with a return period of T; P_T , is found. In hydrology textbooks, the following statistic equation is established by using the mean, $\bar{P}$, and S as standard deviation of P and the frequency factor K_T:

Table 2-2 Result of Calculation for the Time of Concentration and Critical Duration of Storms

Sub-Structure	Longest Flow Path Length (KM)	Catch-ment Relief (m)	Overland Length	Overland Flow Time (minutes)	Channel Flow Time (minutes)	Time of Concen-tration (minutes)	Critical Storm Duration (minutes)
Sazlıdere Cable Stayed Bridge	25.9358	239	300	10	90	100	154.92
VIA-01	16.6935	142	300	10	66	76	135.06

Sub-Structure	Longest Flow Path Length (KM)	Catch-ment Relief (m)	Overland Length	Overland Flow Time (minutes)	Channel Flow Time (minutes)	Time of Concen-tration (minutes)	Critical Storm Duration (minutes)
VIA-03_2	4.3990	150	300	10	18	28	81.98
VIA-03_1	5.4598	147	300	10	18	28	81.98
VIA-04	5.4997	155	300	10	18	28	81.98
VIA-02	5.8237	155	300	10	24	34	90.33
VIA-05	3.3756	80	300	10	18	28	81.98
M02	2.2544	105	300	10	12	22	24.2
M03	2.0457	107	300	10	12	22	24.2
M04	0.8126	73	150	5	6	11	12.1
M09	0.4670	28	150	5	6	11	12.1
M10	0.3100	19	150	5	6	11	12.1
M11	0.8276	68	150	5	6	11	12.1
M12	0.5990	60	150	5	6	11	12.1
M13	1.2984	75	250	8	6	14	15.8
M14	0.3870	39	150	5	6	11	12.1
M15	1.0889	65	150	5	6	11	12.1
M16	0.4380	40	150	5	6	11	12.1
M17	0.3461	31	150	5	6	11	12.1
M18	0.5147	47	150	5	6	11	12.1
M19	0.2500	23	150	5	6	11	12.1

$$P_T = \bar{P} + K_T S$$

Where K_T depends on the return period T and the Probability Density Function (PDF); K_T means the number of standard deviations above and below the mean to achieve the desired quantile. For distribution, a relation between K_T and T can be derived for various distribution. From study area, only five stations data are reliable for analysis: Olimpiyat, Çanta, Kağithane and Teros belong to AKOM with about 15 years of data, and Florya plviugraph belongs to MGM with 75 years of data. The maximum duration for storms analysis was selected as 4 hours proportional to catchments time of concentration. The descriptive statistic parameters of storms in the station with various duration are presented in Table 2-3. Using Gumbel distribution, the storms with 100, 200, and 500 years return periods were calculated for the stations, and their cover value (maximum) is used for the study area. The results of the calculations are presented in Table 2-4. The summary of storms' height and average intensity is shown in Table 2-5, Figure 2-4, and Figure 2-5. In addition, based on critical storm duration, the height and average intensity of the design storms are calculated and presented in Table 2-6.

Table 2-3 Calculated Descriptive Parameters of Storms in the Recording Meteorology Stations

Duration	Description	Stations Name				
		Florya	Kağıthane	Çanta	Olimpiyat	Terkos
5 Minutes	**Maximum**	16.9	13.0	12.8	11.0	13.0
	Average	6.5	6.1	5.6	5.7	6.5
	Standard Deviation	2.9	3.1	3.5	3.2	3.7
	Skewness	1.1	0.4	0.9	-0.1	-0.1
	Kurtosis	2.3	0.6	0.2	-0.8	-0.8
10 Minutes	**Maximum**	21.0	19.4	20.6	15.8	20.2
	Average	9.4	10.1	7.9	8.6	9.4
	Standard Deviation	4.4	5.1	5.4	5.1	5.5
	Skewness	0.7	-0.1	1.2	0.0	0.1
	Kurtosis	-0.1	-0.2	1.3	-1.0	-0.3
15 minutes	**Maximum**	29.2	22.4	25.0	21.2	21.2
	Average	11.4	11.0	9.7	10.4	11.3
	Standard Deviation	5.4	6.0	7.3	6.4	6.5
	Skewness	0.8	0.2	1.2	0.5	-0.2
	Kurtosis	0.6	-0.5	0.3	-0.6	-1.1
30 Minutes	**Maximum**	38.0	24.4	42.6	41.8	29.8
	Average	15.3	14.0	13.4	16.1	16.7
	Standard Deviation	7.2	7.6	10.8	11.9	9.2
	Skewness	0.7	0.1	1.7	1.0	-0.4
	Kurtosis	0.1	-1.2	3.2	0.2	-1.3
1 Hour	**Maximum**	42.9	33.0	83.2	56.4	44.8
	Average	18.8	18.0	19.5	21.5	23.2
	Standard Deviation	8.4	9.2	20.8	16.7	13.2
	Skewness	0.8	-0.1	2.5	1.1	-0.3
	Kurtosis	0.1	-1.3	7.2	0.3	-1.2
2 Hours	**Maximum**	58.2	37.0	103.6	66.6	61.2
	Average	22.8	20.6	23.1	25.4	27.3
	Standard Deviation	10.8	10.1	25.1	19.9	15.7
	Skewness	1.1	0.1	2.9	1.3	0.4
	Kurtosis	1.1	-0.9	9.4	0.9	0.2
3 Hours	**Maximum**	62.4	37.2	103.8	75.8	65.4
	Average	25.5	22.2	26.0	27.2	33.1
	Standard Deviation	12.9	9.4	24.5	21.2	19.3

Duration	Description	Stations Name				
		Florya	Kağıthane	Çanta	Olimpiyat	Terkos
	Skewness	1.3	-0.2	2.8	1.5	0.0
	Kurtosis	1.1	-0.3	8.8	1.3	-1.2
4 Hours	Maximum	67.8	44.6	110.0	78.0	63.2
	Average	27.8	24.3	27.9	28.8	33.3
	Standard Deviation	14.2	11.2	25.9	21.8	19.1
	Skewness	1.2	0.0	2.7	1.5	0.0
	Kurtosis	1.0	-0.7	8.8	1.6	-1.3

Table 2-4 Calculated Design Storms for the Stations

Duration	Return Period (Years)	Station					Study Area
		Florya	Kağıthane	Çanta	Olimpiyat	Terkos	
5 Minutes	100	15.7	20.5	22.8	18.9	19.0	22.8
	200	17.3	22.7	25.5	21.0	20.9	25.5
	500	19.4	25.6	29.1	23.7	23.4	29.1
10 Minutes	100	23.3	26.0	28.7	29.7	28.0	29.7
	200	25.7	28.6	32.0	32.9	30.8	32.9
	500	28.8	32.1	36.3	37.2	34.5	37.2
15 minutes	100	28.2	31.5	33.8	36.0	33.5	36.0
	200	31.1	34.6	37.6	39.8	36.9	39.8
	500	35.0	38.6	42.5	44.8	41.3	44.8
30 Minutes	100	38.0	40.0	48.8	55.6	48.0	55.6
	200	41.9	43.9	54.3	61.7	52.7	61.7
	500	47.1	49.0	61.6	69.7	58.8	69.7
1 Hour	100	45.2	49.8	71.5	78.2	68.0	78.2
	200	49.8	54.5	79.7	86.9	74.8	86.9
	500	55.8	60.7	90.4	98.3	83.6	98.3
2 Hours	100	56.6	71.4	103.0	90.9	80.8	103.0
	200	62.5	79.2	115.8	101.1	88.8	115.8
	500	70.2	89.6	132.7	114.5	99.4	132.7
3 Hours	100	65.9	81.3	104.9	96.9	98.6	104.9
	200	72.9	90.5	117.4	107.8	108.4	117.4
	500	82.1	102.7	133.9	122.1	121.4	133.9
4 Hours	100	72.2	83.9	111.4	100.7	98.4	111.4
	200	79.9	93.1	124.6	111.8	108.1	124.6
	500	90.0	105.3	142.1	126.5	121.0	142.1

Table 2-5 Calculated Height and Average Intensity of Storms

Duration (minutes)	Parameter	Return Periods (Years)		
		100	200	500
5	**Rainfall height (mm)**	22.8	25.5	29.1
10		29.7	32.9	37.2
15		36.0	39.8	44.8
30		55.6	61.7	69.7
60		78.2	86.9	98.3
120		95.0	107.0	123.0
180		104.9	117.4	133.9
240		111.4	124.6	142.1
5	**Rainfall average intensity (mm/hours)**	273.3	305.9	348.8
10		178.0	197.5	223.3
15		143.8	159.0	179.1
30		111.2	123.4	139.5
60		78.2	86.9	98.3
120		47.5	53.5	61.5
180		35.0	39.1	44.6
240		27.9	31.2	35.5

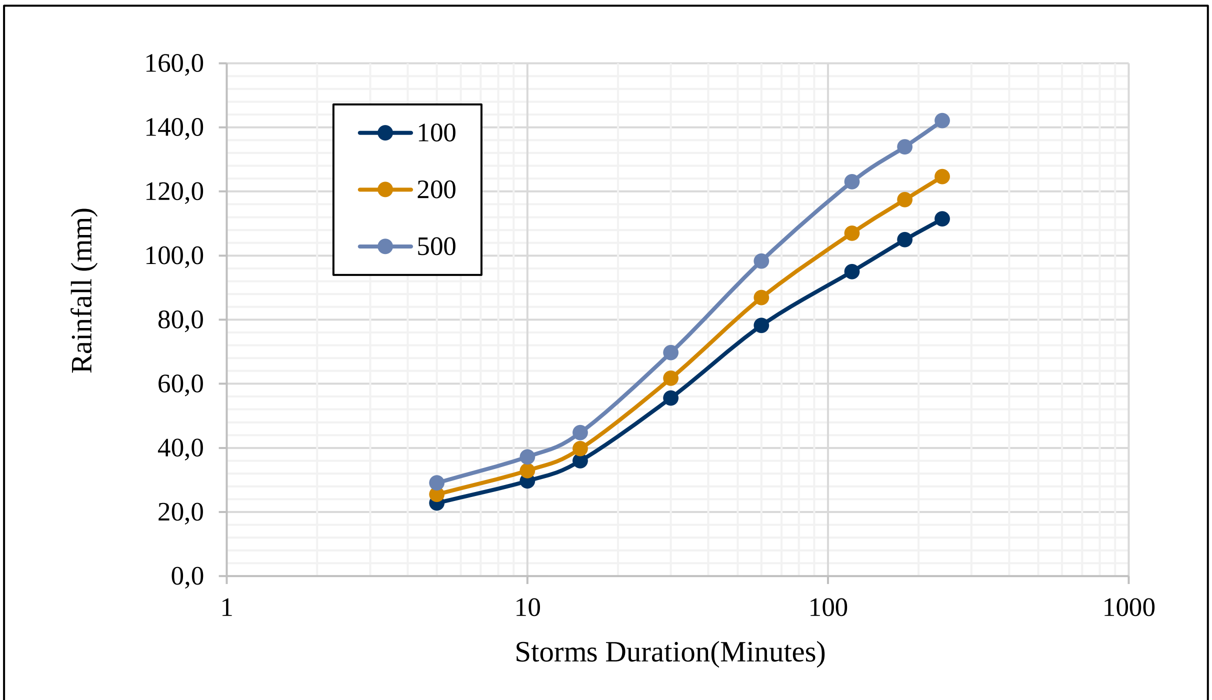


Figure 2-4 Height of Design Storms of 100, 200, and 500 Years

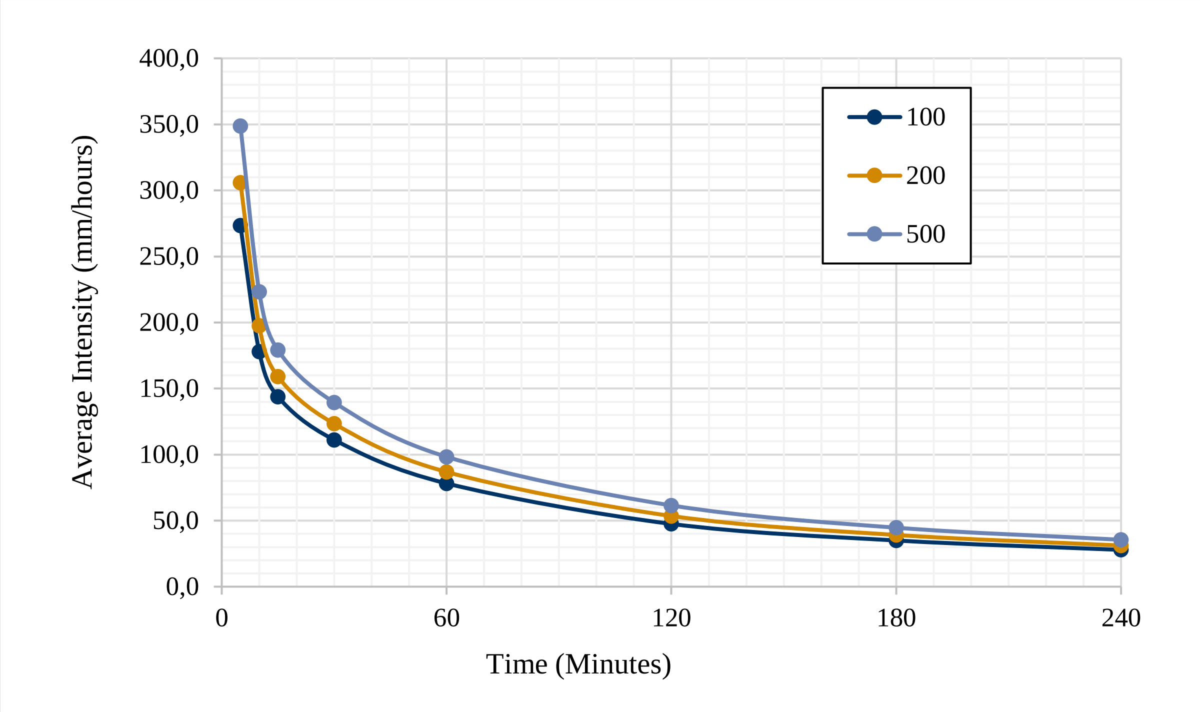


Figure 2-5 Intensity-Duration Curves for 100, 200- and 500-Years Design Storms

Table 2-6 Calculated Height and Average Intensity of Storms for Structures

Sub-Structure	Catchment Area (km²)	Storm Duration (minutes)	Area Reduction Factor	Rainfall Height (mm) or Intensity (mm/hours)			Explanation
				100	200	500	
Sazlıdere Cable Stayed Bridge	168.8400	150.00	0.80	80.0	89.8	102.8	height (mm)
VIA-01	24.2440	135.00	0.96	93.6	105.2	120.7	height (mm)
VIA-03_2	4.7547	80.00	0.98	82.1	91.6	104.3	height (mm)
VIA-03_1	3.4251	80.00	0.98	82.1	91.6	104.3	height (mm)
VIA-04	4.6000	80.00	0.98	82.1	91.6	104.3	height (mm)
VIA-02	6.2533	90.00	0.97	84.0	94.0	107.3	height (mm)
VIA-05	1.8927	85.00	0.99	84.4	94.4	107.6	height (mm)
M02	0.8770	25.0	1.0	121.9	135.1	152.5	intensity (mm/hours)
M03	0.8490	25.0	1.0	121.9	135.1	152.5	intensity (mm/hours)
M04	0.2800	15.0	1.0	143.8	159.0	179.1	intensity (mm/hours)
M09	0.0270	15.0	1.0	143.8	159.0	179.1	intensity (mm/hours)

Sub-Structure	Catchment Area (km²)	Storm Duration (minutes)	Area Reduction Factor	Rainfall Height (mm) or Intensity (mm/hours)			Explanation
				100	200	500	
M10	0.0350	15.0	1.0	143.8	159.0	179.1	intensity (mm/hours)
M11	0.1250	15.0	1.0	143.8	159.0	179.1	intensity (mm/hours)
M12	0.1100	15.0	1.0	143.8	159.0	179.1	intensity (mm/hours)
M13	0.4150	15.0	1.0	143.8	159.0	179.1	intensity (mm/hours)
M14	0.0390	15.0	1.0	143.8	159.0	179.1	intensity (mm/hours)
M15	0.5040	15.0	1.0	143.8	159.0	179.1	intensity (mm/hours)
M16	0.0860	15.0	1.0	143.8	159.0	179.1	intensity (mm/hours)
M17	0.0420	15.0	1.0	143.8	159.0	179.1	intensity (mm/hours)
M18	0.0950	15.0	1.0	143.8	159.0	179.1	intensity (mm/hours)
M19	0.0150	15.0	1.0	143.8	159.0	179.1	intensity (mm/hours)

2.4 Regional Peak Flow Analysis

The observed peak flow of the existing hydrometric stations around the study area can be used for extracting the required design food for the sub-structures. The stations' recorded annual daily peak flow is extracted and analyzed to determine regional floods peak amount with 100, 200, and 500 years return periods (1, 0.5, and 0.2 percent of risk) on the sub-structures' sites. The records are extracted from 1966 to 2018 by downloading their daily data from the state Hydraulic Works (DSI) website and sparse data of projects in the study area. The primary statistic properties of the annual daily peak flow of the hydrometric stations are computed and presented in Table 2-7. The data have non-zero skewness and kurtosis, which shows that they have not followed Normal distribution, and maybe their best fitting will be Gumbel or Log Pearson Type III. Therefore, this study establishes a relation between catchment area and annual average and standard division of station peak flow. Then by extracting K_T for desired return period, the design flood peak is calculated for the sub-structures. In this study considering skewness and kurtosis, the Gumbel Type I with following K_T is used in calculation for T return period:

$$K_T = (\frac{\sqrt{6}}{\pi})(0.57721 - Ln(Ln\left(\frac{T}{T-1}\right)))$$

The simple relationship between the mean and standard division of annual peak flow and upstream catchment area of the stations are shown in Figure 2-6 and. The relations selected for stations with more than 20 years of records and stations with large catchment areas were removed from the regional analysis. The mean and standard division of annual peak flow is calculated for sub-structures from the relations. In addition, K_T of Gumbel distribution for 100, 200 and 500 years are used for computation of design floods and results presented in Table 2-8. In this method, the design floods values are calculated for viaducts. This method results are under-design for culverts with small catchments, respectively.

Table 2-7 Summary of Statistics Descriptions of Annual Peak Flow for Selected Stations

Station Name	İnceğiz	Bahşayış DDY Köp.	Bosna	Halkalı	Tepecik	Pirinçköy	Ahmediye	Kağıthane
Stream Name	Karasu	Sarısu	Sazlıdere	Nakkaş Dere.	Çakıldere	Malava D.	Çakil Dere.	Kağıthane Dere
Station Code	D02A015	D02A021	D02A022	D02A023	D02A024	D02A047	D02A117	D02A136
Catch-ment Area (km^2)	174.9	143.0	84.0	44.0	133.0	111.8	55.0	182.8
Maximum	350.0	91.0	190.0	34.0	41.0	165.0	45.0	119.0
Average	79.7	30.6	47.9	11.1	17.6	59.7	23.4	67.6
Minimum	5.7	3.0	10.0	0.2	4.6	8.1	4.9	26.4
Standard Division	67.0	30.7	33.1	10.1	9.6	38.0	13.7	32.6
Skewness	1.9	0.9	2.9	1.4	0.9	1.0	0.3	0.4
Kurtosis	5.1	-0.9	11.1	0.8	0.2	0.5	-1.3	-1.3

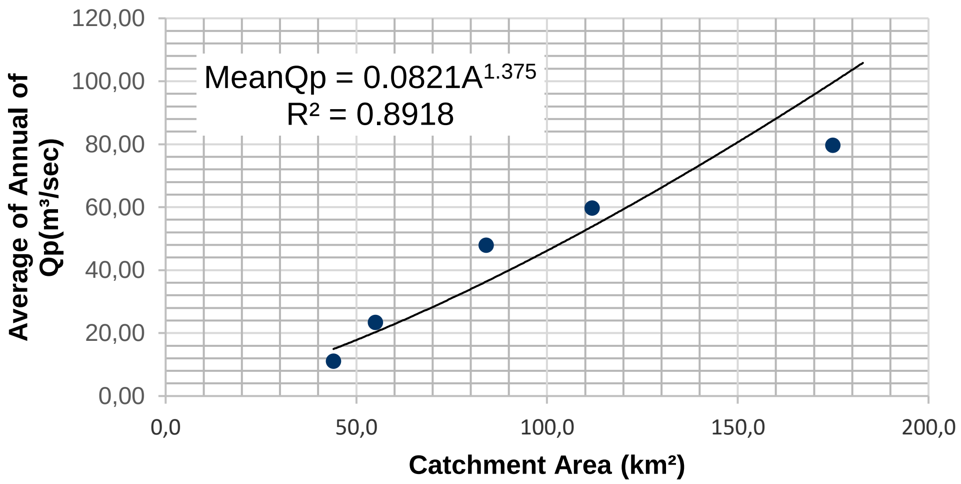


Figure 2-6 Relationship of the Average annual Peak Flow and Catchment Area of Hydrometric Stations

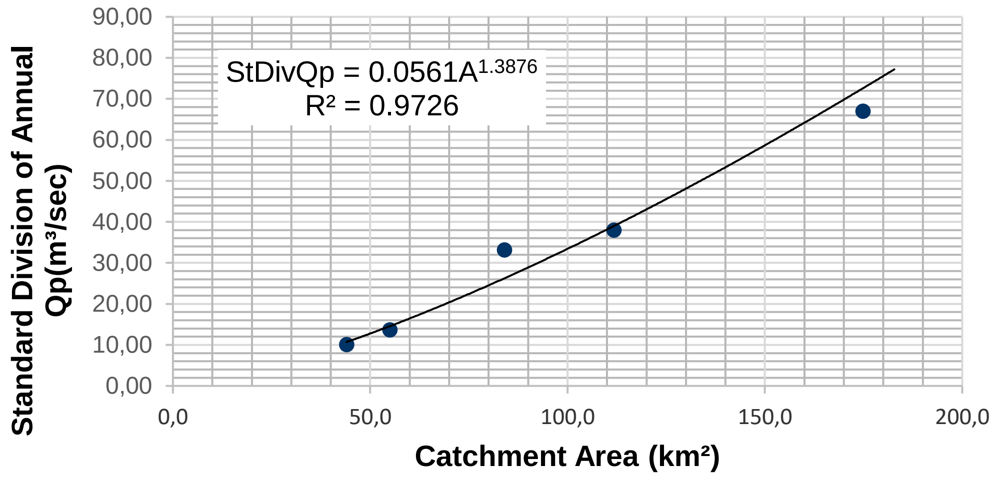


Figure 2-7 Relationship of the Average Annual Peak Flow and Catchment Area of Hydrometric Stations

Table 2-8 Design Flood Calculation for Sub-Structures (Viaducts)

Sub-Structure	Catchment Area (km²)	Average of annual peak flow (m³/sec)	Standard Division of Annual Peak Flow (m³/sec)	Peak Flows (m³/sec) for the Return Periods (Years)		
				100	200	500
Sazlıdere Cable Stayed Bridge	168.84	94.8684	69.1525	374.03	411.54	461.03
VIA-01	24.244	6.5792	4.6799	25.47	28.01	31.36
VIA-03_2	4.7547	0.7005	0.4881	2.67	2.94	3.29
VIA-03_1	3.4251	0.4462	0.3097	1.70	1.86	2.09
VIA-04	4.6	0.6693	0.4662	2.55	2.80	3.14
VIA-02	6.2533	1.0209	0.7139	3.90	4.29	4.80
VIA-05	1.8927	0.1974	0.1360	0.75	0.82	0.92

2.5 Rainfall-Runoff Model

As mentioned in the methodology, in the lack of convenient long-term observed peak and flood data, flood calculation is done by rainfall-based methods. Based on the TGM manual, the rational method is used for flood peak calculation in small catchments (with less than 1 km² area); however, the Synthetic Unit Hydrograph method is recommended for greater catchments.

2.5.1 Rational Method

The Rational method is appropriate for estimating peak flow for small catchments of culverts as mentioned in the KGM manual for less than 1 km² or 100 hectares. The rational formula estimates the peak flow as a function of the drainage area, runoff coefficient, and mean rainfall intensity for a duration equal to the time of concentration. The rational formula is:

$$Q = \frac{CiA}{3.60}$$

Where:

Q: peak flow (m^3/sec.)

C: runoff coefficient

i: average rainfall intensity (mm/hr.)

A: catchment area (km^2)

The runoff coefficient and the intensity of the average storm are calculated in the following sections.

2.5.2 Runoff Coefficients

The runoff coefficient is presented as a table in the KGM guideline as shown in Table 2-9.

Table 2-9 The Runoff Coefficient for Various Land Use Conditions

Road Platform	C Runoff Coefficients
Road Platform and Paved Areas	0.9
High Slope Cut or Fill Slopes (α>45°)	0.8
Low Slope Cut or Fill Slopes (α ≤45°)	0.5
Regulated Low Slope Areas (Refuge etc.)	0.3
Rural Basins	
Impermeable	0.90-0.95
Flat-Bare	0.80-0.90
Wavy- Bare	0.60-0.80
Soft- Bare	0.50-0.70
Wavy-Meadow	0.40-0.65
Deciduous Forest	0.35-0.60
Pine Forest	0.25-0.50
Fruit Wooded	0.15-0.40
Agricultural Land	0.15-0.40
Urban Basins	
Dense and Continuously Built-Up Urban Area	0.80-0.90
Commercial/Urban Area, Near Construction	0.70-0.85
Urban Housing Area, Limited Gardens	0.45-0.75
Residential Area with Garden in the Suburban	0.35-0.65
Entirely Built Suburban on a Sand Layer	0.25-0.55
Park Garden and Meadows	0.15-0.45

This table is convenient for constant land use conditions; however, the development condition is shortly assumed for the small catchment of this area. For the medium condition of the urban area, the coefficient is assumed to be 0.75. Therefore, having this coefficient and storms intensity from previous sections (Table 2-6), the peak flow value for culverts is extracted and presented in Table 2-10.

Table 2-10 Calculated Peak Flow of 100-, 200- and 500-Years Floods in the Location of Culverts

Sub-Structure	Catchment Area (km²)	Rainfall Height (mm) or Intensity (mm/hours)			Design Floods Peak Flow (m³/sec)		
		100	200	500	100	200	500
M02	0.8770	121.9	135.1	152.5	22.28	24.69	27.87
M03	0.8490	121.9	135.1	152.5	21.57	23.90	26.98
M04	0.2800	143.8	159.0	179.1	8.39	9.28	10.44
M09	0.0270	143.8	159.0	179.1	0.81	0.89	1.01
M10	0.0350	143.8	159.0	179.1	1.05	1.16	1.31
M11	0.1250	143.8	159.0	179.1	3.75	4.14	4.66
M12	0.1100	143.8	159.0	179.1	3.30	3.64	4.10
M13	0.4150	143.8	159.0	179.1	12.44	13.75	15.48
M14	0.0390	143.8	159.0	179.1	1.17	1.29	1.45
M15	0.5040	143.8	159.0	179.1	15.10	16.70	18.80
M16	0.0860	143.8	159.0	179.1	2.58	2.85	3.21
M17	0.0420	143.8	159.0	179.1	1.26	1.39	1.57
M18	0.0950	143.8	159.0	179.1	2.85	3.15	3.54
M19	0.0150	143.8	159.0	179.1	0.45	0.50	0.56

2.5.3 Synthetic Unit Hydrograph Method

This method is used for ungagged catchments areas greater than one square kilometer. The specific flow of one-millimeter runoff is calculated based on catchment physiographic parameters. Then excess rainfall (rainfall minus loss rate) of the catchment is used for calculation time and peak flow of flood hydrograph. A shape of the hydrograph is recommended in the KGM to generate a flood hydrograph. The peak flow formulas are:

$$Q = \frac{A q_p h_a}{1000}$$

Where:

Q: peak flow (m³/sec.)

A: catchment area (km²)

q_p: is the specific flow rate of the unit hydrograph (m³/sec/ km²) and calculated as the following formula:

$$q_p = \frac{414}{\left[A^{0.225} \left(\frac{LL_c}{\sqrt{S}}\right)^{0.16}\right]}$$

The parameters in this formula were explained in the previous Section. The following formula calculates the peak flow of the unit hydrograph:

$$Q_p = A\, q_p$$

h_a: excess rainfall height (mm) and calculated as the following formula:

$$h_a = \frac{(H_y - 0.2\, SC)^2}{(H_y + 0.8\, SC)}$$

Where:

H_y: rainfall height (mm)

SC: storage of catchment land cover (mm) calculated using curve number as the following formula:

$$SC = \frac{25400}{CN} - 254$$

The value of CN was calculated for land cover and hydrologic soil groups, which are presented in the following table for the various land cover of the study area.

The following equation also calculates the time to peak flow for unit hydrograph:

$$T_p = 0.73\, \frac{1000\, A}{Q_p}$$

The synthetic unit hydrographs parameters are calculated and presented in Table 2-11.

Table 2-11 Synthetic Unit hydrograph Parameters for Viaducts Catchments

Sub-Structure	LLc/sqrt(S)	qp (m³/sec/km²)	Qp (m³/sec)	Tp (hours)	Tb (hours)
Sazlıdere Cable Stayed Bridge	3762	34.974	5.90	5.80	29.00
VIA-01	1386	63.501	1.54	3.20	16.00
VIA-03_2	51	155.576	0.74	1.40	7.00
VIA-03_1	87	153.616	0.53	1.40	7.00
VIA-04	70	148.812	0.68	1.40	7.00
VIA-02	80	135.946	0.85	1.50	7.50
VIA-05	29	209.314	0.40	1.00	5.00

The dimensionless shape of the unit hydrograph for catchments was extracted from the KGM manual. The loss rate is calculated based on various land use curve numbers. The calculation is done based on Table 2-12 and by the intersection of digital maps of soil and land use. For each catchment, the combination of soil and land cover is calculated in the GIS environment and then the value of CN is extracted for the catchments. The calculated CN is increased by 20 percent (limited to 98) for considering the urban area and impervious surface development in the future and used for runoff calculation. In addition, 2 mm is assumed as baseflow of hydrographs.

Table 2-12 Land Cover Parameters for Calculation Loss Rate in Synthetic Unit Hydrograph Method

Row	Land Use	Impervious Ratio	Surface Roughness	Curve Number			
				A	B	C	D
1	Agricultural Land-Generic	0.05	0.14	67	77	83	87
2	Baren or Sparsely Vegetated	0.12	0.15	39	61	74	80
3	Dryland Cropland and Pasture	0.05	0.15	58	73	81	86
4	Cropland/Grassland Mosaic	0.05	0.15	58	73	81	86
5	Irrigated Cropland and Pasture	0.05	0.15	58	73	81	86
6	Deciduous Broadleaf Forest	0.05	0.1	45	66	77	83
7	Forest-Deciduous	0.05	0.1	45	66	77	83
8	Forest-Mixed	0.05	0.1	36	60	73	79
9	Vineyard	0.1	0.14	45	66	77	83
10	Grassland	0.05	0.15	49	69	79	84
11	Orchard	0.1	0.15	45	66	77	83
12	Pasture	0.05	0.15	49	69	79	84
13	Rice	0.05	0.14	62	73	81	84
14	Bare Ground Tundra	0.05	0.13	35	60	73	80
15	Wooded Tundra	0.05	0.13	35	60	73	80
16	Airports	0.98	0.015	98	98	98	98
17	Industrial	0.84	0.015	81	88	91	93
18	Institutional	0.51	0.015	68	79	86	89
19	Mineral Extraction Sites	0.38	0.015	81	88	91	93
20	Port Areas	0.98	0.015	98	98	98	98
21	Construction Sites	0.38	0.015	72	82	87	89
22	Residential-High Density	0.6	0.015	77	85	90	92
23	Residential-Low Density	0.12	0.015	51	68	79	84
24	Residential-Medium Density	0.38	0.015	61	75	83	87
25	Transportation	0.98	0.015	98	98	98	98
26	Water	0.98	0.01	92	92	92	92
27	Wetlands-Mixed	0.12	0.05	49	69	79	84

Table 2-13 Land Cover Parameters for Calculation Loss Rate in Synthetic Unit Hydrograph Method

Sub-Structure	Hy(Mm)			Loss Rate Parameters		Ha(Mm)		
	100	200	500	CN	Ia (mm)	100	200	500
Sazlıdere Cable Stayed Bridge	80.0	89.8	102.8	98.0	1.037	74.07	83.84	96.80
VIA-01	93.6	105.2	120.7	98.0	1.037	87.64	99.25	114.70
VIA-02	84.0	94.0	107.3	96.4	1.906	73.55	83.48	96.69
VIA-03_1	82.1	91.6	104.3	98.0	1.037	76.15	85.70	98.36
VIA-03_2	82.1	91.6	104.3	98.0	1.037	76.15	85.70	98.36
VIA-04	82.1	91.6	104.3	94.1	3.189	65.61	74.94	87.37
VIA-05	84.4	94.4	107.6	98.0	1.037	78.48	88.42	101.62

For a duration of more than one hour, the rainfall is distributed into sequence 1-hour height of rainfall considering the loss rate to calculate the height of excess rainfall. The accumulated flood hydrograph is calculated, considering 1-hour delays for sequence excess rainfall heights and unit hydrograph. The calculated hydrographs for the location of the viaducts are presented in Figure 2-8 and Figure 2-9. The peak flow of design floods in the location of the viaducts is summarized in Table 2-14.

Table 2-14 Summary of the Calculated Peak Flow of Design Floods by Synthetic Unit Hydrograph

Sub-Structure	Hydrograph Peak flow (m3/s)		
	100	200	500
Sazlıdere Cable Stayed Bridge	434.2	491.4	567.36
VIA-01	134.0	151.8	175.37
VIA-02	62.2	70.6	81.72
VIA-03_1	39.8	44.8	51.45
VIA-03_2	56.0	63.0	72.33
VIA-04	44.7	51.0	59.46
VIA-05	30.9	34.8	40.02

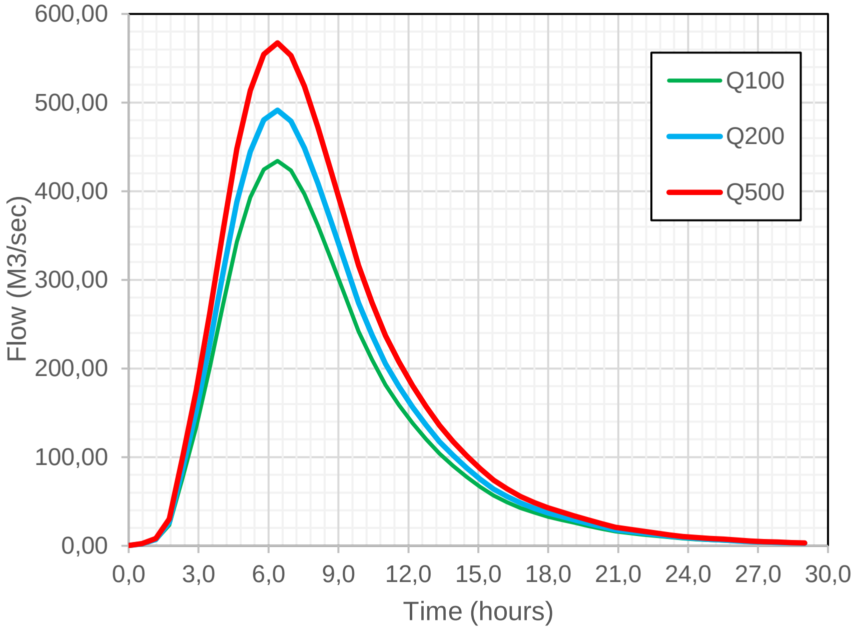
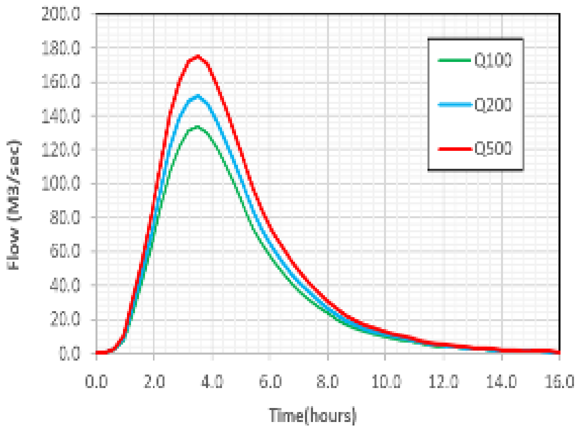
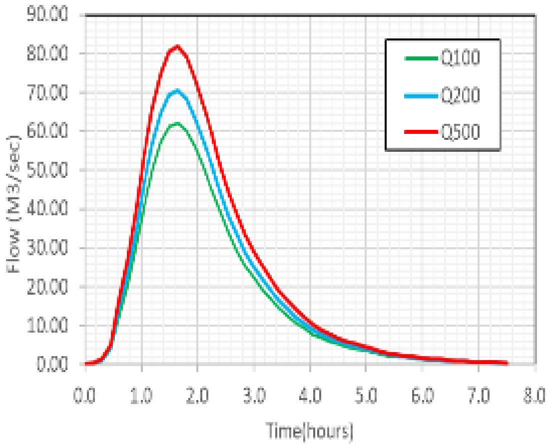


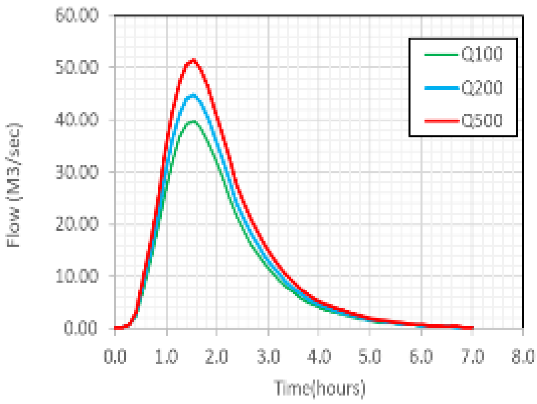
Figure 2-8 Natural Design Flood Hydrographs for Viaduct of Sazlıdere Downstream of the Reservoir of Sazlıdere Dam



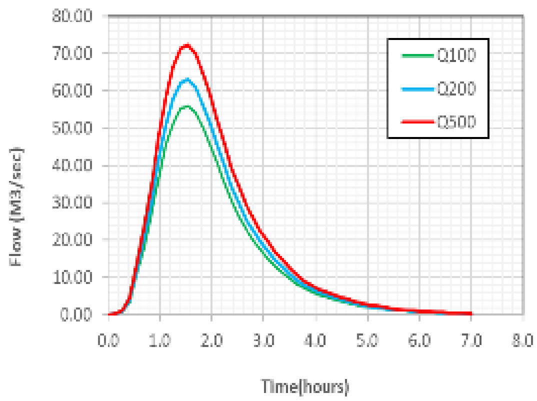
Viaduct 01



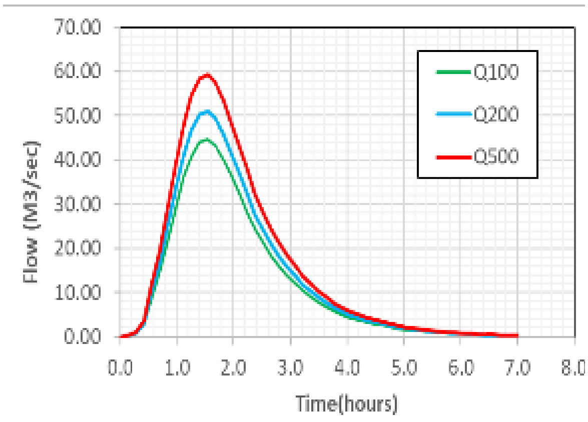
Viaduct 02



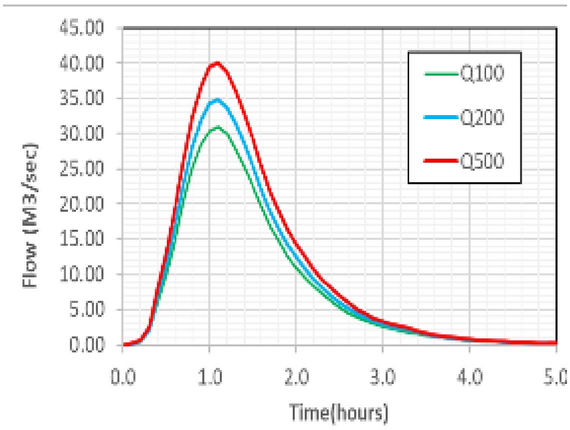
Viaduct 03_1



Viaduct 03_2



Viaduct 04



Viaduct 05

Figure 2-9 Design Flood Hydrographs for Viaducts 01 to 05

2.6 Selected Design Floods

Two methods were used to calculate viaducts to design floods. The results showed that regional analysis under-estimated the flooding conditions for viaducts. For this reason, the synthetic unit hydrograph result was selected for the next analysis of the water surface profile. In addition, the result of the rational method is selected for culverts. The selected design peak flow is presented in Table 2-15.

Table 2-15 Selected Peak Flow for the Sub-Structures

Sub-Structure	Design Peak Flow (m³/sec)		
	100-yr	200-yr	500-yr
Sazlıdere Cable Stayed Bridge	434.2	491.4	567.36
VIA-01	134.0	151.8	175.37
VIA-02	62.2	70.6	81.72
VIA-03_1	39.8	44.8	51.45
VIA-03_2	56.0	63.0	72.33
VIA-04	44.7	51.0	59.46
VIA-05	30.9	34.8	40.02
M02	22.28	24.69	27.87
M03	21.57	23.90	26.98
M04	8.39	9.28	10.44
M09	0.81	0.89	1.01
M10	1.05	1.16	1.31
M11	3.75	4.14	4.66
M12	3.30	3.64	4.10
M13	12.44	13.75	15.48
M14	1.17	1.29	1.45
M15	15.10	16.70	18.80
M16	2.58	2.85	3.21

Sub-Structure	Design Peak Flow (m^3/sec)		
	100-yr	200-yr	500-yr
M17	1.26	1.39	1.57
M18	2.85	3.15	3.54
M19	0.45	0.50	0.56

There is no significant catchments area for M08, M10, M23, M43, M47, M53, and M55 culverts. In addition, M09, M10, M11, M12, M14, and M16-M19 catchments design floods are low compared to the 2×2 size of the culverts, and their hydraulics are not evaluated here.

3. VIADUCT HYDRAULICS AND FLOOD RISK ASSESSMENT

The flood risk is assessed for each bridge (viaduct) and culvert, using the hydraulic model and information from the alignment, plan profile of sub-structure, hydrology, and stream geometry. Water surface profile and mapping are prepared for 100 years of design flood considering State Water Works (DSI) and General Directorate of Highways (KGM) manual and standards.

3.1 Sazlıdere Cable-Stayed Bridge on Sazlıdere Downstream of the Sazlıdere Dam

This viaduct is located downstream of the Sazlidere dam at 1619 meters in length. Its deck elevation is approximately 30 meters height from river level. The Sazlidere river downstream of the dam is protected and designed with a 60 meters width. Therefore, the river flows freely without restriction by the viaduct and its piers. The plan and location of the bridge and the cross-sections on the river are presented in Figure 3-1. The calculated design floods are conducted using hydraulic calculation considering the effect of 30 percent Sazlidere reservoir flood routing. The water surface profile and the viaduct section are presented in Figure 3-2. As shown in the figures, both the bridge and channel capacities are adequate for a 100-year flood passage. It will cause problems around the bridge if the release of the spillway will be more than 506,39 m^3/s (Routed flood with the 500-year return period and the spillway design flood).

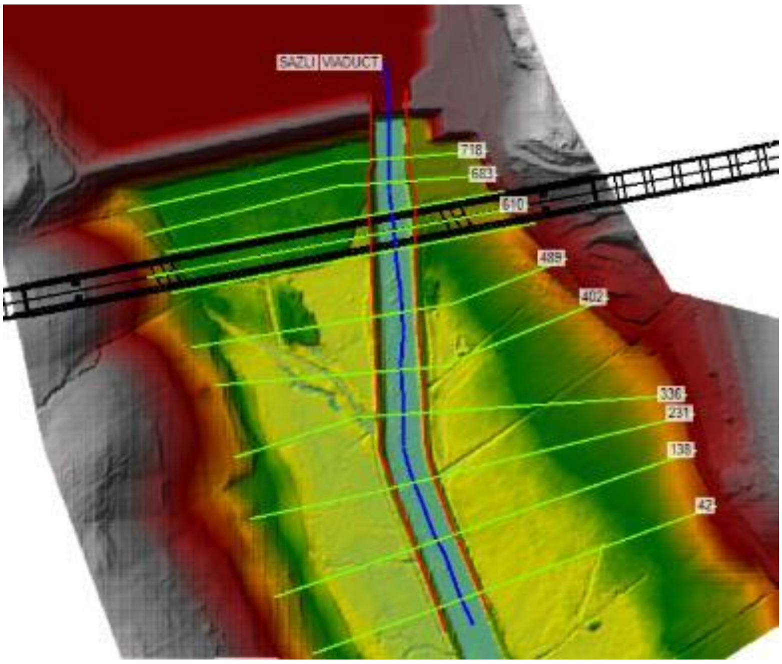
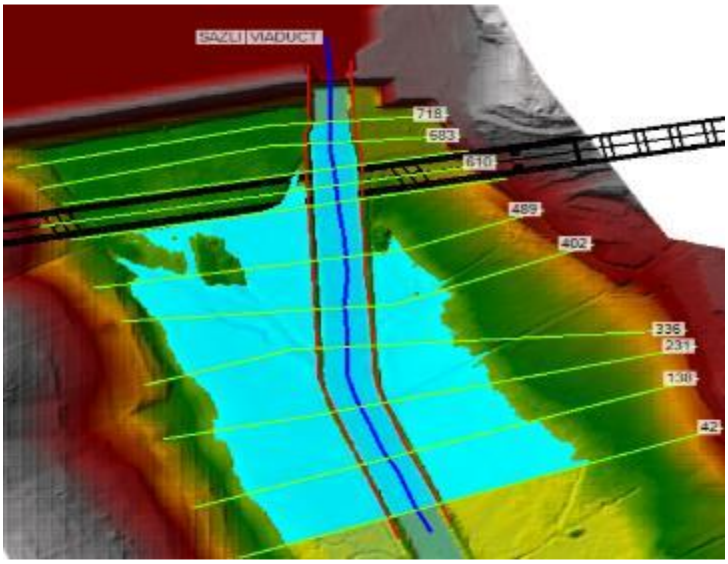
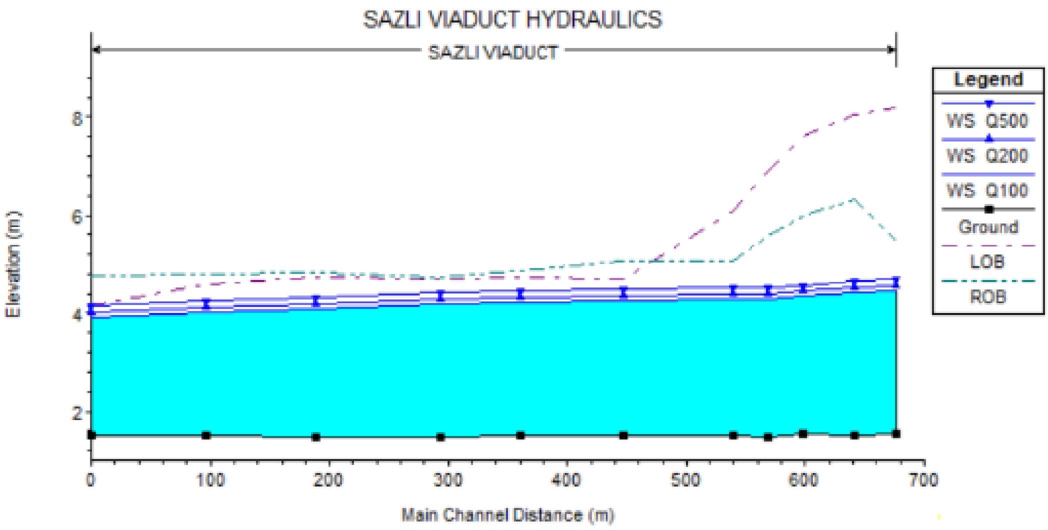


Figure 3-1 Location and Cross-Sections for Hydraulic Modeling of Sazlıdere Cable-Stayed Bridge Downstream of Sazlidere Dam





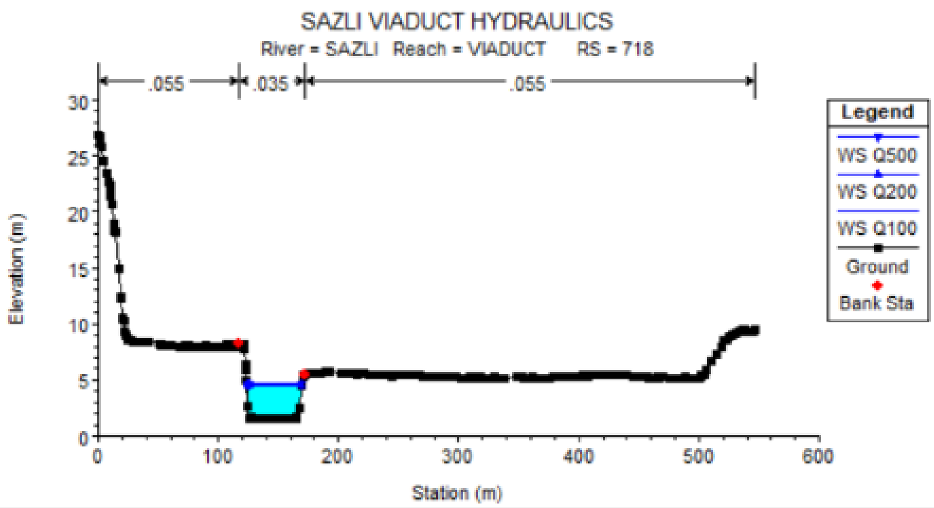
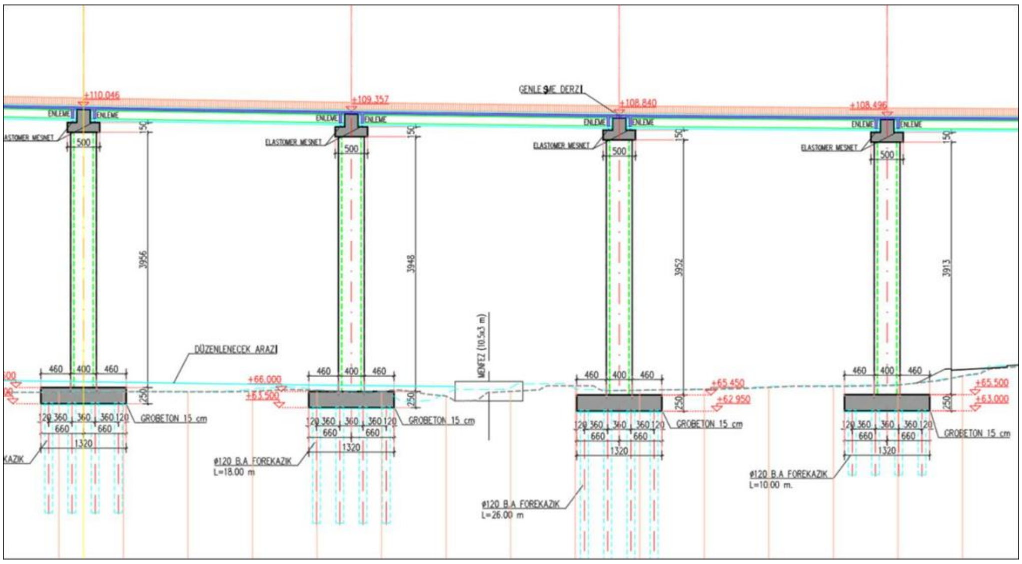
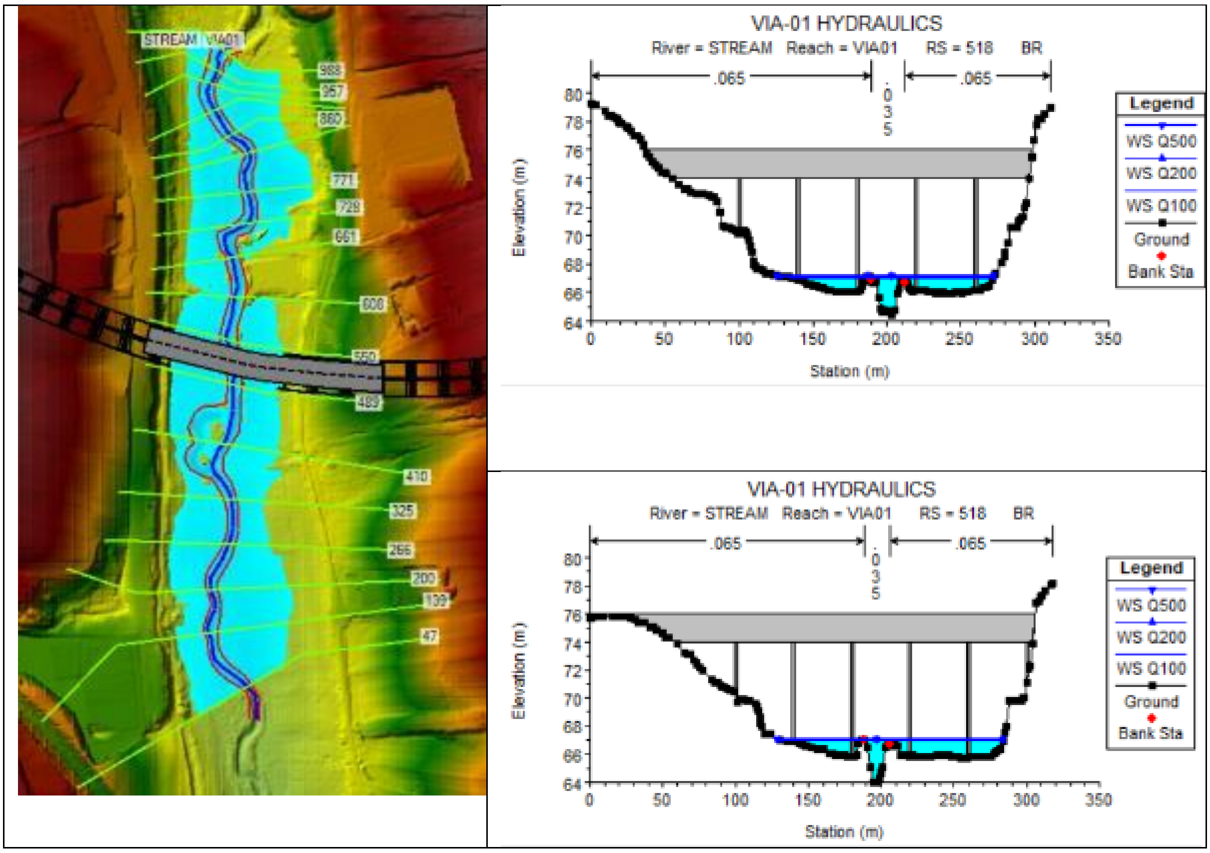


Figure 3-2 100-Years flooding Map and Longitudinal and Cross Section Water Surface for Sazlıdere Cable Stayed Bridge

3.2 Hydraulics and Flood Risk of Viaduct-01

This viaduct is located on Eskinoz dere stream with 968 meters in length. Its deck elevation is approximately 42 meters from stream level, with five spans (40 meters) of the viaduct located in the stream bank. The plan and location of the bridge and the cross-sections on the bridge, including the water surface profile, are presented in Figure 3-3. As shown in the figures, the bridge capacity is adequate for a 100-year flood passage. Still, the channel capacity is not enough for the flood. Therefore, the area around the bridge will be flooded. The upstream and downstream hydraulic properties for the bridge are presented in Table 3-1.





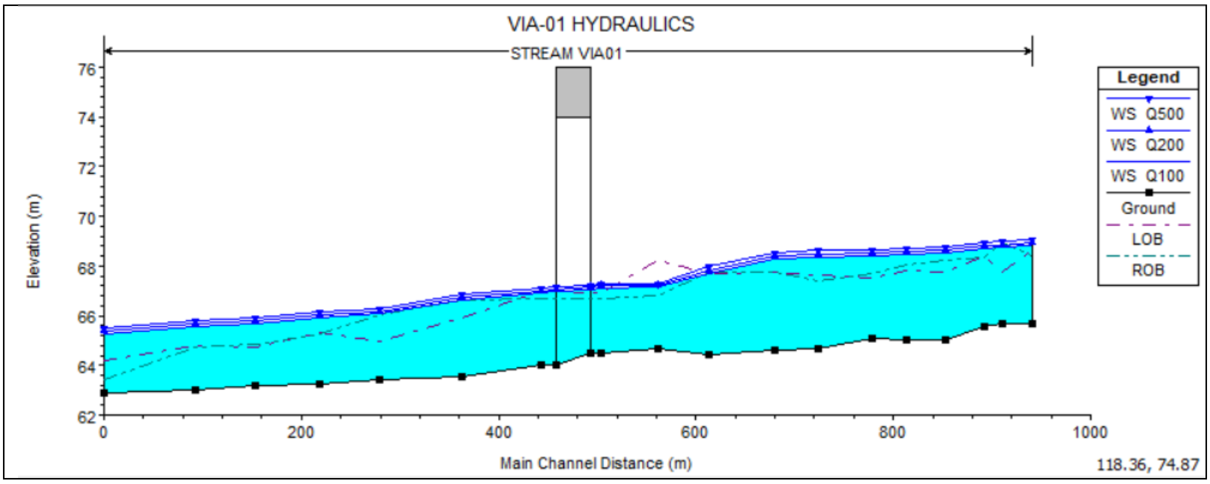


Figure 3-3 Water Surface for Plan, Longitudinal, and Cross-Sections of Viaduct 01

Table 3-1 Result of 100 Years of Flood Hydraulic Properties Upstream and Downstream of Viaduct 01

Reach	River Sta	Profile	E.G. Elev (m)	W.S. Elev (m)	Crit W.S. (m)	Frctn Loss (m)	C & E Loss (m)	Top Width (m)	Q Left (m3/s)	Q Channel (m3/s)	Q Right (m3/s)	Vel Chnl (m/s)
VIA01	608	Q100	67.75	67.13	67.31	0.23	0.09	87.75	2.01	103.72	28.27	3.90
VIA01	550	Q100	67.21	67.11	66.73	0.03	0.00	141.97	23.02	65.03	45.95	1.91
VIA01	518 BR U	Q100	67.18	67.05	66.74	0.11	0.01	126.42	23.74	67.76	42.50	2.07
VIA01	518 BR D	Q100	67.06	66.96	66.63	0.05	0.00	139.40	18.30	56.60	59.10	2.05
VIA01	489	Q100	67.01	66.91		0.25	0.00	144.91	17.55	55.50	60.95	2.07
VIA01	410	Q100	66.76	66.64		0.37	0.02	137.69	20.63	102.28	11.09	1.74

3.3 Hydraulics and Flood Risk of Viaduct-02

This viaduct is located on Hasanoğlu stream with 540 meters in length. Hasanoğlu stream was designed as a trapezoidal section with a bottom width of 10 meters. 9 of 14 spans of the viaduct are in the river valley. The section becomes an enclosed section. The plan and location of the bridge and the cross-sections on the bridge, including the water surface profile, are presented in Figure 3-4. In addition, the water surface profile for longitudinal of the stream is illustrated in Figure 3-5. As shown in the figures, the bridge and channel capacity are adequate for 100- to 500-years flood passage. Therefore, there is no risk of flooding caused by this bridge. The bridge's upstream and downstream hydraulic properties are presented in Table 3-2.

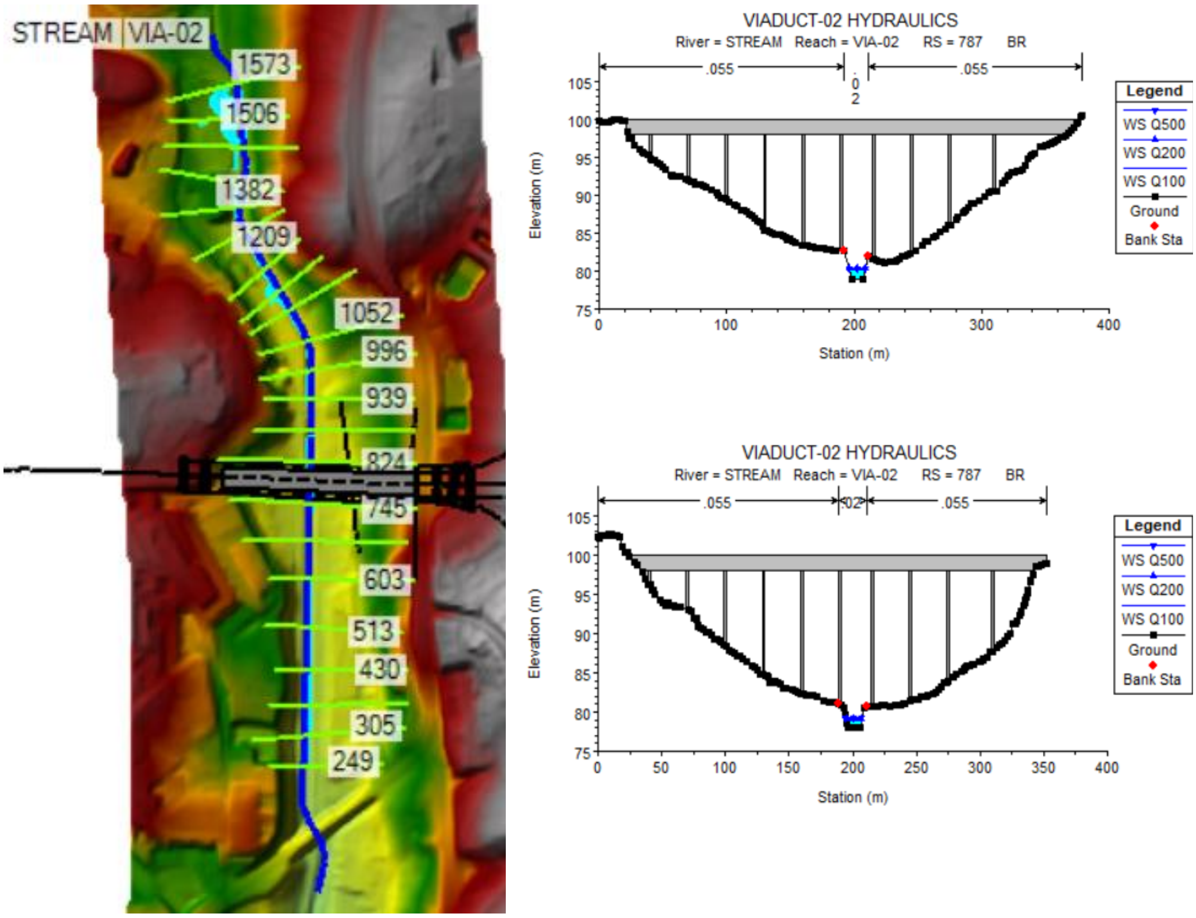


Figure 3-4 Water Surface for Plan and Cross-Sections of Viaduct 02

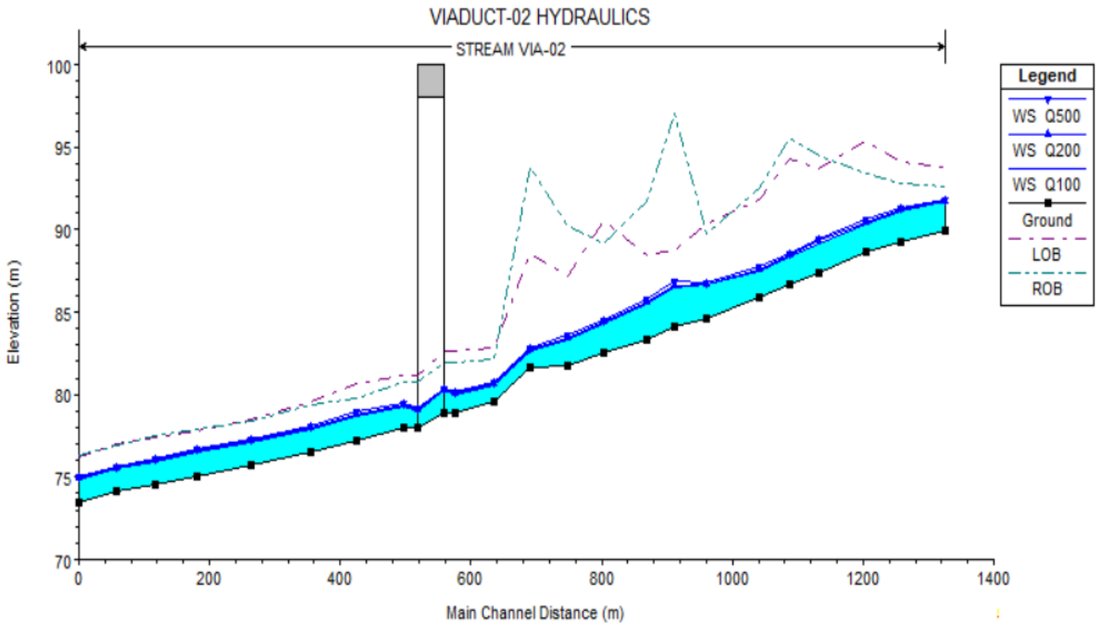


Figure 3-5 Water Surface Profile for Viaduct 02

Table 3-2 Result of 100-, 200- and 500-Years Flood Properties Upstream and Downstream of Viaduct 02

Reach	River Sta	Profile	E.G. Elev (m)	W.S. Elev (m)	Crit W.S. (m)	Frctn Loss (m)	C & E Loss (m)	Top Width (m)	Q Left (m3/s)	Q Channel (m3/s)	Q Right (m3/s)	Vel Chnl (m/s)
VIA-02	884	Q100	82.82	80.54	81.17	1.12	0.24	12.35		62.20		6.69
VIA-02	884	Q200	83.02	80.62	81.29	1.11	0.23	12.53		70.60		6.86
VIA-02	884	Q500	83.27	80.72	81.44	1.09	0.22	12.76		81.72		7.07
VIA-02	824	Q100	81.46	79.97	80.44	0.15	0.17	12.24		62.20		5.40
VIA-02	824	Q200	81.69	80.05	80.56	0.16	0.16	12.47		70.60		5.66
VIA-02	824	Q500	81.97	80.15	80.72	0.17	0.16	12.76		81.72		5.97
VIA-02	787 BR U	Q100	81.14	80.22	80.44	0.40	0.07	12.97		62.20		4.25
VIA-02	787 BR U	Q200	81.36	80.27	80.56	0.42	0.07	13.12		70.60		4.62
VIA-02	787 BR U	Q500	81.63	80.35	80.72	0.44	0.06	13.35		81.72		5.01
VIA-02	787 BR D	Q100	80.67	79.02	79.53	0.22	0.22	12.28		62.20		5.70
VIA-02	787 BR D	Q200	80.87	79.10	79.65	0.23	0.22	12.50		70.60		5.89
VIA-02	787 BR D	Q500	81.12	79.21	79.81	0.23	0.21	12.78		81.72		6.13
VIA-02	745	Q100	80.24	79.30	79.52	0.51	0.01	13.02		62.20		4.28
VIA-02	745	Q200	80.43	79.38	79.64	0.53	0.00	13.22		70.60		4.54
VIA-02	745	Q500	80.68	79.47	79.80	0.55	0.00	13.46		81.72		4.86
VIA-02	673	Q100	79.72	78.72	78.97	0.60	0.02	12.84		62.20		4.41
VIA-02	673	Q200	79.90	78.82	79.10	0.59	0.02	13.09		70.60		4.60
VIA-02	673	Q500	80.13	78.94	79.25	0.59	0.02	13.40		81.72		4.83

3.4 Hydraulics and Flood Risk of Viaduct-03

This viaduct is located before the confluence of Menekşe ve Oyak dere streams with 570 meters in length. The Menekşe streams in this location were designed as a concrete rectangular channel with 8 meters bottom width. However, Nakkaş dere stream had an earthen bed with a top width of about 35 meters. A box culvert 10×3 meters in the stream before receiving to Oyak stream was designed built. After the confluence of the streams, the channel section was changed to 15×2.75 meters. The plan and location of the bridge and culvert are presented in Figure 3-6. The calculated longitudinal profile of Menekşe ve Oyak streams is illustrated in Figure 3-7 and Figure 3-8. As shown in the figures, the capacity of the existing culvert on the Oyak stream is low, and its backwater effects come to the viaduct location on the Oyak tributary. However, the viaduct capacity in both streams is enough for passing the 100-, 200- and 500-years floods. The hydraulic properties for the bridge upstream and downstream of the viaduct and culvert are presented in Table 3-3 and Table 3-4.

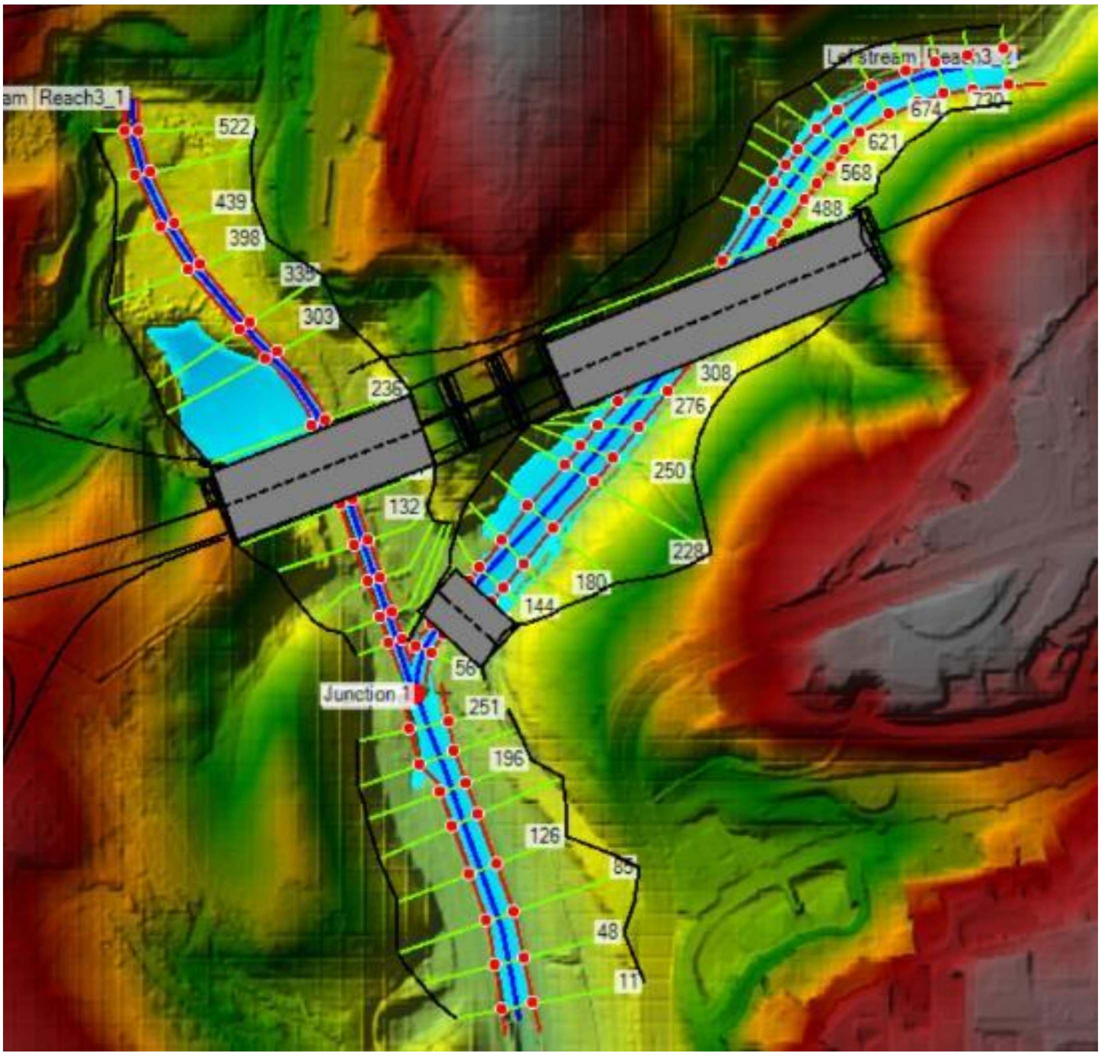


Figure 3-6 Water Surface for Plan, Longitudinal, and Cross-Sections of Viaduct 03

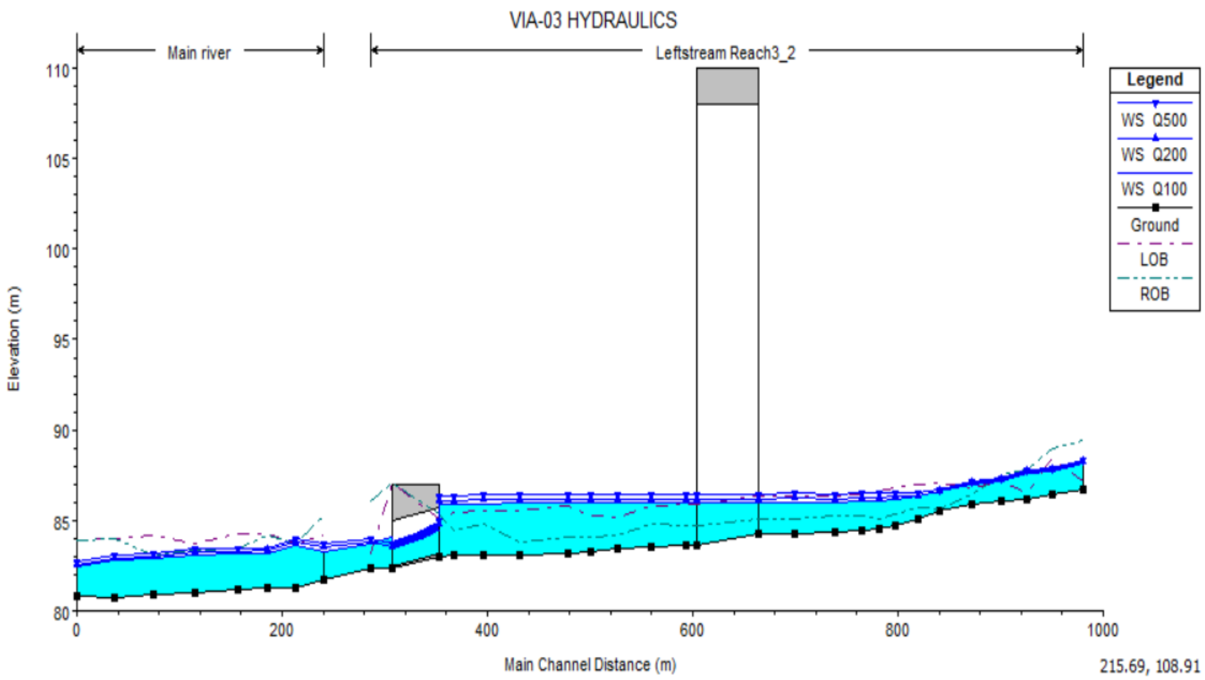


Figure 3-7 Water Surface Profile for Nakkaş Stream around Viaduct 03

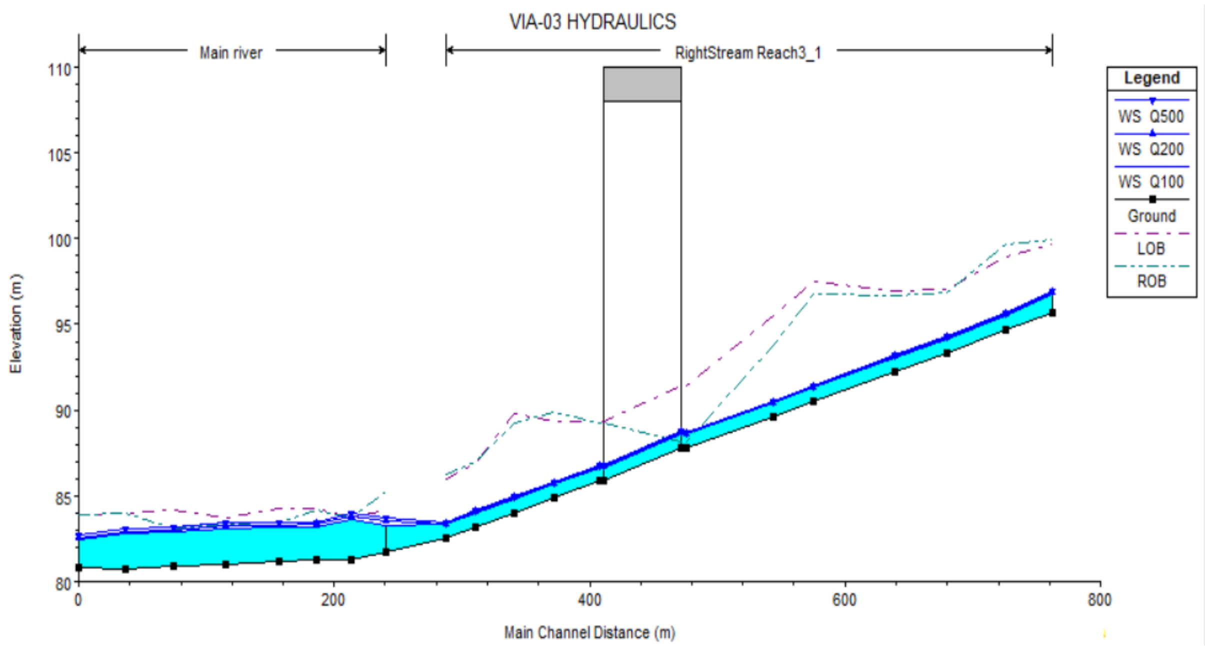


Figure 3-8 Water Surface Profile for Fener Stream around Viaduct 03

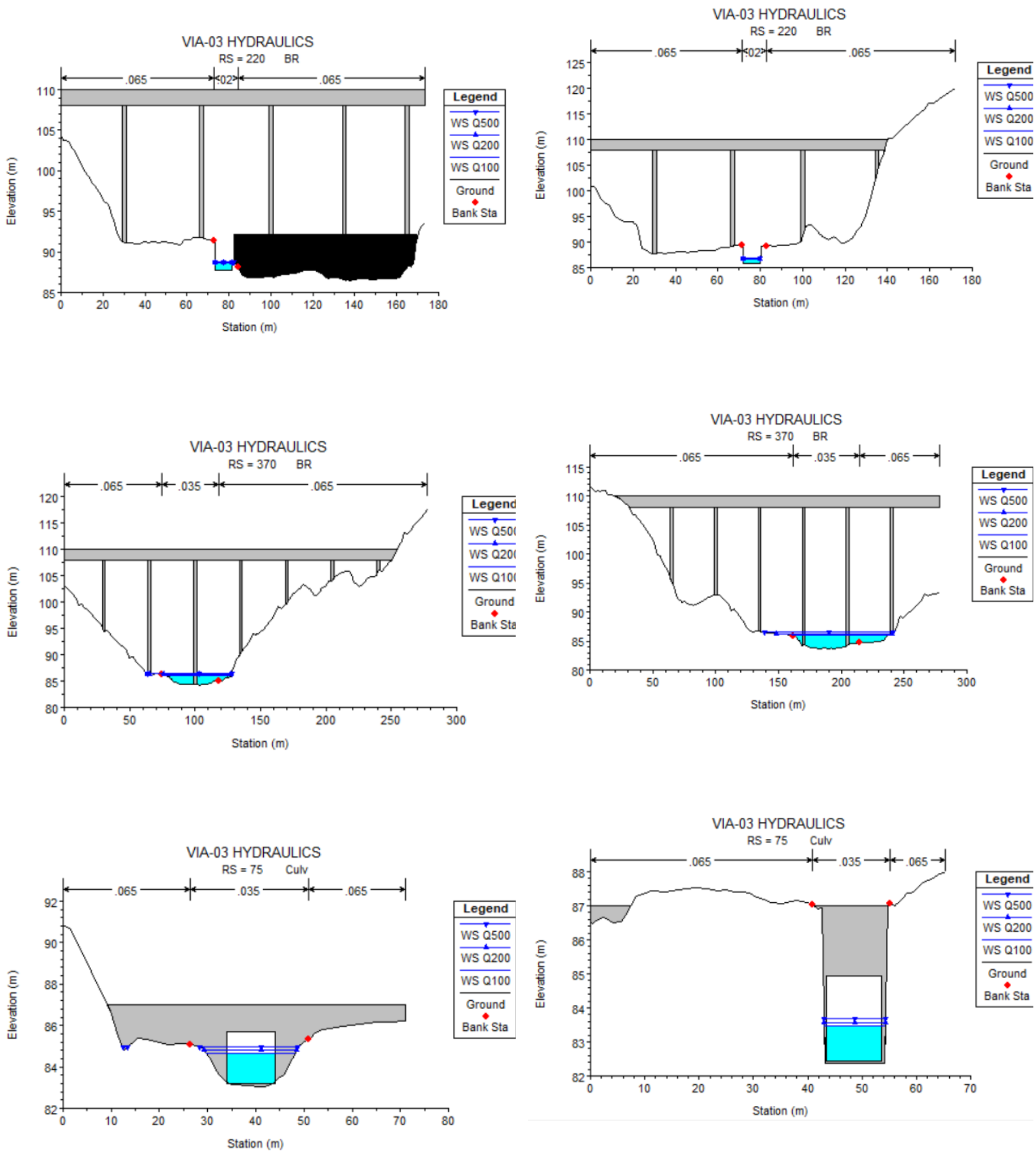


Figure 3-9 Water Surface for Cross-Sections of Viaduct 03 and the Existing Culvert

Table 3-3 Results of 100 Years of Flood Hydraulic Properties Upstream and Downstream of Viaduct 03

River	Reach	River Sta	Profile	E.G. Elev (m)	W.S. Elev (m)	Crit W.S. (m)	Frctn Loss (m)	C & E Loss (m)	Top Width (m)	Q Left (m3/s)	Q Channel (m3/s)	Q Right (m3/s)	Vel Chnl (m/s)
RightStream	Reach3_1	303	Q100	92.42	90.40	90.97	1.86	0.01	8.30		39.80		6.29
RightStream	Reach3_1	236	Q100	90.55	88.57	89.14	0.12	0.07	8.00		39.80		6.23
RightStream	Reach3_1	220 BR U	Q100	90.36	88.62	89.14	1.53	0.04	8.00		39.80		5.83
RightStream	Reach3_1	220 BR D	Q100	88.96	86.61	87.23	0.10	0.06	8.03		39.80		6.79
RightStream	Reach3_1	167	Q100	88.78	86.64	87.23	1.03	0.03	8.03		39.80		6.48
RightStream	Reach3_1	132	Q100	87.73	85.69	86.26	0.89	0.00	8.00		39.80		6.33
Leftstream	Reach3_2	488	Q100	86.15	86.00		0.05	0.03	30.72		52.84	3.16	1.80
Leftstream	Reach3_2	449	Q100	86.07	86.02	85.13	0.03	0.00	49.52	0.00	54.34	1.66	0.98
Leftstream	Reach3_2	370 BR U	Q100	86.04	85.98	85.17	0.02	0.01	46.67		54.24	1.76	1.07
Leftstream	Reach3_2	370 BR D	Q100	86.00	85.98	84.51	0.00	0.00	73.73	0.00	50.60	5.40	0.58
Leftstream	Reach3_2	342	Q100	86.00	85.98		0.01	0.00	77.70	0.00	51.31	4.69	0.54
Leftstream	Reach3_2	308	Q100	85.99	85.97		0.01	0.00	62.61	0.02	50.98	5.00	0.64

Table 3-4 Results of 100 Years of Flood Hydraulic Properties Upstream and Downstream of Viaduct 03

River	Reach	River Sta	Profile	E.G. Elev (m)	W.S. Elev (m)	Vel Head (m)	Frctn Loss (m)	C & E Loss (m)	Q Left (m3/s)	Q Channel (m3/s)	Q Right (m3/s)	Top Width (m)
Leftstream	Reach3_2	116	Q100	85.93	85.88	0.05	0.01	0.00	0.88	54.69	0.44	44.40
Leftstream	Reach3_2	102	Q100	85.92	85.87	0.05			2.98	52.93	0.10	45.04
Leftstream	Reach3_2	75		Culvert								
Leftstream	Reach3_2	56	Q100	84.54	84.11	0.42	0.19	0.02		56.00		11.37
Leftstream	Reach3_2	36	Q100	84.33	83.72	0.61	0.39	0.00	0.66	55.34		14.25

3.5 Hydraulics and Flood Risk of Viaduct-04

This viaduct is located on Karanlık stream with 525 meters in length. The plan and location of the bridge and the cross-sections on the bridge, including the water surface profile, are presented in Figure 3-3. As shown in the figures, the bridge capacity is enough for a 100-years flood passage. Still, the channel capacity is not enough for the flood. Therefore, the area around the bridge will be flooded. The upstream and downstream hydraulic properties for the bridge are presented in Figure 3-10.

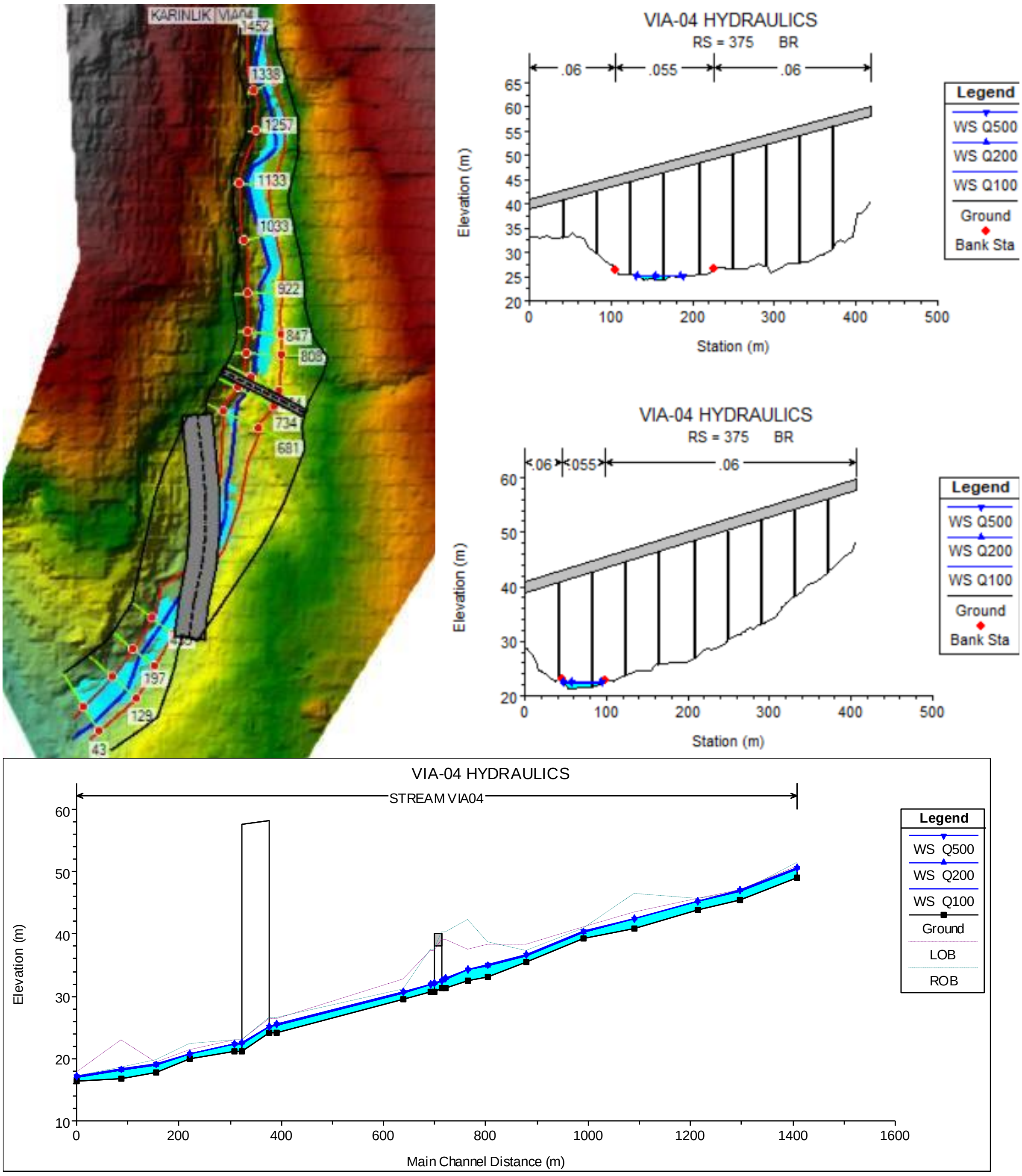


Figure 3-10 Water Surface for Plan, Longitudinal, and Cross-Sections of Viaduct 01

Table 3-5 Results of 100 Years of Flood Hydraulic Properties Upstream and Downstream of Viaduct 01

River	Reach	River Sta	Profile	E.G. Elev	W.S. Elev	Crit W.S.	Frctn Loss	C & E Loss	Top Width	Q Left	Q Channel	Q Right	Vel Chnl	Hydr Depth
				(m)	(m)	(m)	(m)	(m)	(m)	(m3/s)	(m3/s)	(m3/s)	(m/s)	(m)
STREAM	VIA04	375 BR U	Q100	25.27	25.06	25.06	0.72	0.04	49.68		44.70		2.07	0.44
STREAM	VIA04	375 BR U	Q200	25.33	25.10	25.10	0.73	0.04	51.49		51.00		2.13	0.46
STREAM	VIA04	375 BR U	Q500	25.41	25.16	25.16	0.76	0.04	55.16		59.46		2.20	0.49
STREAM	VIA04	375 BR D	Q100	22.47	22.39	22.00	2.96	0.02	43.34		44.70		1.24	0.83
STREAM	VIA04	375 BR D	Q200	22.54	22.45	22.05	2.94	0.02	44.08		51.00		1.32	0.88
STREAM	VIA04	375 BR D	Q500	22.64	22.54	22.11	2.91	0.02	45.50		59.46		1.40	0.93

3.6 Hydraulics and Flood Risk of Viaduct-05

This viaduct is located on one of the watercourses of Ayamama stream with 470 meters in length. The watercourse passes here through the Sular Vadisi social collection and park. The plan and location of the viaduct and its upstream and downstream cross-sections, including the water surface profile presented in Figure 3-3. As shown in the figure, the bridge capacity is enough for a 100-years flood passage. Still, the social and park area will be inundated about 0.6 meters depth around the viaduct by a 100-years flood. The upstream and downstream hydraulic properties for the bridge are presented in Table 3-6.

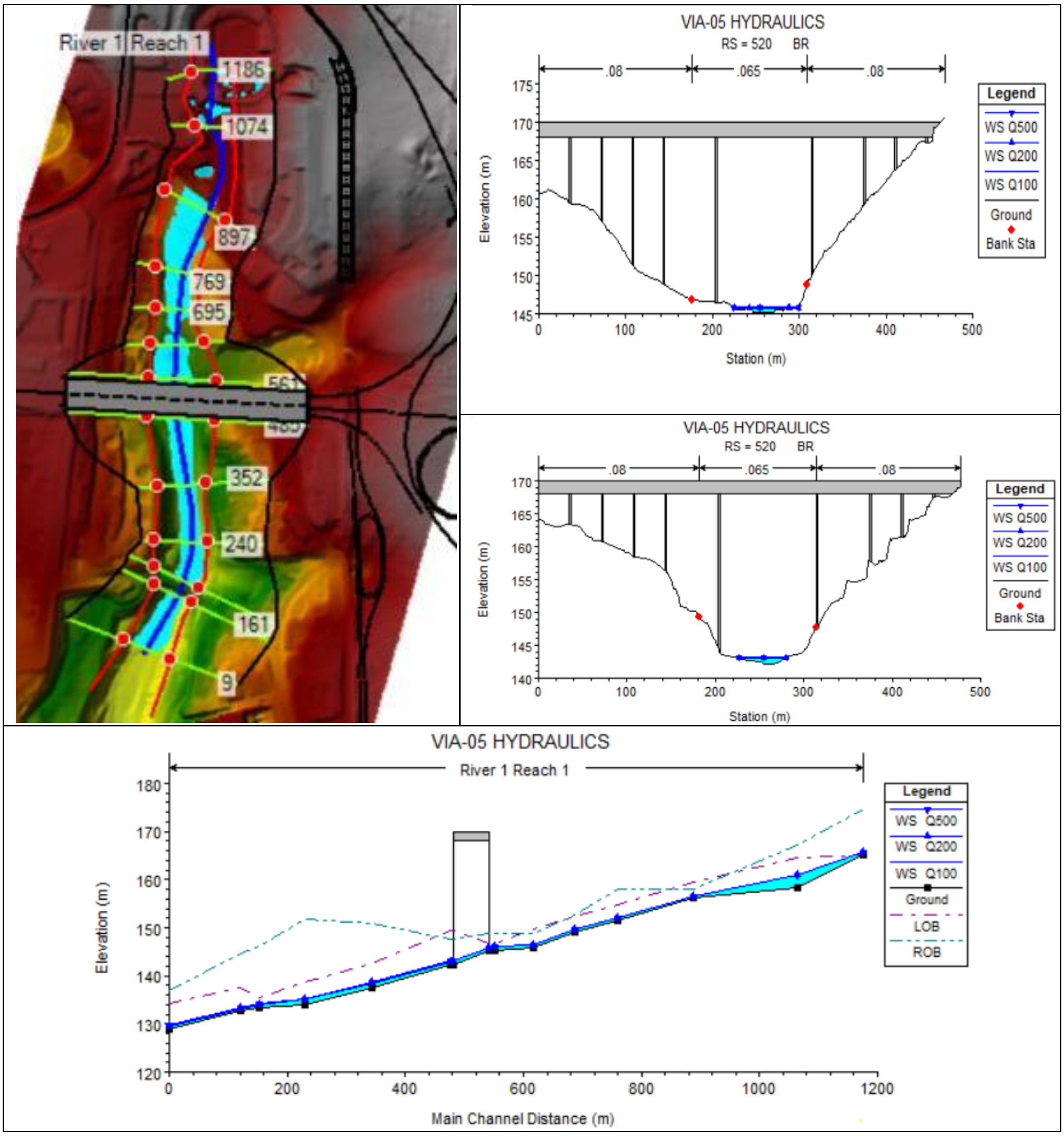


Figure 3-11 Water Surface for Plan, Longitudinal, and Cross-Sections of Viaduct 05

Table 3-6 Results of 100 Years of Flood Hydraulic Properties Upstream and Downstream of Viaduct 05

Reach	River Sta	Profile	E.G. Elev	W.S. Elev	Crit W.S.	Frctn Loss	C & E Loss	Top Width	Q Left	Q Channel	Q Right	Vel Chnl	Hydr Depth
			(m)	(m)	(m)	(m)	(m)	(m)	(m3/s)	(m3/s)	(m3/s)	(m/s)	(m)
Reach 1	626	Q100	146.50	146.46	146.12	0.45	0.00	50.69		30.90		0.96	0.64
Reach 1	626	Q200	146.55	146.50	146.15	0.46	0.00	51.69		34.80		1.01	0.67
Reach 1	626	Q500	146.62	146.56	146.19	0.48	0.01	52.87		40.02		1.07	0.71
Reach 1	561	Q100	146.05	146.02	145.75	0.15	0.01	77.19		30.90		0.79	0.50
Reach 1	561	Q200	146.09	146.05	145.78	0.16	0.01	77.46		34.80		0.84	0.54
Reach 1	561	Q500	146.13	146.09	145.81	0.16	0.01	77.78		40.02		0.90	0.57
Reach 1	520 BR U	Q100	145.89	145.75	145.75	1.23	0.02	68.10		30.90		1.64	0.28
Reach 1	520 BR U	Q200	145.92	145.78	145.78	1.25	0.02	73.26		34.80		1.67	0.28
Reach 1	520 BR U	Q500	145.96	145.81	145.81	1.26	0.03	75.11		40.02		1.73	0.31
Reach 1	520 BR D	Q100	143.12	143.06	142.80	3.66	0.00	52.91		30.90		1.04	0.56
Reach 1	520 BR D	Q200	143.16	143.10	142.83	3.61	0.01	53.58		34.80		1.10	0.59
Reach 1	520 BR D	Q500	143.22	143.15	142.87			54.34		40.02		1.16	0.63
Reach 1	485	Q100	143.03	142.94		4.28	0.01	50.62		30.90		1.32	0.46
Reach 1	485	Q200	143.07	142.98		4.27	0.01	51.28		34.80		1.38	0.49
Reach 1	485	Q500	143.13	143.02		4.27	0.01	52.13		40.02		1.46	0.53
Reach 1	352	Q100	138.74	138.54	138.54	2.84	0.03	40.38		30.90		1.96	0.39
Reach 1	352	Q200	138.79	138.58	138.58	2.81	0.04	40.91		34.80		2.03	0.42
Reach 1	352	Q500	138.85	138.62	138.62	2.80	0.04	41.57		40.02		2.12	0.45

4. CULVERTS FLOOD RISK ASSESSMENT

The list of the culverts and their size throughout the project is presented in the first chapter. As mentioned in the previous chapter, there is no significant catchment area for M08, M10, M23, M43, M47, M53, and M55 culverts. In addition, the design floods of M09, M10, M11, M12, M14, and M16-M19 catchments are low compared to the slope and size of culverts (2×2 meters). Therefore, in this chapter, the hydraulics and floods risks are assessed for the list in Table 4-1.

Table 4-1 The Selected Culverts for Flood Risk Assessment

Culvert Name	KM	Dimension		Culvert Control Levels (masl)		Design Peak Flow (m^3/sec)		
		Width (m)	Height (m)	Inlet	Outlet	100-yr	200-yr	500-yr
M02	38+032	4.0	2.5	80.67	72.24	22.28	24.69	27.87
M03	38+480	5.0	2.5	71.04	66.79	21.57	23.90	26.98
M04	38+895	4.0	2.5	93.77	87.06	8.39	9.28	10.44
M13	53+296	3.0	2.5	88.28	81.69	12.44	13.75	15.48
M15	54+148	4.0	2.0	97.02	93.91	15.10	16.70	18.80

Each culvert geometry is constructed into the HEC-RAS model, and then, by considering inlet and outlet invert levels, the water head upstream and downstream is calculated. Based on the flood maps and the potential hazard and risks, the level of risk is evaluated.

4.1 Hydraulics and Flood Risk Assessment for Culvert M02

This culvert is located in 38+032 KM with a 4.0 ×2.5 meters cross-section. The 100 years flood for its catchment is calculated at 22.28 m^3/sec. The inlet and outlet elevations are 80.666 and 72.236 meters above sea level. The constructed plan and flood map the upstream and downstream of the culvert are presented in Figure 4-1. In addition, longitudinal and cross-sections of the culvert with water surface profile are presented in Figure 4-2 . The hydraulic properties for the upstream and downstream of the culvert are also presented in Table 4-2. As shown in the table, the culvert capacity is enough for a 100-years flood passage. Still, the inlet control of water height receives 3.02 meters, and the generated backwater will flood the area around the culvert. For this culvert, the stream has to be designed for 100 years of flood capacity in the future.

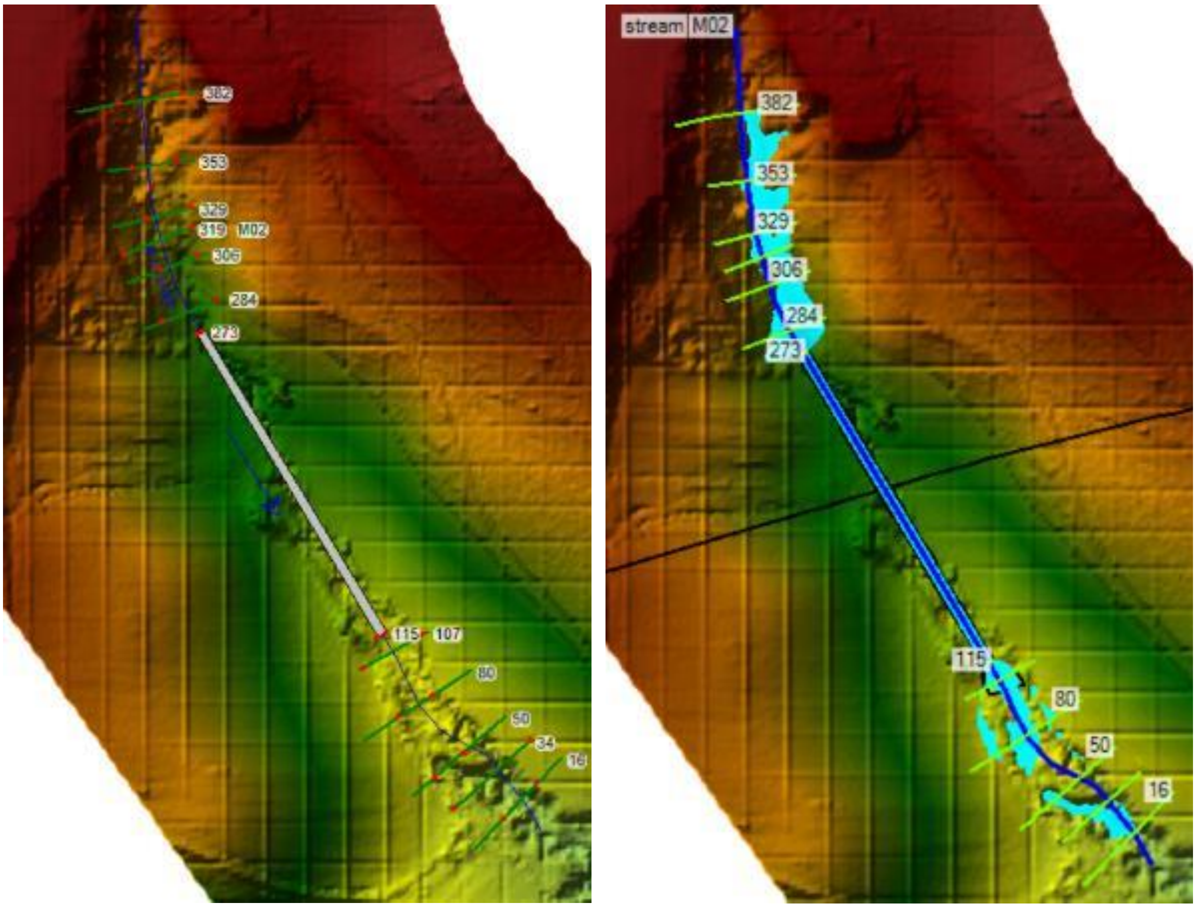


Figure 4-1 Constructed Model of M02 Culvert and Flood Map Plan for Q100

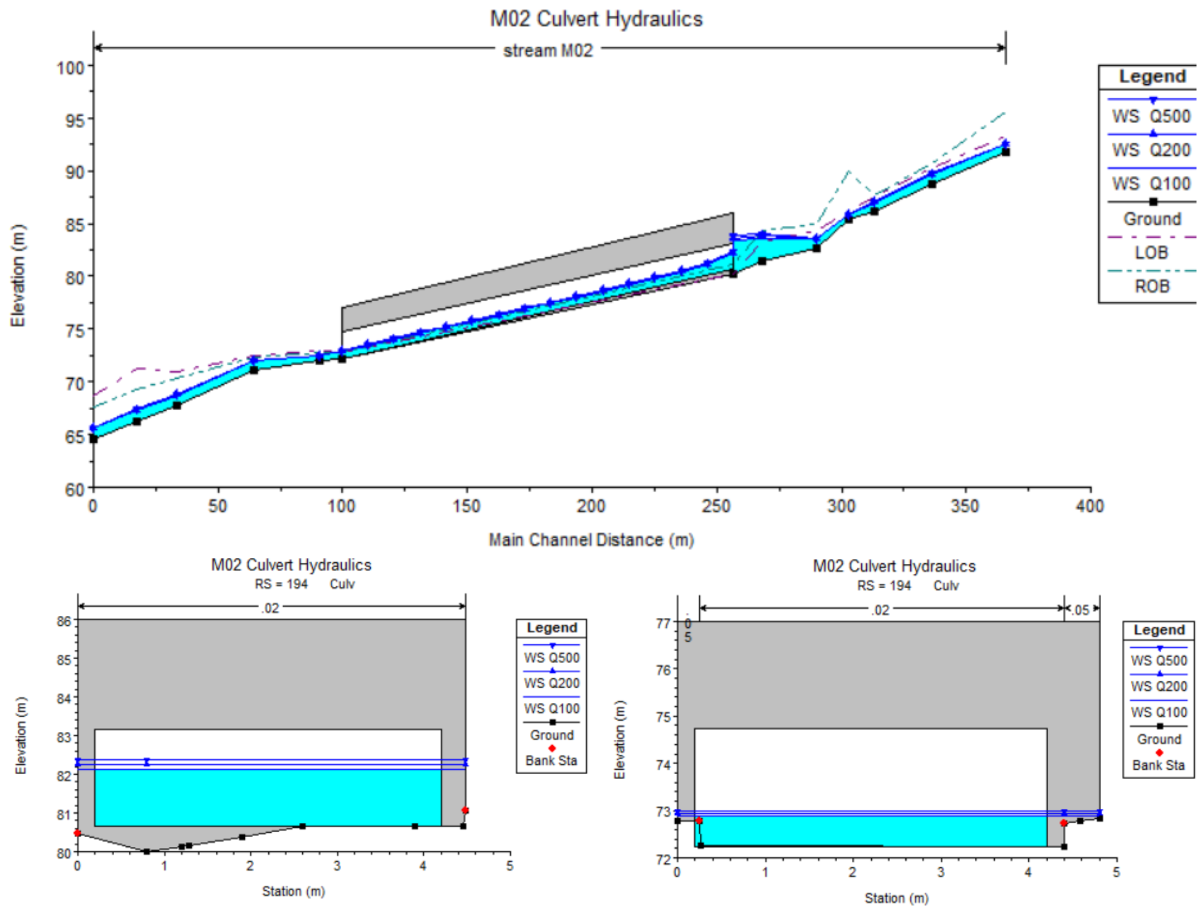


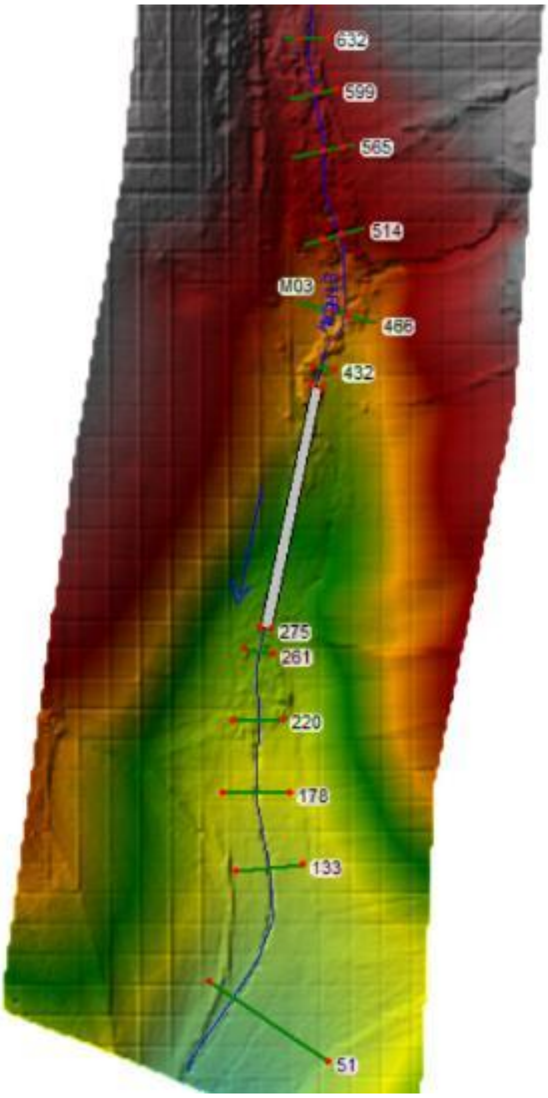
Figure 4-2 Culvert M02 Longitudinal and Upstream and Downstream Cross-Section Water Surface Profile

Table 4-2 Results of Hydraulics for Upstream and Downstream of M02

Reach	River Sta	Profile	E.G. Elev	W.S. Elev	Vel Head	Frctn Loss	C & E Loss	Q Left	Q Channel	Q Right	Top Width	Hydr Depth
			(m)	(m)	(m)	(m)	(m)	(m3/s)	(m3/s)	(m3/s)	(m)	(m)
M02	284	Q100	83.62	83.59	0.04	0.01	0.01	0.02	22.26		23.78	1.13
M02	284	Q200	83.83	83.80	0.03	0.01	0.01	0.07	24.62		25.36	1.27
M02	284	Q500	84.09	84.07	0.03	0.01	0.01	0.18	27.69		27.40	1.43
M02	273	Q100	83.60	83.46	0.14				22.28		4.48	3.02
M02	273	Q200	83.81	83.66	0.15				24.69		4.48	3.21
M02	273	Q500	84.07	83.91	0.16				27.87		4.48	3.46
M02	194		Culvert									
M02	115	Q100	76.36	72.89	3.47	1.22	0.33	0.03	22.21	0.04	4.80	0.58
M02	115	Q200	76.63	72.94	3.69	1.22	0.31	0.05	24.57	0.08	4.80	0.62
M02	115	Q500	76.96	73.00	3.96	1.21	0.28	0.07	27.68	0.12	4.80	0.68
M02	107	Q100	74.82	72.44	2.38	1.84	0.58		22.28		15.74	0.21
M02	107	Q200	75.11	72.45	2.66	1.99	0.65		24.69		16.07	0.21
M02	107	Q500	75.47	72.46	3.01	2.17	0.73		27.87		16.53	0.22

4.2 Hydraulics and Flood Risk Assessment for Culvert M03

This culvert is located in 38+480 KM with 5.0 ×2.5 meters. The 100 years flood for its catchment is calculated at 21.57 m³/sec. The inlet and outlet elevations are 71.04 and 66.79 meters above sea level. The constructed plan and flood map the upstream and downstream of the culvert are presented in Figure 4-3. In addition, longitudinal and cross-sections of the culvert with water surface profile are presented in Figure 4-4. The hydraulic properties for the culvert upstream and downstream are also presented in Table 4-3. As shown in the table, the culvert capacity is enough for a 100-years flood passage. Still, the inlet control of water height is 2.66 meters, and the generated backwater will flood the area around the culvert. For this culvert, the stream has to be changed for 100 years of flood capacity in the future.



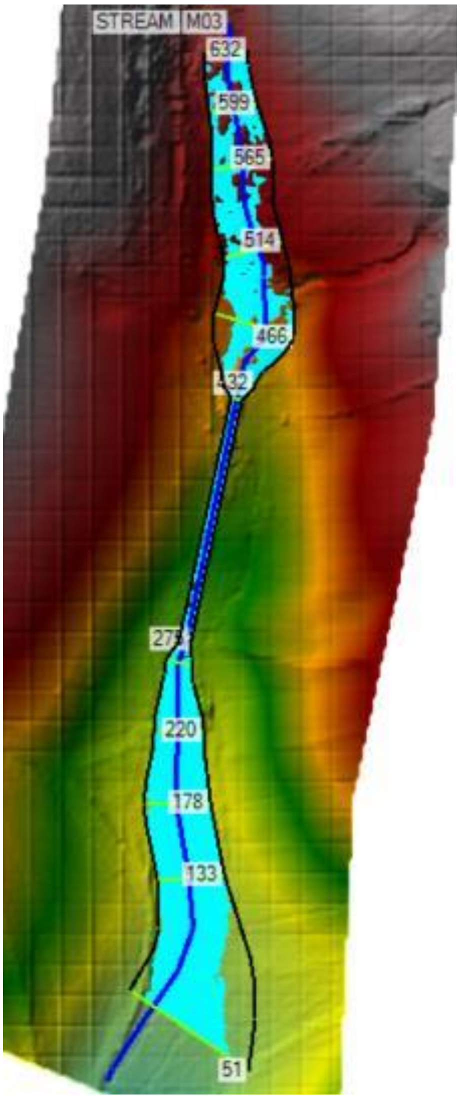


Figure 4-3 Constructed Model of M03 Culvert and Flood Map Plan for Q100

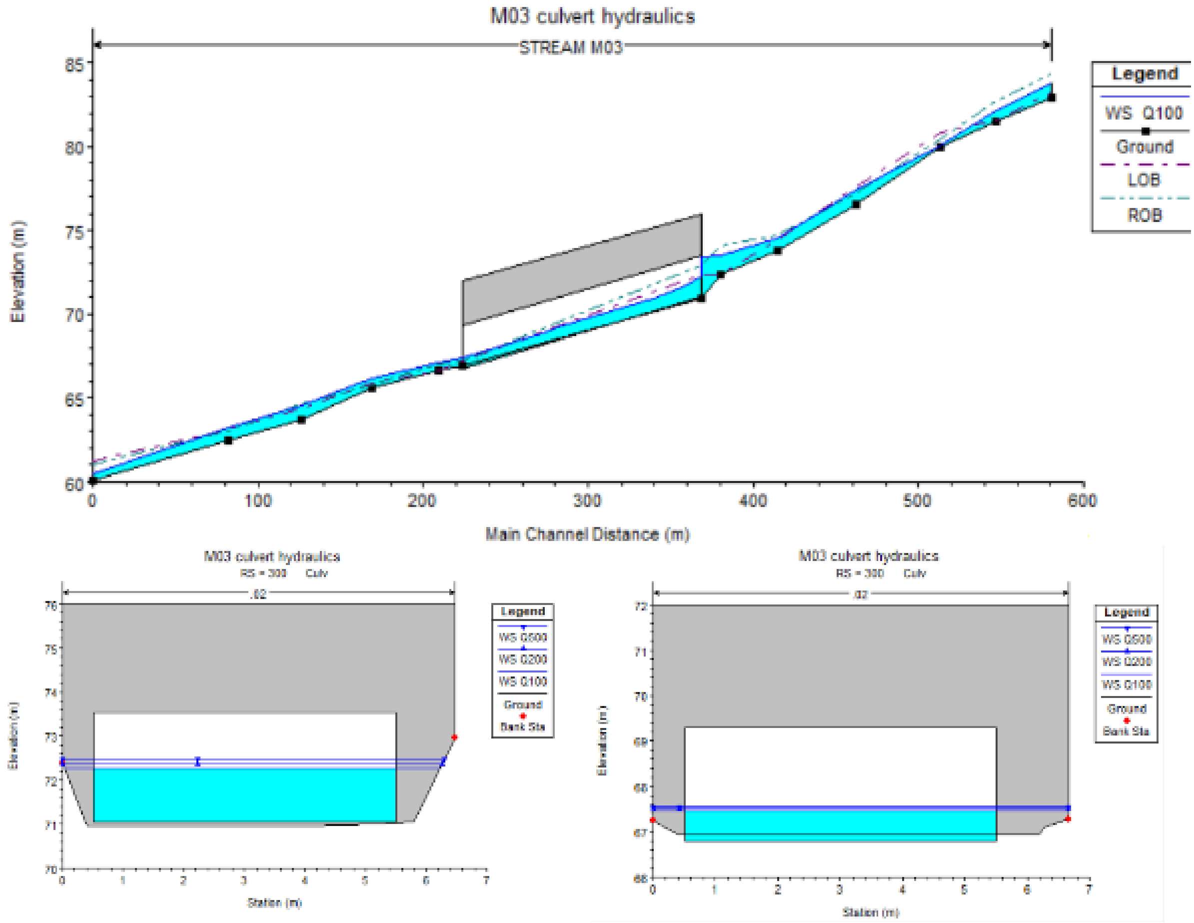


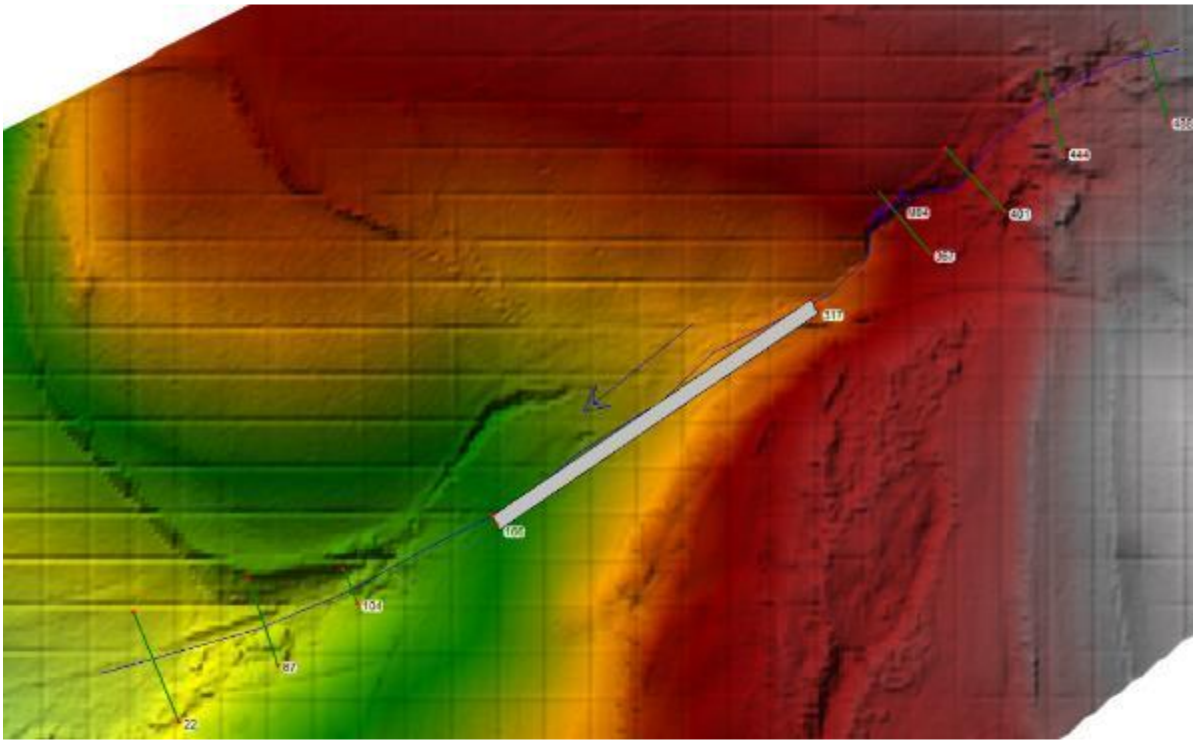
Figure 4-4 M03 Longitudinal and Upstream and Downstream Cross-Section Water Surface Profile

Table 4-3 Results of 100 Years of Flood Hydraulic Properties Upstream and Downstream of M03

Reach	River Sta	Profile	E.G. Elev (m)	W.S. Elev (m)	Vel Head (m)	Frctn Loss (m)	C & E Loss (m)	Q Left (m3/s)	Q Channel (m3/s)	Q Right (m3/s)	Top Width (m)	Hydr Depth (m)
M03	432	Q100	73.97	73.54	0.43	0.02	0.10		21.57		8.73	0.85
M03	432	Q200	74.05	73.67	0.38	0.02	0.08		23.85	0.05	12.24	0.72
M03	432	Q500	74.15	73.75	0.40	0.02	0.08		26.83	0.15	12.91	0.76
M03	420	Q100	73.52	73.41	0.11				21.57		6.48	2.28
M03	420	Q200	73.69	73.58	0.12				23.90		6.48	2.45
M03	420	Q500	73.91	73.79	0.12				26.98		6.48	2.66
M03	300		Culvert									
M03	275	Q100	69.67	67.45	2.22	1.23	0.41		21.57		6.68	0.49
M03	275	Q200	69.77	67.50	2.28	1.22	0.39		23.90		6.68	0.54
M03	275	Q500	69.91	67.55	2.36	1.19	0.36		26.98		6.68	0.59
M03	261	Q100	68.03	67.17	0.86	1.13	0.03		21.57		16.10	0.33
M03	261	Q200	68.18	67.18	1.00	1.10	0.03		23.90		16.10	0.34
M03	261	Q500	68.36	67.19	1.17	1.06	0.03		26.98		16.10	0.35

4.3 Hydraulics and Flood Risk Assessment for Culvert M04

This culvert is located in KM 38+895 with 4.0 ×2.5 meters. The 100 years flood for its catchment is calculated at 8.39 m³/sec. The inlet and outlet elevations are 93.77 and 87.06 meters above sea level. The constructed plan and flood map, and water surface profile for longitudinal and cross-sections of the culvert are presented in Figure 4-5. The hydraulic properties for the upstream and downstream of the culvert are also presented in Table 4-4. As shown in the table, the culvert capacity is enough for 100- to 500-years flood passage, and there is no flood inundation risk around the culvert and its upstream.



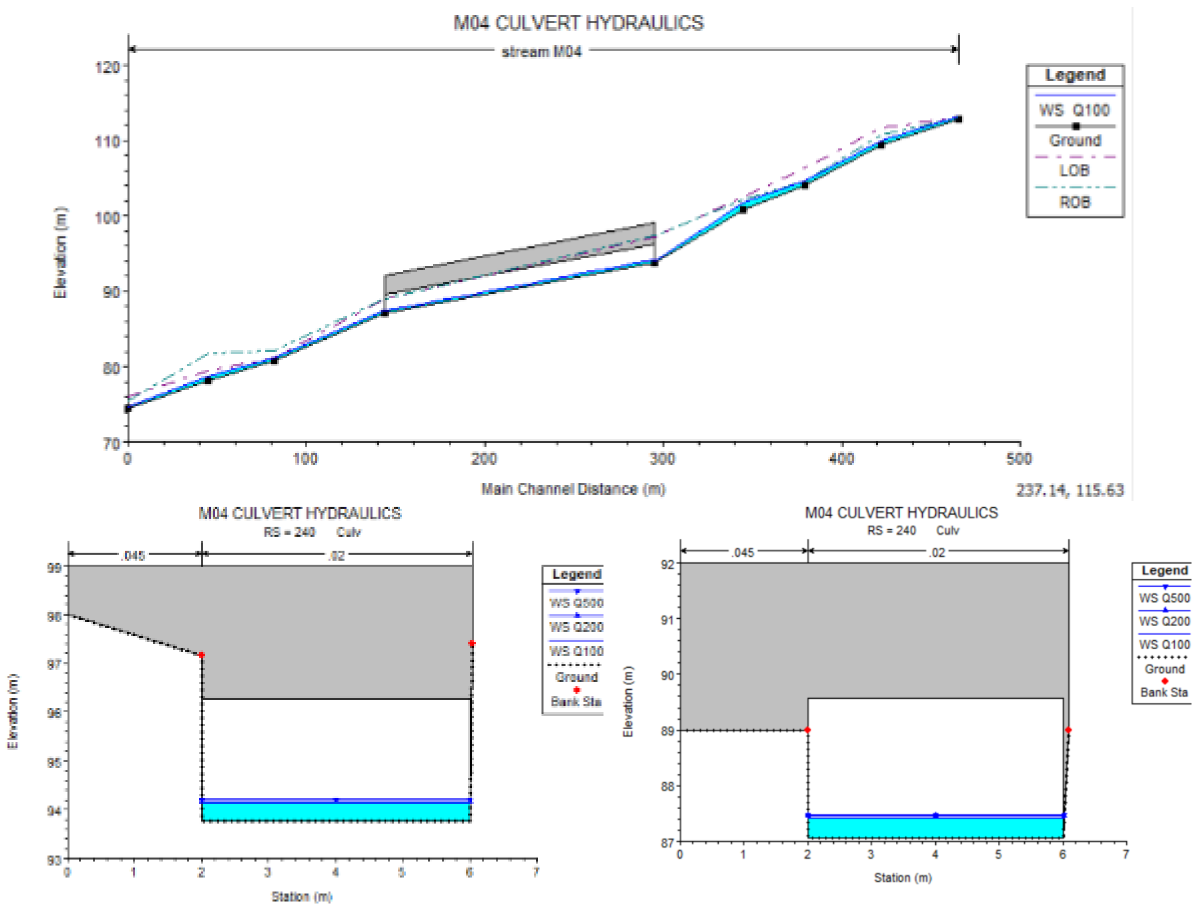


Figure 4-5 Water surface for the Plan, Longitudinal, and Cross-Sections of M04

Table 4-4 Results of 100 Years of Flood Hydraulic Properties Upstream and Downstream of M04

Reach	River Sta	Profile	E.G. Elev	W.S. Elev	Vel Head	Frctn Loss	C & E Loss	Q Left	Q Channel	Q Right	Top Width	Hydr Depth
			(m)	(m)	(m)	(m)	(m)	(m3/s)	(m3/s)	(m3/s)	(m)	(m)
M04	367	Q100	101.96	101.71	0.25	4.30	0.31		8.39		12.28	0.31
M04	367	Q200	102.00	101.73	0.26	4.18	0.32		9.28		12.63	0.32
M04	367	Q500	102.04	101.76	0.28	4.05	0.33		10.44		13.06	0.34
M04	317	Q100	97.36	94.03	3.33				8.39		4.00	0.26
M04	317	Q200	97.50	94.05	3.45				9.28		4.00	0.28
M04	317	Q500	97.66	94.08	3.58				10.44		4.00	0.31
M04	240		Culvert									
M04	166	Q100	89.06	87.43	1.63	6.75	0.24		8.39		4.02	0.37
M04	166	Q200	89.21	87.45	1.75	6.80	0.25		9.28		4.02	0.39
M04	166	Q500	89.41	87.48	1.92	6.88	0.28		10.44		4.02	0.42
M04	104	Q100	82.07	81.24	0.83	2.93	0.19		8.39		9.36	0.22
M04	104	Q200	82.16	81.25	0.90	2.96	0.21		9.28		9.52	0.23
M04	104	Q500	82.25	81.27	0.98	3.00	0.23		10.44		9.73	0.24

4.4 Hydraulics and Flood Risk Assessment for Culvert M13 and M15

These culverts are located in KM 53+296 and KM 54+148 with a size of 3.0 ×2.5 and 4.0 ×2.0 meters on the two main watercourses of Kayabaşı stream. The constructed geometry of these watercourses and the culverts are presented in Figure 4-5. The calculated flood map for both of these culverts is presented in Figure 4-7. Based on the results for each culvert the risk assessment is evaluated as following.

4.4.1 Culvert M13

This culvert's inlet and outlet elevations are 88.28 and 81.69 meters above sea level. Its longitudinal and cross-section water surface profile is presented in Figure 4-8. The hydraulic properties for the culvert upstream and downstream are also presented in Table 4-5. As shown in the table, the culvert capacity is enough for 100- to 500-years flood passage, and there is no flood inundation risk around the culvert and its upstream.

4.4.2 Culvert M15

This culvert's inlet and outlet elevations are 97.02 and 93.91 meters above sea level. Its longitudinal and cross-section water surface profile is presented in Figure 4-9. The hydraulic properties for the culvert upstream and downstream are also presented in Table 4-6. As shown in the table, the culvert capacity is enough for 100- to 500-years flood passage. Still, the urban area upstream and downstream of the culverts is inundated because of inconvenient the existing structures. Then a design implementation is required upstream and downstream of this culvert to mitigate the potential risk of the flood around this culvert.

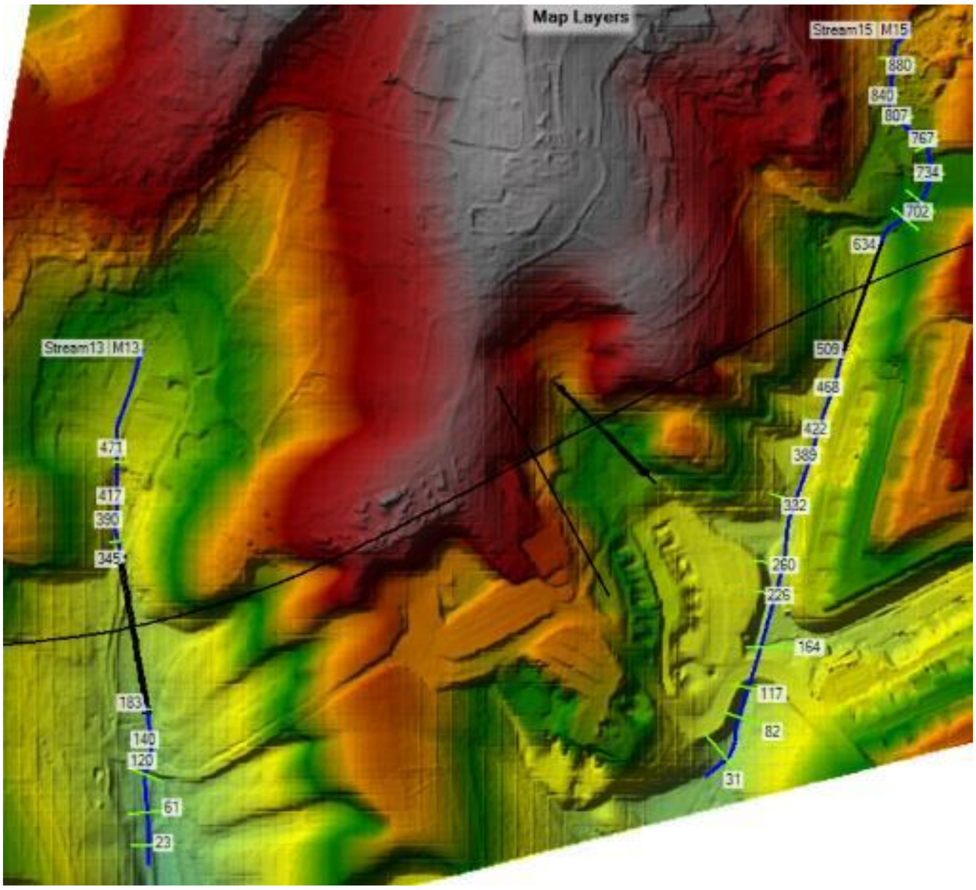


Figure 4-6 Plan of Constructed Model for M13 and M15 Culverts

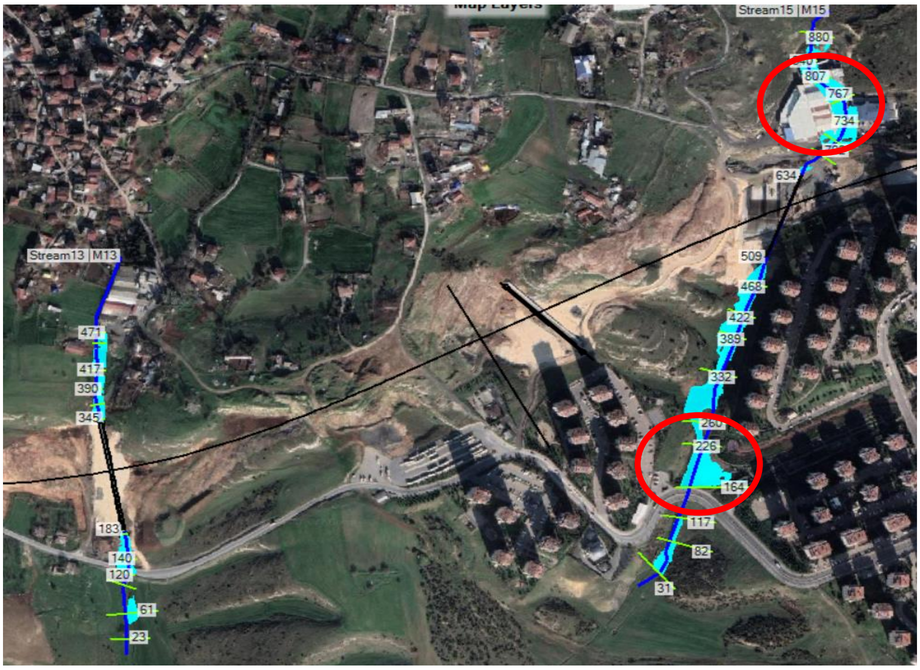


Figure 4-7 Flood Mapping Results for M13 and M15 Culverts

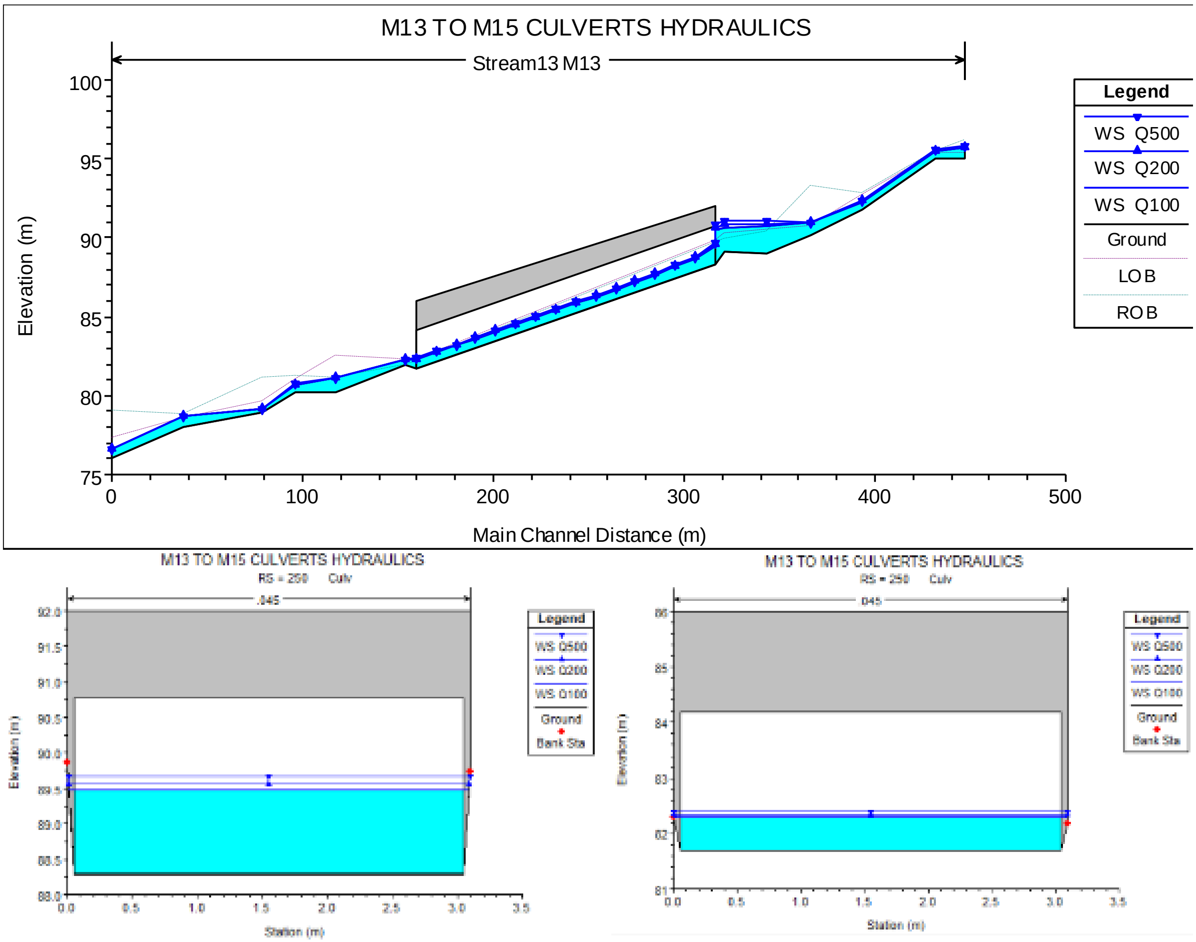


Figure 4-8 Water Surface for the Plan, Longitudinal, and Cross-Sections of M13

Table 4-5 Results of 100 Years of Flood Hydraulic Properties Upstream and Downstream of M13

Reach	River Sta	Profile	E.G. Elev (m)	W.S. Elev (m)	Vel Head (m)	Frctn Loss (m)	C & E Loss (m)	Q Left (m3/s)	Q Channel (m3/s)	Q Right (m3/s)	Top Width (m)	Hydr Depth (m)
M13	345	Q100	90.73	90.64	0.09	0.03	0.01		12.44		9.56	0.98
M13	345	Q200	90.89	90.82	0.08	0.03	0.01		13.75		9.56	1.16
M13	345	Q500	91.10	91.03	0.07	0.02	0.01		15.48		9.56	1.38
M13	340	Q100	90.69	90.52	0.17				12.44		3.10	2.19
M13	340	Q200	90.86	90.68	0.18				13.75		3.10	2.34
M13	340	Q500	91.07	90.87	0.20				15.48		3.10	2.54
M13	250		Culvert									
M13	183	Q100	84.61	82.29	2.31	1.06	0.46		12.44		3.10	0.60
M13	183	Q200	84.81	82.34	2.47	1.13	0.47		13.75		3.10	0.64
M13	183	Q500	85.04	82.39	2.65	1.19	0.48		15.48		3.10	0.69
M13	177	Q100	83.08	82.31	0.78	0.54	0.06		12.44		9.40	0.34
M13	177	Q200	83.21	82.32	0.89	0.55	0.06		13.75		9.40	0.35
M13	177	Q500	83.37	82.33	1.04	0.56	0.07		15.48		9.40	0.37

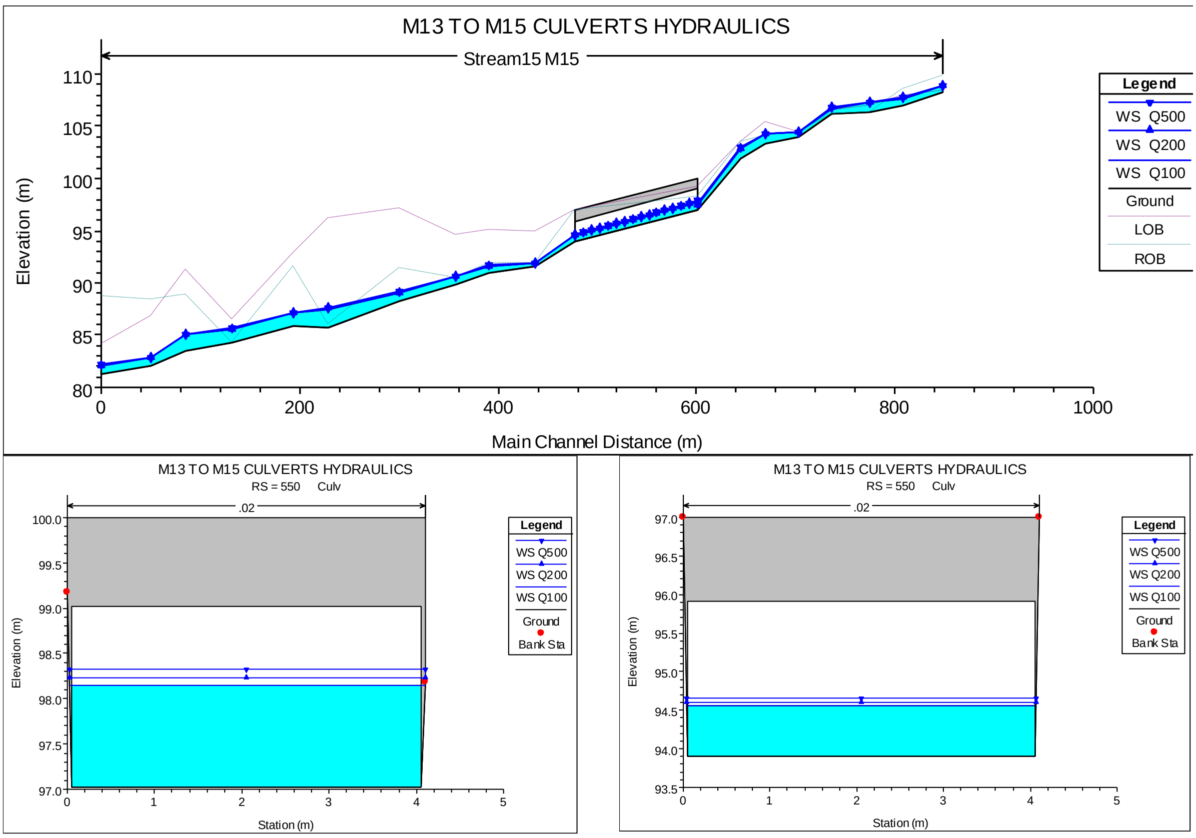


Figure 4-9 Water Surface Profile for Longitudinal and Cross-Sections of M15

Table 4-6 Results of 100 Years of Flood Hydraulic Properties Upstream and Downstream of M15

Reach	River Sta	Profile	E.G. Elev	W.S. Elev	Vel Head	Frctn Loss	C & E Loss	Q Left	Q Channel	Q Right	Top Width	Hydr Depth
			(m)	(m)	(m)	(m)	(m)	(m3/s)	(m3/s)	(m3/s)	(m)	(m)
M15	676	Q100	103.44	102.82	0.61	2.63	0.24		15.10		6.91	0.63
M15	676	Q200	103.49	102.91	0.58	2.41	0.27		16.70		7.33	0.68
M15	676	Q500	103.58	102.98	0.59	2.32	0.28		18.80		7.90	0.70
M15	634	Q100	100.56	97.51	3.06				15.10		4.03	0.48
M15	634	Q200	100.81	97.54	3.28				16.70		4.03	0.52
M15	634	Q500	100.99	97.59	3.39				18.80		4.04	0.57
M15	550		Culvert									
M15	509	Q100	96.27	94.56	1.70	3.14	0.22		15.10		4.02	0.65
M15	509	Q200	96.42	94.61	1.81	3.17	0.22		16.70		4.02	0.70
M15	509	Q500	96.63	94.66	1.97	3.22	0.22		18.80		4.02	0.75
M15	468	Q100	92.91	91.94	0.97	2.97	0.23		15.10		16.17	0.21
M15	468	Q200	93.04	91.95	1.09	0.58	0.01		16.70	0.00	16.36	0.22
M15	468	Q500	93.20	91.96	1.24	2.99	0.30		18.80	0.00	16.62	0.23

5. CONCLUSION AND RECOMMENDATION

This report covers the study of hydraulics and flood risk assessment of the Project structures, including six viaducts and 18 culverts. There is no significant catchment area for M08, M10, M23, M43, M47, M53, and M55 culverts. In addition, the design floods of M09, M10, M11, M12, M14, and M16-M19 catchments are low compared to the slope and size of culverts (2×2 meters)

The risk assessment for the remaining crossings is as follows:

Sub-Structure	Results
Sazlıdere Cable Stayed Bridge	The bridge and channel capacity are adequate for 100- to 500-year flood passage. Therefore, there is no risk of flooding caused by this bridge. It will cause problems around the bridge if the release of the spillway will be more than 506,39 m3/s (Routed flood with the 500-year return period and the spillway design flood).
VIA-01	The bridge capacity is adequate for a 100-years flood passage but the channel capacity may not sufficient for the flood.
VIA-02	The bridge and channel capacity are adequate for 100- to 500-years flood passage. Therefore, there is no risk of flooding caused by this bridge.
VIA-03_1	The viaduct capacity in both streams is adequate for passing the 100-, 200- and 500-years floods.
VIA-03_2	The viaduct capacity in both streams is enough for passing the 100-, 200- and 500-years floods.
VIA-04	The bridge capacity is adequate for a 100-years flood passage. Still, the channel capacity may not sufficient for the flood. Therefore, the area around the bridge may be flooded.
VIA-05	The bridge capacity is adequate for a 100-years flood passage. Still, the social and park area may be inundated
M02	The culvert capacity is adequate for a 100-year flood passage. The generated backwater may flood the area around the culvert.
M03	The culvert capacity is adequate for a 100-year flood passage. The generated backwater may flood the area around the culvert.
M04	The culvert capacity is adequate for 100- to 500-years flood passage, and there is no flood inundation risk around the culvert and its upstream.
M13	The culvert capacity is adequate for 100- to 500-years flood passage, and there is no flood inundation risk around the culvert and its upstream.
M15	The culvert capacity is adequate for 100- to 500-years flood passage. Still, the urban area upstream and downstream of the culverts may be inundated

According to hydraulic calculations, in the floodplain of the catchment area defined in Figure 1-2, the capacity of viaducts is high because of their deck height and multi-span structure. Among the flooding conditions for the culverts that were assessed, Backwater of M02, M03, and M15 must be considered, and the required measures such as river design walls must be implemented upstream of the structures. In addition, downstream of M15, the existing structure with low capacity must be considered for resizing or restoration.

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