Hydraulic Structures Design for Flood Risks Reduction in Four Urban Sub-catchments in Rwanda

October 2021

Table of Contents

List of Tables

List of Figures

1

RUSIZI DRAINAGE DESIGN STRUCTURAL CALCULATIONS

1.1 Introduction

The following structural calculation relates to the design of drains intended to facilitate the drainage of the runoff water for the areas of Mount Cyangungu, Kadashya and Gihundwe, located in Rusizi District.

This document presents the calculations that were carried out in order to determine the section for the drains

1.2 Drain characteristics

The selection of type and dimensions of drains is based on hydraulic calculations taking into account the velocity and free board in order to prevent overflowing.

The flow properties in the drains are obtained from Manning's formula as follows:

$$Q = V \text{ x } S$$

$$V = K\ R^{2/3} I^{1/2}$$

(Manning Strickler formula)

$$S = Y u \text{ x} B$$

$$K = \frac{1}{n}$$

$$R = \frac{S}{\mathrm{P}}$$

Q	: Flow rate in m^3/s
V	: Flow velocity in m/s
S	: Flow cross-section in m^2
n	: Roughness coefficient
R	: Hydraulic radius in m
P	: Wetted perimeter in m
I	: Bottom slope along the canal in m/m
Yu	: Water level in m
B	: Bed width in m

The roughness coefficient depends on the lining of canal walls and bed: for stone masonry canals, the roughness coefficient is taken as 0.015. This drain lining is selected since the stone masonry with cement mortar will reduce the velocity of the flow due to its higher roughness.

The dimensions of the drain are selected in order to limit the velocity to 2 m/s.

The Manning's formula gives the results below for the drainage of Mount Cyangungu, Kadashya and Gihundwe.

Table 1: Mount Cyangugu and Kadashya hydraulic verification through Manning's formula

		Station		Length	Q	i	n	it (V/H)	B	Yu	Free board	h_{tot}	V	S	P	R
	Channel	From	To	m	m3/s	m/m	-	m/m	m	m	m	m	m	m2	m	m
Mount Cyangungu	D01	0+000	0+550	549.58	0.700	0.0040	0.0150	-	0.80	0.55	0.25	0.80	1.59	0.44	1.90	0.23
		0+550	0+862	312.68	0.750	0.0040	0.0150	-	0.80	0.58	0.22	0.80	1.61	0.46	1.96	0.24
	D02	0+000	0+112	112.00	0.150	0.0060	0.0150	-	0.60	0.20	0.40	0.60	1.25	0.12	1.00	0.12
		0+112	0+185	73.01	0.150	0.0060	0.0150	-	0.80	0.16	0.64	0.80	1.20	0.12	1.11	0.11
		0+185	0+376	191.34	0.190	0.0060	0.0150	-	0.80	0.18	0.62	0.80	1.30	0.15	1.17	0.13
	D03	0+000	0+178	178.21	0.100	0.0050	0.0150	-	0.50	0.19	0.41	0.60	1.06	0.09	0.88	0.11
	D04	0+000	0+245	245.20	0.100	0.0050	0.0150	-	0.50	0.19	0.41	0.60	1.06	0.09	0.88	0.11
	D05	0+000	0+310	310.16	0.100	0.0050	0.0150	-	0.50	0.19	0.41	0.60	1.06	0.09	0.88	0.11
	D06	0+000	0+432	431.84	0.290	0.0060	0.0150	-	0.50	0.39	0.21	0.60	1.48	0.20	1.29	0.15
Kadashya	D07	0+000	0+327	326.60	0.290	0.0060	0.0150	-	0.80	0.25	0.55	0.80	1.47	0.20	1.29	0.15
	D08	0+000	0+238	237.96	0.100	0.0060	0.0150	-	0.80	0.12	0.68	0.80	1.05	0.10	1.04	0.09

Table 2: Gihundwe hydraulic verification through Manning's formula

		Station		Length	Q	i	n	it (V/H)	B	Yu	Free board	h_{tot}	V	S	P	R
	Channel	From	To	m	m3/s	m/m	-	m/m	m	m	m	m	m	m2	m	m
Gihundwe 1	D09	0+000	0+148	148.00	0.100	0.0050	0.0150	-	0.60	0.16	0.44	0.60	1.04	0.10	0.92	0.10
		0+148	0+609	461.46	0.250	0.0050	0.0150	-	0.80	0.24	0.56	0.80	1.32	0.19	1.27	0.15
	D10	0+000	0+254	253.98	0.150	0.0050	0.0150	-	0.50	0.25	0.35	0.60	1.18	0.13	1.01	0.13
	D11	0+000	0+111	111.00	0.100	0.0050	0.0150	-	0.60	0.16	0.44	0.60	1.04	0.10	0.92	0.10
		0+111	0+289	178.40	0.200	0.0050	0.0150	-	0.60	0.26	0.34	0.60	1.27	0.16	1.12	0.14
	D12	0+000	0+185	185.00	0.100	0.0060	0.0150	-	0.60	0.15	0.45	0.60	1.11	0.09	0.90	0.10
		0+185	0+400	215.00	0.200	0.0060	0.0150	-	0.60	0.25	0.35	0.60	1.36	0.15	1.09	0.13
		0+400	0+646	246.00	0.350	0.0060	0.0150	-	0.60	0.37	0.23	0.60	1.56	0.22	1.35	0.17
		0+646	0+847	201.00	0.350	0.0060	0.0150	-	0.80	0.28	0.52	0.80	1.55	0.23	1.36	0.17
	D13	0+000	0+169	168.65	0.100	0.0050	0.0150	-	0.50	0.19	0.41	0.60	1.06	0.09	0.88	0.11
	D14	0+000	0+153	152.88	0.150	0.0050	0.0150	-	0.50	0.25	0.35	0.60	1.18	0.13	1.01	0.13
	D15	0+000	0+355	355.00	0.150	0.0050	0.0150	-	0.60	0.21	0.39	0.60	1.18	0.13	1.03	0.12
		0+355	0+561	206.00	0.250	0.0050	0.0150	-	0.80	0.24	0.56	0.80	1.32	0.19	1.27	0.15
	D16	0+000	0+116	116.00	0.050	0.0050	0.0150	-	0.60	0.10	0.50	0.60	0.84	0.06	0.80	0.07
		0+116	0+281	165.00	0.250	0.0050	0.0150	-	0.60	0.31	0.29	0.60	1.35	0.19	1.22	0.15
		0+281	0+408	127.00	0.350	0.0050	0.0150	-	0.80	0.30	0.50	0.80	1.46	0.24	1.40	0.17
	D17	0+000	0+321	320.96	0.100	0.0050	0.0150	-	0.50	0.19	0.41	0.60	1.06	0.09	0.88	0.11
	D18	0+000	0+049	49.49	0.050	0.0050	0.0150	-	0.50	0.12	0.48	0.60	0.87	0.06	0.73	0.08
	D19	0+000	0+149	148.89	0.100	0.0050	0.0150	-	0.50	0.19	0.41	0.60	1.06	0.09	0.88	0.11
Gihundwe 2	D20	0+000	0+128	128.00	0.250	0.0050	0.0150	-	0.50	0.38	0.22	0.60	1.33	0.19	1.25	0.15
		0+128	0+193	65.00	0.500	0.0050	0.0150	-	0.80	0.39	0.41	0.80	1.60	0.31	1.58	0.20
		0+193	0+323	130.00	0.710	0.0050	0.0150	-	0.80	0.51	0.29	0.80	1.74	0.41	1.82	0.22
	D21	0+000	0+390	389.76	0.250	0.0050	0.0150	-	0.50	0.38	0.22	0.60	1.33	0.19	1.25	0.15
	D22	0+000	0+345	345.45	0.250	0.0060	0.0150	-	0.50	0.35	0.25	0.60	1.43	0.17	1.20	0.15

The dimensions selected for each drain are the following:

Table 3: Drain dimensions

	ID	Width B (m)	Height H (m)	Bed Slope i (m/m)
Mount Cyangungu	**D01**	0.80	0.80	0.040
	D02	0.60	0.60	0.060
		0.80	0.80	0.060
	D03	0.50	0.60	0.050
	D04	0.50	0.60	0.050
	D05	0.50	0.60	0.050
	D06	0.50	0.60	0.060
Kadashya	**D07**	0.80	0.80	0.060
	D08	0.80	0.80	0.060
Gihundwe 1	**D09**	0.60	0.60	0.050
		0.80	0.80	0.050
	D10	0.50	0.60	0.050
	D11	0.60	0.60	0.050
	D12	0.60	0.60	0.060
		0.80	0.80	0.060
	D13	0.50	0.60	0.050
	D14	0.50	0.60	0.050
	D15	0.60	0.60	0.050
		0.80	0.80	0.050
	D16	0.60	0.60	0.050
		0.80	0.80	0.050
	D17	0.50	0.60	0.050
	D18	0.50	0.60	0.050
	D19	0.50	0.60	0.050
Gihundwe 2	**D20**	0.50	0.60	0.050
		0.80	0.80	0.050
	D21	0.50	0.60	0.050
	D22	0.50	0.60	0.060

1.3 Culvert characteristics

Culverts are needed where the drains should cross the roads. Two ranges of section have been retained depending on the emplacement and the runoff capacity in each cell:

- 0.60m x 0.60m
- 0.80m x 0.80m

The culvert walls and base will be in stone masonry 45cm thick. The culvert cover will be a reinforced precast slab of 0.15 m thick.

The table below presents a summary of the number of culverts of each type in each cell.

Table 4: Culvert sections

	Section	Number
Mount Cyangungu	0.60m x 0.60m	3
	0.80m x 0.80m	2
Kadashya	0.60m x 0.60m	3
	0.80m x 0.80m	-
Gihundwe 1	0.60m x 0.60m	15
	0.80m x 0.80m	-
Gihundwe 2	0.60m x 0.60m	-
	0.80m x 0.80m	2
Total number of culverts		**25**

1.4 Specific considerations in Gihundwe area

Figure 1: Flood hotspots in Gihundwe

Figure 2: Location 1 flood hotspot in Gihundwe

Figure 3: Location 2 flood hotspot in Gihundwe

The following are the proposed flood remedial actions:

- Location 1: it is recommended to install a covered drain that crosses through existing houses to connect with the new proposed drain. Local authorities will need to speak to the house owners through which the drain will pass to determine the space required, and if deemed necessary, house expropriation may need to be carried out.
- Location 2: It is recommended to install a drain that passes through the adjacent plot of land that has not yet been built. Purchase of this land will need to be done. It is noted that road levels have now been altered allowing for levels on the road to fall towards the unbuilt plot of land.

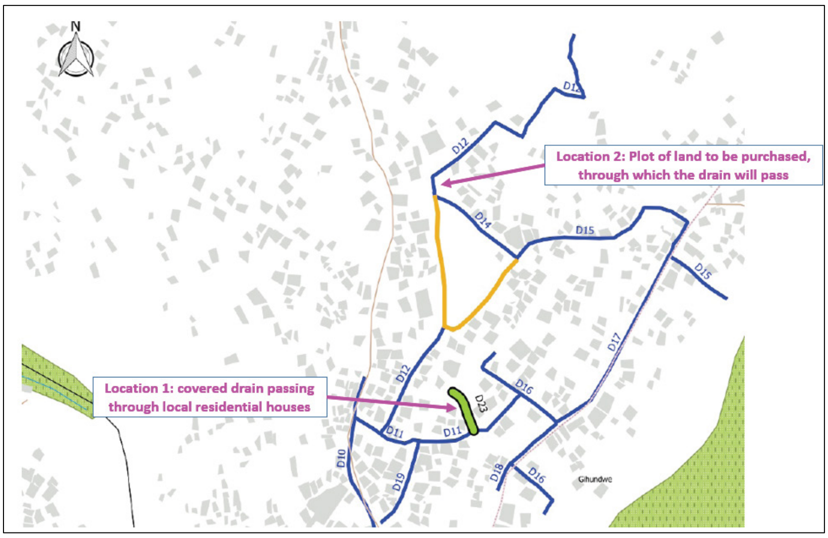


Figure 4: Flood hotspot mitigation proposal in Gihundwe

2

RWABAYANGA STRUCTURAL CALCULATIONS

2.1 Introduction

The following structural calculations relate to the design of the new bridges intended to replace the existing bridges at Rwabayanga site. The bridges are built at crossing points of existing drains and roads.
Hydraulic modelling identified the hydraulic sections necessary, which are incorporated into the geometry of the new bridges. Details can be obtained in the hydraulic modelling report submitted as part of Interim Report No.2.

This document presents the calculations and verifications that were carried out in order to pre-dimension the structural elements of the bridges.

2.2 General presentation of the existing structures

2.2.1 Location of the structures

The location of the structures is presented on drawing no. RW114-IR3-RW-001.

Figure 5: RW-01 current state

Figure 6: RW-04 current state

Figure 7: RW-06 current state

Figure 8: RW-09 current state

2.3 Geometrical characteristics of the proposed structures

The structures will be reinforced concrete beam bridges supported on reinforced concrete abutments with spread footings. The characteristics of the four new bridges at Rwabayanga site are presented in the following tables:

Table 5: Bridges facility components

ID	Number of span	Span(m)	Total height (m)	Hydraulic section		
				Length(m)	Width(m)	Height(m)
RW-01	1	6.90	3.80	6.50	7.20	2.00
RW-04	1	7.40	3.85	7.00	7.20	2.00
RW-06	1	5.40	4.10	5.00	7.20	2.50
RW-09	1	3.70	2.95	3.50	2.00	1.50

Table 6: Bridges geometry

ID	Number of lane	Carriageway (m)	Number of sidewalks	Bridge Type
RW-01	1	4.70	1	Rural
RW-04	1	4.70	1	Rural
RW-06	1	4.70	1	Rural
RW-09	1	1.50	-	Rural

2.4 General hypotheses

2.4.1 Reference documents

The design and calculation of the structures is carried out according to the requirements of the Rwanda Transport Development Agency (RTDA) complemented by the American Association of State Highway and Transportation Officials (AASHTO) Standard.

The reference documents used in this structural calculation are as follows:

- RTDA Bridge Design Manual
- RTDA Road Geometric Design Manual
- AASHTO Load and Resistance Factor Design(LRFD) Bridge Design Specifications.
- Seismic hazard assessment of the Kivu rift segment based on a new seismotectonic zonation model (westernbranch, East African Rift system)
- Open-Channel Hydraulics – CHOW Ven Te – New York – 1959

2.4.2 Material characteristics

- Concrete:
  o Resistance: C25/30
  o Weight: 2403 Kg/m3
- Reinforcement: B500

Given the proximity of the structures to the river and the permeability of the soil, the groundwater level is assumed equal to the water level in the river

2.4.3 Geotechnical data

Since a geotechnical study has not yet been conducted, the following characteristics are assumed based on the recommendations from RTDA Bridge Design Manual (§6.14)

Table 7: Geotechnical data

Symbol	Characteristic	Value	Unit
	Backfill (sandy soil)		
$?_R$	Weight of soil	1922	kg/m3
$?_R$	Angle of friction of soil	30	degree
K_a	Coefficient of earth pressure	0.296	
	Foundation (sandy soil)		
$?_s$	Angle friction of soil	30	degree
$?_s$	Angle of friction between soil and concrete	30	degree
q_a	Nominal bearing resistance	0.240	MPa

2.4.4 Hydrological data

Table 8: Hydrological parameters for the hydraulic design of the structures

Parameter	Unit	RW-01	RW-04	RW-06	RW-09
Discharge for 25-year return period (Q25)	m^3/s	8.444	6.961	3.278	1.138
Corresponding water level	m	0.686	0.575	0.450	0.295
Discharge for 100-year return period (Q100)	m^3/s	15.188	12.097	5.070	1.840
Corresponding water level	m	1.007	0.820	0.597	0.401

2.4.5 Seismic loads

The Peak Ground Acceleration A is taken as 0.26g for Rwabayanga according to the reference "Seismic hazard assessment of the Kivu rift segment based on a new seismotectonic zonation model (western branch, East African Rift system)".

The horizontal and vertical coefficients are therefore:

- kh = 0.5x 0.26 = 0.13 AASHTO 11.8.6
- kv = 0.5x 0.13 = 0.065

2.4.6 Live loads

According to RTDA Bridge Design Manual §6.7, typical bridges in Rwanda shall be designed for HL-93 live load and supplemented by AASHTO LRFD 3.6. For rural structures that do not see heavy traffic, a lesser live load equal to ¾ of HL-93 may be designed for with the approval of RTDA.

The HL-93 load model was developed using probability to account for how many vehicles and of what weights would be expected on a bridge at the same time. The load model also assumes that these vehicles will have some spacing between them in the traffic stream.

HL-93 is the maximum of:

- Design Tandem + Design Lane load
- Design Truck + Design Lane load

Design Truck:
The weights and spacing of axles and wheels for the design truck are specified in the figure below

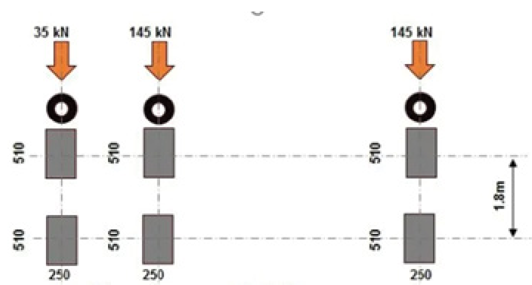


Figure 9: Design Tandem specification

Qtruck = 35 + 145 + 145 = 325 KN or 32.5T

Design Tandem:

It consists of two axles weighing 110KN each and spaced 1200mm apart. The transverse spacing of wheels shall be taken as 1800mm.

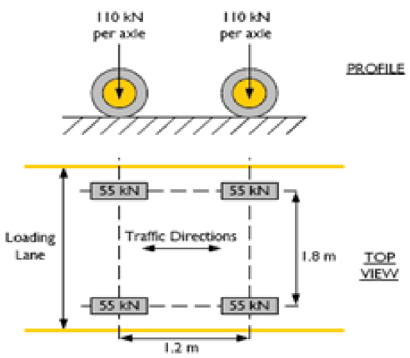


Figure 10: Design Tandem specification

Qtandem = 2 x 110 = 220KN or 22T

Design Lane load

The design lane load consists of a uniformly distributed load of 9.3 N/mm and assumed to occupy 3000 mm transversally

Qlane = 9.3 x 3 = 27.9 KN or 2.79T

The HL-93 system is summarized in the figures below

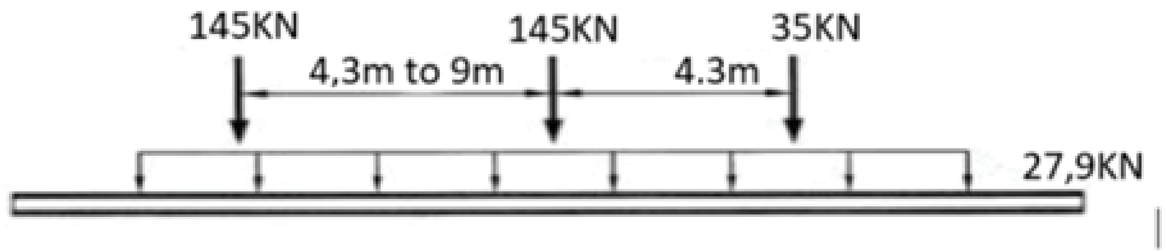


Figure 11: Design Truck and Design Lane distribution

Design Truck + Design Lane load = 35,29T

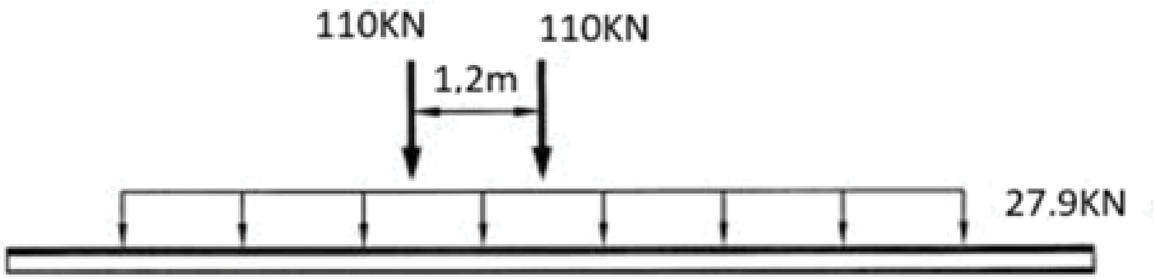


Figure 12: Design Truck and Design Lane distribution

Design Tandem + Design Lane load = 24,79T

So HL-93 = 35,29T for Urban bridges whilst for the rural bridges the design load shall be ¾ x 35.29 = 26,47T.

For the sidewalk a uniformly distributed load of 5 kN/m2 is considered.

2.5 Structural design of the bridges

2.5.1 Load combinations and load factors

Loads factors and loads combinations follow the requirement from AASHTO 3.4.1 are summarized in the table below, where

a – maximum load factor
b – minimum load factor

Table 9: Loads combinations and loads factors

Loads Combinations and Loads Factors							
	DC	Br	LL	LS	EV	EQ	EH
Strength I-a	1.25	1.75	1.75	1.75	1.35	0	1.5
Strength I-b	0.9	1.75	1.75	1.75	1	0	0.9
Strength II-a	1.25	1.35	1.35	1.35	1.35	0	1.5
Strength II-b	0.9	1.35	1.35	1.35	1	0	0.9
Strength III-a	1.25	0	0	0	1.35	0	1.5
Strength III-b	0.9	0	0	0	1	0	0.9
Strength IV-a	1.5	0	0	0	1.35	0	1.5
Strength IV-b	0.9	0	0	0	1	0	0.9
Strength V-a	1.25	1.35	1.35	1.35	1.35	0	1.5
Strength V-b	0.9	1.35	1.35	1.35	1	0	0.9
Extreme Event I-a	1.25	1	1	1	1.35	1	0
Extreme Event I-b	0.9	1	1	1	1	1	0
Extreme Event II-a	1.25	0.5	0.5	0.5	1.35	1	1.5
Extreme Event II-b	0.9	0.5	0.5	0.5	1	0	0.9
Service I	1	1	1	1	1	0	1
Service II	1	1.3	1.3	1.3	1	0	1
Service III	1	0.8	0.8	0.8	1	0	1
Service IV	1	0	0	0	1	0	1

2.5.2 Superstructure

Deck slab

The minimum recommended standard slab width for beam spacing between 1600 mm and 2900 mm is 200mm (RTDA Bridge Design Manual §16.5). Table 10 presents the deck slab width for the four bridges.

Table 10: Deck slab width

ID	Deck slab width (m)
RW-01	0.20
RW-04	0.20
RW-06	0.25
RW-09	0.20

Beams

For simple span the depth of the beam is given by

$h = \frac{L}{13}$ RTDA Bridge Design Manual 3.2

L: Span of the bridge

Table 11: Beams dimensions

ID	Span(m)	Beam depth (m)	Beam width (m)
RW-01	6.90	0.55	0.35
RW-04	7.40	0.60	0.35
RW-06	5.40	0.45	0.30
RW-09	3.70	0.30	0.20

2.5.3 Substructure

Abutments

The abutments and footing are dimensioned to ensure stability against bearing capacity failure, excessive eccentricity and sliding.

Applied load

LLsup- Live load reaction on superstructure and transmitted to the beam seat
DCsup- Dead Load of superstructure components
DC1-DC2-DC3- Dead load of back wall, stem and footing
BR- Vehicular Braking Force
EV1-EV2- Vertical pressure from dead load of earth fill on heel and toe
EH- Horizontal earth pressure load on the structures
EQ- Soil lateral pressure on the structures due to seismic load
LS_H- LS_v – Horizontal and vertical Live load traffic surcharge
R- Resultant of all forces applied on the structure

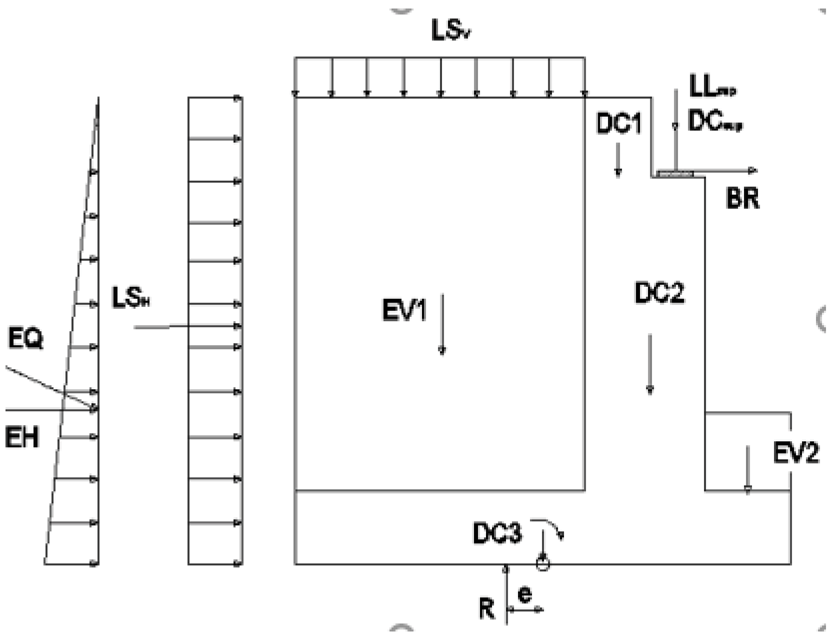


Figure 13: Loads applied on substructure

The table below summarises factored forces and moments around the centre of the base.

Table 12: Factored forces and moments around the centre of the base

	RW-01			RW-04			RW-06			RW-09		
	VV (KN/m)	VH (KN/m)	M (KN)	VV (KN/m)	VH (KN/m)	M (KN)	VV (KN/m)	VH (KN/m)	M (KN)	VV (KN/m)	VH (KN/m)	M (KN)
Strength Ia	844.78	126.37	377.95	971.57	128.12	567.69	710.91	136.16	370.04	220.46	66.42	69.10
Strength Ib	662.45	101.68	306.74	755.01	102.78	452.20	566.83	107.42	311.49	170.29	51.54	55.88
Strength II a	804.13	111.59	339.42	930.92	113.31	520.82	670.26	121.45	321.38	212.26	59.74	60.07
Strength II b	621.80	86.90	268.21	714.36	87.98	405.33	526.18	92.72	262.83	162.09	44.86	46.84
Strength III a	666.93	61.70	209.38	793.72	63.34	362.64	533.06	71.83	157.17	184.58	37.19	29.58
Strength III b	484.60	37.02	138.17	577.16	38.00	247.14	388.98	43.10	98.62	134.42	22.31	16.35
Strength IV a	757.81	61.70	266.56	897.71	63.34	459.42	589.74	71.83	195.64	206.87	37.19	33.55
Strength IV b	484.60	37.02	138.17	577.16	38.00	247.14	388.98	43.10	98.62	134.42	22.31	16.35
Strength V a	804.13	111.59	339.42	930.92	113.31	520.82	670.26	121.45	321.38	212.26	59.74	60.07
Strength V b	621.80	86.90	268.21	714.36	87.98	405.33	526.18	92.72	262.83	162.09	44.86	46.84
Extreme Event I a	768.56	108.00	317.54	895.35	110.06	492.26	634.69	120.46	295.03	205.08	57.41	55.63
Extreme Event I b	586.23	108.00	277.59	678.79	110.06	409.28	490.61	120.46	275.75	154.92	57.41	57.03
Extreme Event II a	717.74	110.09	295.43	844.54	112.66	460.77	583.87	126.02	266.94	194.83	61.46	56.52
Extreme Event II b	535.41	55.50	186.33	627.97	56.51	305.73	439.79	61.48	159.44	144.67	30.66	27.64
Service I	622.58	78.09	262.58	720.39	79.24	408.45	513.28	84.64	242.19	163.83	41.49	42.96
Service II	653.07	89.17	291.47	750.87	90.35	443.60	543.77	95.67	278.68	169.98	46.51	49.74
Service III	602.26	70.70	243.31	700.06	71.84	385.02	492.96	77.29	217.86	159.73	38.15	38.45
Service IV	520.95	41.14	166.25	618.75	42.23	291.28	411.65	47.89	120.55	143.33	24.79	20.38

The verification of the stability is carried out according to AASHTO

Table 13: RW-01 and RW-04 Stability verifications and summary results

	RW-01									RW-04								
	Eccentricity			Bearing resistance			Sliding			Eccentricity			Bearing resistance			Sliding		
	$\Sigma M/\Sigma V_v$	e max	$\Sigma M/\Sigma V_v$ < e max	σv	Qn	σv < Qn	VH	Rn	VH < Rn	$\Sigma M/\Sigma V_v$	e max	$\Sigma M/\Sigma V_v$ < e max	σv	Qn	σv < Qn	VH	Rn	VH < Rn
Strength Ia	0.45	0.85	OK	337.21	403.54	OK	126.37	390.19	OK	0.58	1.03	OK	331.44	447.08	OK	128.12	448.75	OK
Strength Ib	0.46	0.85	OK	267.77	401.62	OK	101.68	305.97	OK	0.60	1.03	OK	260.16	445.27	OK	102.78	348.72	OK
Strength II a	0.42	0.85	OK	314.63	406.63	OK	111.59	371.41	OK	0.56	1.03	OK	312.28	450.14	OK	113.31	429.97	OK
Strength II b	0.43	0.85	OK	245.06	405.51	OK	86.90	287.20	OK	0.57	1.03	OK	240.91	449.16	OK	87.98	329.95	OK
Strength III a	0.31	0.85	OK	240.59	419.61	OK	61.70	308.04	OK	0.46	1.03	OK	249.11	462.55	OK	63.34	366.60	OK
Strength III b	0.29	0.85	OK	171.25	423.01	OK	37.02	223.83	OK	0.43	1.03	OK	177.94	465.96	OK	38.00	266.58	OK
Strength IV a	0.35	0.85	OK	281.03	415.12	OK	61.70	350.02	OK	0.51	1.03	OK	291.80	455.95	OK	63.34	414.64	OK
Strength IV b	0.29	0.85	OK	171.25	423.01	OK	37.02	223.83	OK	0.43	1.03	OK	177.94	465.96	OK	38.00	266.58	OK
Strength V a	0.42	0.85	OK	314.63	406.63	OK	111.59	371.41	OK	0.56	1.03	OK	312.28	450.14	OK	113.31	429.97	OK
Strength V b	0.43	0.85	OK	245.06	405.51	OK	86.90	287.20	OK	0.57	1.03	OK	240.91	449.16	OK	87.98	329.95	OK
Extreme Event I a	0.41	1.36	OK	298.62	407.72	OK	108.00	354.98	OK	0.55	1.64	OK	298.41	451.32	OK	110.06	413.54	OK
Extreme Event I b	0.47	1.36	OK	238.99	400.33	OK	108.00	270.77	OK	0.60	1.64	OK	234.54	444.77	OK	110.06	313.52	OK
Extreme Event II a	0.41	0.85	OK	278.54	407.91	OK	110.09	331.51	OK	0.55	1.03	OK	280.69	451.84	OK	112.66	390.07	OK
Extreme Event II b	0.35	0.85	OK	198.01	415.56	OK	55.50	247.30	OK	0.49	1.03	OK	200.87	458.96	OK	56.51	290.05	OK
Service I	0.42	0.85	OK	243.53	240.00	OK	78.09	287.56	OK	0.57	1.03	OK	242.88	240.00	OK	79.24	332.73	OK
Service II	0.45	0.85	OK	260.46	240.00	OK	89.17	301.64	OK	0.59	1.03	OK	257.29	240.00	OK	90.35	346.81	OK
Service III	0.40	0.85	OK	232.35	240.00	OK	70.70	278.17	OK	0.55	1.03	OK	233.35	240.00	OK	71.84	323.34	OK
Service IV	0.32	0.85	OK	188.63	240.00	OK	41.14	240.62	OK	0.47	1.03	OK	195.90	240.00	OK	42.23	285.79	OK

Table 14: RW-06 and RW-09 Stability verifications and summary results

	RW-06									RW-09								
	Eccentricity			Bearing resistance			Sliding			Eccentricity			Bearing resistance			Sliding		
	$\Sigma M/\Sigma V_v$	e max	$\Sigma M/\Sigma V_v$ < e max	σv	Qn	σv < Qn	VH	Rn	VH < Rn	$\Sigma M/\Sigma V_v$	e max	$\Sigma M/\Sigma V_v$ < e max	σv	Qn	σv < Qn	VH	Rn	VH < Rn
Strength Ia	0.52	0.80	OK	329.28	338.74	OK	136.16	328.36	OK	0.31	0.45	OK	187.93	231.63	OK	66.42	101.83	OK
Strength Ib	0.55	0.80	OK	269.80	335.67	OK	107.42	261.81	OK	0.33	0.45	OK	148.89	230.42	OK	51.54	78.65	OK
Strength II a	0.48	0.80	OK	299.09	343.04	OK	121.45	309.58	OK	0.28	0.45	OK	172.01	234.11	OK	59.74	98.04	OK
Strength II b	0.50	0.80	OK	239.07	340.94	OK	92.72	243.03	OK	0.29	0.45	OK	132.64	233.63	OK	44.86	74.87	OK
Strength III a	0.29	0.80	OK	204.21	361.82	OK	71.83	246.21	OK	0.16	0.45	OK	124.76	243.74	OK	37.19	85.26	OK
Strength III b	0.25	0.80	OK	144.44	365.90	OK	43.10	179.66	OK	0.12	0.45	OK	86.35	246.64	OK	22.31	62.08	OK
Strength IV a	0.33	0.80	OK	232.50	358.15	OK	71.83	272.39	OK	0.16	0.45	OK	140.19	243.59	OK	37.19	95.55	OK
Strength IV b	0.25	0.80	OK	144.44	365.90	OK	43.10	179.66	OK	0.12	0.45	OK	86.35	246.64	OK	22.31	62.08	OK
Strength V a	0.48	0.80	OK	299.09	343.04	OK	121.45	309.58	OK	0.28	0.45	OK	172.01	234.11	OK	59.74	98.04	OK
Strength V b	0.50	0.80	OK	239.07	340.94	OK	92.72	243.03	OK	0.29	0.45	OK	132.64	233.63	OK	44.86	74.87	OK
Extreme Event I a	0.46	1.28	OK	279.56	344.56	OK	120.46	293.15	OK	0.27	0.72	OK	163.08	235.06	OK	57.41	94.72	OK
Extreme Event I b	0.56	1.28	OK	236.34	334.33	OK	120.46	226.60	OK	0.37	0.72	OK	145.63	227.08	OK	57.41	71.55	OK
Extreme Event II a	0.46	0.80	OK	255.45	345.36	OK	126.02	269.68	OK	0.29	0.45	OK	159.73	233.54	OK	61.46	89.99	OK
Extreme Event II b	0.36	0.80	OK	177.70	355.05	OK	61.48	203.13	OK	0.19	0.45	OK	102.03	241.38	OK	30.66	66.82	OK
Service I	0.47	0.80	OK	227.49	240.00	OK	84.64	237.08	OK	0.26	0.45	OK	128.44	240.00	OK	41.49	75.67	OK
Service II	0.51	0.80	OK	250.01	240.00	OK	95.67	251.16	OK	0.29	0.45	OK	139.93	240.00	OK	46.51	78.51	OK
Service III	0.44	0.80	OK	212.84	240.00	OK	77.29	227.69	OK	0.24	0.45	OK	121.14	240.00	OK	38.15	73.78	OK
Service IV	0.29	0.80	OK	157.46	240.00	OK	47.89	190.13	OK	0.14	0.45	OK	94.57	240.00	OK	24.79	66.20	OK

ID	Abutment					Footing				
	Backwall height (m)	Backwall thickness (m)	Beam seat (m)	Stem (m)	Stem height (m)	Thickness (m)	Width (m)	Toe (m)	Heel (m)	Embedment (m)
RW-01	0.60	0.55	0.45	1.00	2.60	0.60	3.40	0.70	1.70	0.65
RW-04	0.65	0.55	0.45	1.00	2.60	0.60	4.10	0.70	2.40	0.65
RW-06	0.50	0.25	0.45	0.70	3.00	0.60	3.20	0.50	2.00	0.65
RW-09	0.35	0.25	0.25	0.50	2.10	0.50	1.80	0.50	0.80	0.65

2.5.4 Wing walls

The wing walls will have a width of 0.35m and supported by the same footing as the abutment. As the abutment is subjected to additional horizontal braking loads, a verification of stability is not needed for these.

BISHENYI STRUCTURAL CALCULATIONS

3.1 Introduction

The following structural calculation relates to the design of the new bridges intended to replace the existing structures at Bishenyi site.

The hydraulic modelling identified the hydraulic sections necessary, which are incorporated into the geometry of the new bridges. Details can be obtained in the hydraulic modelling report submitted as part of Interim Report No.2.

This document presents the calculations and verifications that were carried out in order to pre-dimension the structural elements of the bridges and related stilling basins.

3.2 General presentation of the existing structures

3.2.1 Location of the structures

The location of the structures is presented on drawing no. RW114-IR3-BI-001.

Figure 14: BI-02 current state

Figure 15: BI-03 current state

Figure 16: BI-05 current state

Figure 17: BI-06 current state

3.3 Geometrical characteristics of the proposed structures

The structures will be reinforced concrete beam bridges supported on reinforced concrete abutments with spread footings.

The characteristics of the new bridges at Bishenyi site are presented in the following tables:

Table 15: Bridges geometries

ID	Number of spans	Span(m)	Total height (m)	Hydraulic section		
				Length(m)	Width(m)	Height(m)
BI-02	1	4.90	5.10	4.50	7.20	4.20
BI-03	1	4.40	4.00	4.00	7.20	2.50
BI-05	1	7.90	5.35	7.50	7.20	4.10
BI-06	1	6.40	5.75	6.00	12.90	4.60

Table 16: Bridges facilities components

ID	Number of lane	Carriageway (m)	Number of sidewalks	Bridge Type
BI-02	1	4.70	1	Rural
BI-03	1	4.70	1	Rural
BI-05	1	4.70	1	Rural
BI-06	2	9.40	2	Urban

3.4 General hypotheses

3.4.1 Reference documents

The design and calculation of the structures is carried out according to the requirements of the Rwanda Transport Development Agency (RTDA) complemented by the American Association of State Highway and Transportation Officials (AASHTO) Standard.

The reference documents used in this structural calculation are as follows:

- RTDA Bridge Design Manual
- RTDA Road Geometric Design Manual
- AASHTO Load and Resistance Factor Design(LRFD) Bridge Design Specifications
- Seismic hazard assessment of the Kivu rift segment based on a new seismotectonic zonation model (western branch, East African Rift system)
- Open-Channel Hydraulics – CHOW Ven Te – New York – 1959

3.4.2 Material characteristics

- Concrete:
  - o Resistance: C25/30
  - o Weight: 2403 Kg/m3
- Reinforcement: B500

Given the proximity of the structures to the river and the permeability of the soil, the groundwater level is assumed equal to the water level in the river.

3.4.3 Geotechnical data

Since a geotechnical study has not yet been conducted, the following characteristics are assumed based on the recommendations from RTDA Bridge Design Manual (§6.14)

Table 17: Geotechnical data

Symbol	Characteristic	Value	Unit
	Backfill (sandy soil)		
□$_R$	Weight of soil	1922	kg/m3
□ $_R$	Angle of friction of soil	30	degrees
K_a	Coefficient of earth pressure	0,296	
	Foundation (sandy soil)		
□ $_s$	Angle friction of soil	30	degrees
□$_s$	Angle of friction between soil and concrete	30	degrees
q_a	Nominal bearing resistance	0,240	MPa

3.4.4 Hydrological data

The following parameters were used for the hydraulic design of the structures:

Table 18 : Hydrological parameters for the hydraulic design of the structures

Parameter	Unit	BI-02	BI-03	BI-05	BI-06
Discharge for 25-year return period (Q25)	m3/s	16.371	6.36	38.82	18.408
Corresponding water level	m	1.44	0.82	1.69	1.22
Discharge for 100-year return period (Q100)	m3/s	30.206	14	77.02	33.516
Corresponding water level	m	2.24	1.43	2.73	1.85

3.4.5 Seismic loads

The Peak Ground Acceleration A is taken as 0.18g for Bishenyi according to the reference "Seismic hazard assessment of the Kivu rift segment based on a new seismotectonic zonation model (western branch, East African Rift system)".

The horizontal and vertical coefficients are therefore:

- $k_h = 0.5 \times 0.18 = 0.09$ AASHTO 11.8.6
- $k_v = 0.5 \times 0.09 = 0.045$

3.4.6 Live loads

According to RTDA Bridge Design Manual §6.7, road bridges in Rwanda shall be designed for HL-93 live load and supplemented by AASHTO Load and Resistance Factor Design (LRFD) 3.6. For rural structures that do not see heavy traffic, a lesser live load equal to ¾ of HL-93 may be designed for with the approval of RTDA.

The HL-93 load model was developed using probability to account for how many vehicles and of what weights would be expected on a bridge at the same time. The load model also assumes that these vehicles will have some spacing between them in the traffic stream.

HL-93 is the maximum of:
- Design Tandem + Design Lane load
- Design Truck + Design Lane load

Design Truck:
The weights and spacing of axles and wheels for the design truck are specified in the figure below

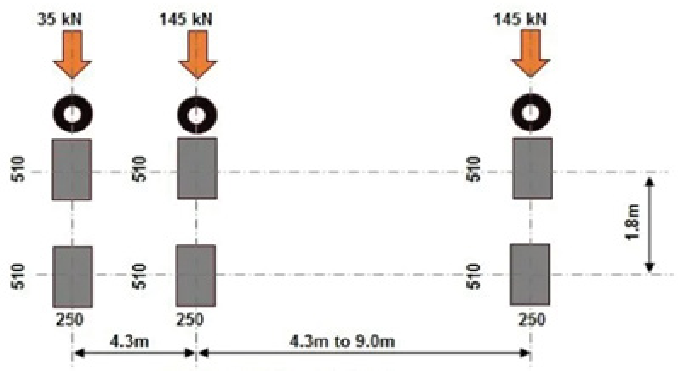
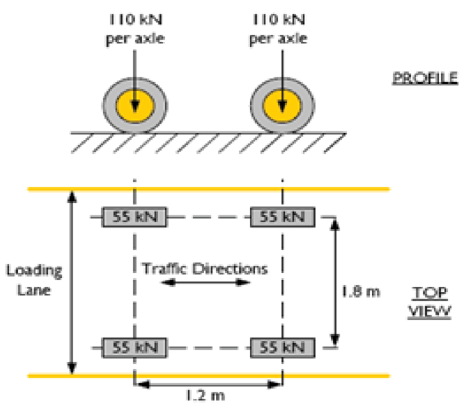


Figure 18: Design Truck specification

Qtruck = 35 + 145 + 145 = 325 KN or 32.5T

Design Tandem:

It consists of two axles weighing 110KN each and spaced 1200mm apart. The transverse spacing of wheels shall be taken as 1800mm.



Qtandem = 2 x 110 = 220KN or 22T

Design Lane load

The design lane load consists of a uniformly distributed load of 9.3 N/mm and assumed to occupy 3000 mm transversally

Qlane = 9.3 x 3 = 27.9 KN or 2.79T

Figure 19: Design Tandem specification

The HL-93 system is summarised in the figures below

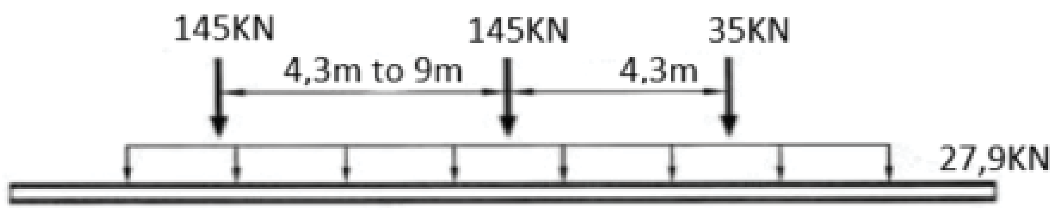


Figure 20: Design Truck and Design Lane distribution

Design Truck + Design Lane load = 35.29T

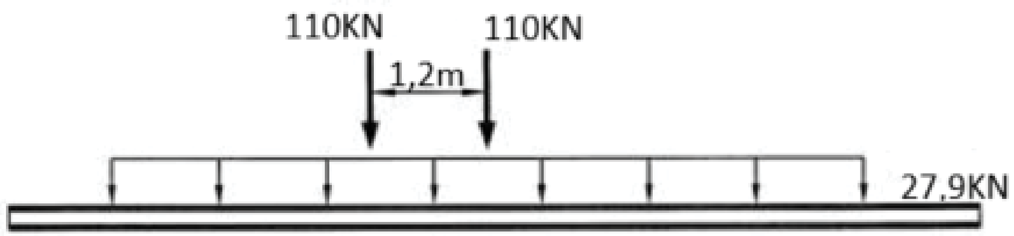


Figure 21: Design Tandem and Design lane distribution

Design Tandem + Design Lane load = 24.79T
So HL-93 = 35,29T for Urban bridges whilst for the rural bridges the design load shall be ¾ x 35.29 = 26,47 T.
For the sidewalk, a uniformly distributed load of 5 kN/m2 is considered.

3.5 Stilling basins

In order to keep the road level of the bridges BI-02 and BI-06 at the same level of the existing road, excavations are needed. These excavations require a drop in the canal bed upstream of the bridges. After verification of results, it was found that a stilling basin will be needed under each of the two bridges.

3.4.1 Reference documents

The following table presents a summary of the different design parameters

Table 19: Design parameter for basin BI-02 and BI-06

Symbol	Explanation	Unit	BI-02	BI-06
L	Drop width	m	4.50	6.00
Q	Discharge	m3/s	30.21	33.52
q	Discharge per unit width	m2/s	6.71	5.59
h	Drop height	m	2.10	1.60

Manning's formula is used to determine the characteristics of the flow in the canal downstream of the basin.

Table 20: Flow properties in the canals downstream of basins BI-02 and BI-06

Symbol	Explanation	Unit	BI-02	BI-06
B_3	Canal width	m	6	6
S	Bed slope	m/m	0.005	0.005
n	Roughness coefficient	-	0.015	0.015
m	Canal wall slope	m/m	0	0
y_3	Water level	m	1.04	1.11

The flow geometry of the hydraulic jump is determined using the equations presented in the book "Open Channel Hydraulics". These equations describe the geometry as functions of the drop number, which is defined as:

$$D = \frac{q^{2}}{gh^{3}}$$

The properties of the hydraulic jump in the basins are thus the following:

Table 21: Properties of the hydraulic jump in basins BI-02 and BI-06

Symbol	Explanation	Unit	BI-02	BI-06
L_d	Drop length	m	7.47	6.43
y_p	Pool depth under the nappe	m	1.80	1.51
y1	Depth at the beginning of the hydraulic jump	m	0.84	0.78
y2	Tailwater depth	m	2.88	2.48
L	Length of the hydraulic jump	m	17.31	14.90

The length thus recommended for the basins is given by the length of the drop and the length of the jump:

$$\mathrm{Lb} = \mathrm{Ld} + \mathrm{L}$$

The dimensions selected for the basins are as follows:

Table 22: Geometry of basins BI-02 and BI-06

Symbol	Explanation	Unit	BI-02	BI-06
L_b	Basin length	m	25	21.5
D	Sill height	m	1.20	0.70
h_w	Basin wall height	m	3.15	2.75

3.5.2 Basin walls

The basin walls are gravity masonry walls. They must be designed to resist overturning, sliding and bearing pressure failure. The dimensions of the basin walls are given in Table 9.

Table 23: Basins BI-02 and BI-06 walls dimensions

Symbol	Explanation	Unit	BI-02	BI-06
hw	Height	m	3.15	2.75
$b_{w,top}$	Top width	m	0.30	0.30
$b_{w,bottom}$	Bottom width	m	1.00	0.90

3.6 Structural design of the bridges

3.6.1 Load combinations and load factors

Loads factors and loads combinations follow the requirement from AASHTO 3.4.1 are summarised in the table below, where

a – maximum load factor
b – minimum load factor

Table 24: loads combinations and loads factors

Loads Combinations and Loads Factors							
	DC	Br	LL	LS	EV	EQ	EH
Strength I-a	1.25	1.75	1.75	1.75	1.35	0	1.5
Strength I-b	0.9	1.75	1.75	1.75	1	0	0.9
Strength II-a	1.25	1.35	1.35	1.35	1.35	0	1.5
Strength II-b	0.9	1.35	1.35	1.35	1	0	0.9
Strength III-a	1.25	0	0	0	1.35	0	1.5
Strength III-b	0.9	0	0	0	1	0	0.9
Strength IV-a	1.5	0	0	0	1.35	0	1.5
Strength IV-b	0.9	0	0	0	1	0	0.9
Strength V-a	1.25	1.35	1.35	1.35	1.35	0	1.5
Strength V-b	0.9	1.35	1.35	1.35	1	0	0.9
Extreme Event I-a	1.25	1	1	1	1.35	1	0
Extreme Event I-b	0.9	1	1	1	1	1	0
Extreme Event II-a	1.25	0.5	0.5	0.5	1.35	1	1.5
Extreme Event II-b	0.9	0.5	0.5	0.5	1	0	0.9
Service I	1	1	1	1	1	0	1
Service II	1	1.3	1.3	1.3	1	0	1
Service III	1	0.8	0.8	0.8	1	0	1
Service IV	1	0	0	0	1	0	1

3.6.2 Superstructure

Deck slab

The minimum recommended standard slab width for beam spacing between 1600 mm and 2900 mm is 200 mm (RTDA Bridge Design Manual §16.5)

The table below presents the deck slab width for the 4 bridges.

Table 25: Deck slab width

ID	Deck slab width (m)
BI-02	0.20
BI-03	0.20
BI-05	0.25
BI-06	0.20

Beams

For simple span the depth of the beam is given by

$$h = \frac{L}{13}$$ RTDA Bridge Design Manual 3.2

L: Span of the bridge

Table 26: Beam dimensions

ID	Span(m)	Beam depth (m)	Beam width (m)
BI-02	4.90	0.40	0.25
BI-03	4.40	0.35	0.25
BI-05	7.90	0.60	0.35
BI-06	6.40	0.50	0.30

3.6.3 Substructure

Abutments

The abutments and footing are dimensioned to ensure stability against bearing capacity failure, excessive eccentricity and sliding.

Applied loads

LLsup- Live load reaction on superstructure and transmitted to the beam seat
DCsup- Dead Load of superstructure components
DC1-DC2-DC3- Dead load of backwall, stem and footing
BR- Vehicular Braking Force
EV1-EV2- Vertical pressure from dead load of earth fill on heel and toe
EH- Horizontal earth pressure load on the structures
EQ- Soil lateral pressure on the structures due to seismic load
LSH- LSv – Horizontal and vertical Live load traffic surcharge
R- Resultant of all forces applied on the structure

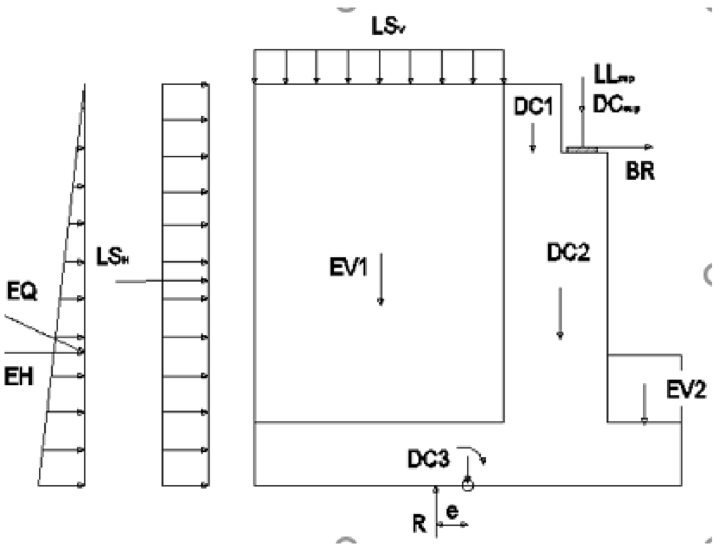


Figure 22: Loads applied on substructure

The table below summarises factored forces and moments around the centre of the base.

Table 27: Factored forces and moments around the centre of the base

	BI-02			BI-03			BI-05			BI-06		
	VV (KN/m)	VH (KN/m)	M (KN)	VV (KN/m)	VH (KN/m)	M (KN)	VV (KN/m)	VH (KN/m)	M (KN)	VV (KN/m)	VH (KN/m)	M (KN)
Strength Ia	746.03	154.77	423.14	598.44	136.16	199.93	1213.82	192.03	885.62	946.86	210.28	113.00
Strength Ib	593.36	119.38	356.05	484.99	107.42	161.17	932.39	143.10	690.14	750.27	153.77	75.43
Strength II a	705.38	139.62	365.21	557.79	121.45	174.39	1173.16	176.09	815.97	896.44	194.51	82.28
Strength II b	552.71	104.23	298.11	444.34	92.72	135.63	891.73	127.17	620.48	699.85	138.00	44.71
Strength III a	568.18	88.47	169.68	420.58	71.83	88.19	1035.96	122.31	580.87	726.27	141.28	-21.41
Strength III b	415.51	53.08	102.58	307.14	43.10	49.42	754.53	73.39	385.38	529.68	84.77	-58.98
Strength IV a	619.88	88.47	210.40	463.08	71.83	108.29	1159.86	122.31	717.72	806.28	141.28	26.88
Strength IV b	415.51	53.08	102.58	307.14	43.10	49.42	754.53	73.39	385.38	529.68	84.77	-58.98
Strength V a	705.38	139.62	365.21	557.79	121.45	174.39	1173.16	176.09	815.97	896.44	194.51	82.28
Strength V b	552.71	104.23	298.11	444.34	92.72	135.63	891.73	127.17	620.48	699.85	138.00	44.71
Extreme Event I a	669.81	137.79	331.86	522.21	116.66	163.07	1137.59	180.80	788.28	852.32	203.58	99.23
Extreme Event I b	517.14	137.79	318.43	408.77	116.66	163.58	856.16	180.80	680.04	655.73	203.58	169.98
Extreme Event II a	618.99	148.33	304.16	471.40	122.23	163.87	1086.78	201.65	773.91	789.29	230.96	151.10
Extreme Event II b	466.33	72.02	175.00	357.95	61.48	81.35	805.35	93.31	472.46	592.70	104.48	-20.58
Service I	537.82	96.87	272.65	425.77	84.64	127.86	905.72	121.38	628.81	687.74	133.62	55.20
Service II	568.31	108.23	316.10	456.26	95.67	147.02	936.21	133.33	681.06	725.55	145.45	78.24
Service III	517.49	89.29	243.68	405.44	77.29	115.09	885.40	113.41	593.98	662.53	125.73	39.84
Service IV	436.19	58.98	127.81	324.14	47.89	64.01	804.09	81.54	454.66	561.68	94.19	-21.61

The verification of the stability is carried out according to AASHTO:

Table 28: BI-03 and BI-03 stability verifications and summary results

	BI-02									BI-03								
	Eccentricity			Bearing resistance			Sliding			Eccentricity			Bearing resistance			Sliding		
	ΣM/ΣVv	e max	ΣM/ΣVv < e max	σv	Qn	σv < Qn	VH	Rn	VH < Rn	ΣM/ΣVv	e max	ΣM/ΣVv < e max	σv	Qn	σv < Qn	VH	Rn	VH < Rn
Strength Ia	0.57	0.88	OK	315.36	334.11	OK	154.77	344.58	OK	0.33	0.68	OK	399.18	412.33	OK	136.16	276.41	OK
Strength Ib	0.60	0.88	OK	258.00	330.97	OK	119.38	274.06	OK	0.33	0.68	OK	308.77	411.21	OK	107.42	224.01	OK
Strength II a	0.52	0.88	OK	286.22	338.80	OK	139.62	325.80	OK	0.31	0.68	OK	377.35	415.89	OK	121.45	257.63	OK
Strength II b	0.54	0.88	OK	228.27	336.76	OK	104.23	255.29	OK	0.31	0.68	OK	286.88	415.86	OK	92.72	205.23	OK
Strength III a	0.30	0.88	OK	195.74	358.79	OK	88.47	262.43	OK	0.21	0.68	OK	306.63	429.68	OK	71.83	194.26	OK
Strength III b	0.25	0.88	OK	138.22	363.34	OK	53.08	191.92	OK	0.16	0.68	OK	216.91	434.66	OK	43.10	141.86	OK
Strength IV a	0.34	0.88	OK	219.72	355.16	OK	88.47	286.31	OK	0.23	0.68	OK	355.52	423.80	OK	71.83	213.89	OK
Strength IV b	0.25	0.88	OK	138.22	363.34	OK	53.08	191.92	OK	0.16	0.68	OK	216.91	434.66	OK	43.10	141.86	OK
Strength V a	0.52	0.88	OK	286.22	338.80	OK	139.62	325.80	OK	0.31	0.68	OK	377.35	415.89	OK	121.45	257.63	OK
Strength V b	0.54	0.88	OK	228.27	336.76	OK	104.23	255.29	OK	0.31	0.68	OK	286.88	415.86	OK	92.72	205.23	OK
Extreme Event I a	0.50	1.40	OK	266.95	340.89	OK	137.79	309.37	OK	0.31	1.08	OK	365.30	416.16	OK	116.66	241.20	OK
Extreme Event I b	0.62	1.40	OK	227.97	329.45	OK	137.79	238.86	OK	0.40	1.08	OK	294.07	405.47	OK	116.66	188.80	OK
Extreme Event II a	0.49	0.88	OK	245.90	341.27	OK	148.33	285.90	OK	0.35	0.68	OK	353.34	414.16	OK	122.23	217.73	OK
Extreme Event II b	0.38	0.88	OK	169.61	351.94	OK	72.02	215.39	OK	0.23	0.68	OK	242.09	427.07	OK	61.48	165.33	OK
Service I	0.51	0.88	OK	216.33	240.00	OK	96.87	248.41	OK	0.30	0.68	OK	291.09	240.00	OK	84.64	196.65	OK
Service II	0.56	0.88	OK	238.03	240.00	OK	108.23	262.49	OK	0.32	0.68	OK	307.45	240.00	OK	95.67	210.74	OK
Service III	0.47	0.88	OK	202.29	240.00	OK	89.29	239.02	OK	0.28	0.68	OK	280.34	240.00	OK	77.29	187.26	OK
Service IV	0.29	0.88	OK	149.69	240.00	OK	58.98	201.47	OK	0.20	0.68	OK	238.66	240.00	OK	47.89	149.71	OK

Table 29: BI-05 and BI-06 stability verifications and summary results

	BI-05									BI-06								
	Eccentricity			Bearing resistance			Sliding			Eccentricity			Bearing resistance			Sliding		
	ΣM/ΣVv	e max	ΣM/ΣVv < e max	σv	Qn	σv < Qn	VH	Rn	VH < Rn	ΣM/ΣVv	e max	ΣM/ΣVv < e max	σv	Qn	σv < Qn	VH	Rn	VH < Rn
Strength Ia	0.73	1.13	OK	399.18	412.33	OK	192.03	560.64	OK	0.12	0.88	OK	290.33	346.66	OK	210.28	437.34	OK
Strength Ib	0.74	1.13	OK	308.77	411.21	OK	143.10	430.65	OK	0.10	0.88	OK	227.43	348.20	OK	153.77	346.54	OK
Strength II a	0.70	1.13	OK	377.35	415.89	OK	176.09	541.86	OK	0.09	0.88	OK	270.30	348.92	OK	194.51	414.05	OK
Strength II b	0.70	1.13	OK	286.88	415.86	OK	127.17	411.87	OK	0.06	0.88	OK	207.53	351.19	OK	138.00	323.25	OK
Strength III a	0.56	1.13	OK	306.63	429.68	OK	122.31	478.49	OK	-0.03	0.88	OK	204.07	358.73	OK	141.28	335.45	OK
Strength III b	0.51	1.13	OK	216.91	434.66	OK	73.39	348.50	OK	-0.11	0.88	OK	142.28	365.24	OK	84.77	244.65	OK
Strength IV a	0.62	1.13	OK	355.52	423.80	OK	122.31	535.72	OK	0.03	0.88	OK	234.84	353.68	OK	141.28	372.40	OK
Strength IV b	0.51	1.13	OK	216.91	434.66	OK	73.39	348.50	OK	-0.11	0.88	OK	142.28	365.24	OK	84.77	244.65	OK
Strength V a	0.70	1.13	OK	377.35	415.89	OK	176.09	541.86	OK	0.09	0.88	OK	270.30	348.92	OK	194.51	414.05	OK
Strength V b	0.70	1.13	OK	286.88	415.86	OK	127.17	411.87	OK	0.06	0.88	OK	207.53	351.19	OK	138.00	323.25	OK
Extreme Event I a	0.69	1.80	OK	365.30	416.16	OK	180.80	525.43	OK	0.12	1.40	OK	260.88	346.89	OK	203.58	393.67	OK
Extreme Event I b	0.79	1.80	OK	294.07	405.47	OK	180.80	395.44	OK	0.26	1.40	OK	219.93	335.00	OK	203.58	302.87	OK
Extreme Event II a	0.71	1.13	OK	353.34	414.16	OK	201.65	501.96	OK	0.19	0.88	OK	253.21	340.69	OK	230.96	364.56	OK
Extreme Event II b	0.59	1.13	OK	242.09	427.07	OK	93.31	371.97	OK	-0.03	0.88	OK	166.05	359.15	OK	104.48	273.76	OK
Service I	0.69	1.13	OK	291.09	240.00	OK	121.38	418.34	OK	0.08	0.88	OK	205.94	240.00	OK	133.62	317.65	OK
Service II	0.73	1.13	OK	307.45	240.00	OK	133.33	432.42	OK	0.11	0.88	OK	220.91	240.00	OK	145.45	335.12	OK
Service III	0.67	1.13	OK	280.34	240.00	OK	113.41	408.95	OK	0.06	0.88	OK	196.03	240.00	OK	125.73	306.01	OK
Service IV	0.57	1.13	OK	238.66	240.00	OK	81.54	371.39	OK	-0.04	0.88	OK	157.03	240.00	OK	94.19	259.43	OK

Table 30: Abutment and footing dimensions

ID	Abutment					Footing				
	Backwall height (m)	Backwall width (m)	Beam seat (m)	Stem (m)	Stem height (m)	Width (m)	Length (m)	Toe (m)	Heel (m)	Embedment (m)
BI-02	0.45	0.25	0.45	0.70	4.20	0.50	5.25	2.25	2.30	0
BI-03	0.40	0.25	0.45	0.70	3.20	0.50	2.70	0.50	1.50	0.65
BI-05	0.65	0.55	0.45	1.00	4.10	0.60	7.55	3.75	2.80	0
BI-06	0.55	0.45	0.45	1.00	4.60	0.60	5.80	3.00	1.80	0

3.6.4 Wing walls

For BI-02 and BI-06 no wing wall will be considered due to the stilling basins associated to the bridges. Each bridge will be connected directly to the walls of the basins.

For BI-03 and BI-05 the wing walls will have a width of 0.35 m and supported by the same footing as the abutment. As the abutment is subjected to additional horizontal braking loads, a verification of stability is not needed for these wing walls.

4

BISHENYI IRRIGATION WATER INTAKES STRUCTURAL CALCULATIONS

4.1 Introduction

The following structural calculation relates to the design of the water intake structures at Bishenyi site.

The hydraulic modelling identified the hydraulic sections necessary for the structures. Details can be obtained in the hydraulic modelling report.

This document presents the calculations and verifications that were carried out in order to pre-dimension the associated canals and stilling basins.

4.2 General presentation of the existing structures

4.2.1 Location of the existing structures

The location of the structures (BI-01 and BI-04) is presented on drawing no. RW114-IR3-BI-001

Figure 23: BI-01 current state

Figure 24: BI-04 current state

4.3 Geometrical characteristics of the proposed structures

The characteristics of the hydraulic structures concerned by this calculation are presented in the following table. BI-001 is connected to the existing pipe culvert BI-01 which will not be redesigned, whereas BI-04 is connected to the existing double pipe culvert (BI-06) which has been redesigned as a bridge with rectangular section (see preceding section).

Table 31 Geometrical characteristics of the structures

ID	Structure Type	Width (m)
BI-001	Irrigation water intake	2.80
BI-04	Irrigation water intake	4.40

4.4 Design hypotheses

4.4.1 Reference documents

The design and calculation of the structures is carried out according to the prescriptions of Eurocodes, complemented by the following documents

- Open-Channel Hydraulics – CHOW Ven Te – New York – 1959
- Seismic hazard assessment of the Kivu rift segment based on a new seismotectonic zonation model (western branch, East African Rift system) – DELVAUX Damien, MULUMBA Jean-Luc, NTABWOBA STANISLAS SEBAGENZI Mwene, BONDO Silvanos Fiama, KERVYN François, HAVENITH Hans-Balder – Journal of African Earth Sciences – 2016

4.4.2 Material characteristics

The following material properties are taken into account for the design of the structures:

- Concrete
  - o Resistance: C25/30
  - o Weight: 2 403 kg/m3
- Masonry
  - o Weight 2 345 kg/m3
- Reinforcement: B500

Given the proximity of the structures to the rivers and the permeability of the soil, the groundwater level is assumed equal to the water level in the river.

4.4.3 Geotechnical data

Since a geotechnical study has not yet been conducted, the following characteristics are assumed based on the recommendations from RTDA Bridge Design Manual (§6.14).

Table 32 Geotechnical characteristics of the soil and foundation

Symbol	Characteristic	Value	Unit
	Backfill (sandy soil)		
?$_R$	Density	1922	kg/m3
?$_R$	Internal friction angle	30	degrees
K_a	Active pressure coefficient	0,296	
	Foundation (sandy soil)		
?$_s$	Internal friction angle	30	degrees
?$_s$	Friction angle between concrete and soil	30	degrees
q_a	Bearing capacity	0,240	MPa

4.4.4 Hydrological data

The following parameters were used for the hydraulic design of the structures:

Table 33 Hydrological parameters for the hydraulic design of the structures

Parameter	Unit	BI-01	BI-001	BI-04
Minimum discharge (Q_{min})	m^3/s	0.0315	0.0315	0.102
Discharge for 25-year return period (Q_{25})	m^3/s	8.88	8.88	18.60
Discharge for 100-year return period (Q_{100})	m^3/s	16.15	16.15	33.77

4.5 Structural design of the stilling basin downstream of pipe culvert BI-01

4.5.1 Stilling basin

The Peak Ground Acceleration A is taken as 0.18g for Bishenyi according to the reference "Seismic hazard assessment of the Kivu rift segment based on a new seismotectonic zonation model (western branch, East African Rift system)". The horizontal and vertical coefficients are therefore:

- kh = 0.5x 0.18 = 0.09 AASHTO 11.8.6
- kv = 0.5x 0.09 = 0.045

Hydraulic design

Figure 25: BI-01 current state at the exit of the existing pipe culvert

Bed and bank erosion occurs when the forces generated by the flow are greater than the stabilising forces of the stream. Also, in the case where water flows out of the culvert whilst the downstream canal is empty, the force of the water's contact with the channel bottom can damage the concrete.

In order to increase the resistance of the canal to the flow, the culvert should be is complemented by the design of a structure that allows both the formation of a water "mattress" and the dissipation of energy. This structure, also known as a stilling basin, is to located directly downstream of the culvert and will allow the flow to be brought to a lower level while dissipating the excess energy locally.

The constraints of the type of drop (vertical) and the design of a structure in a small river lead to the choice of a depressed basin.

The design of the basin includes determining the length and depth of the basin. The determination of these parameters requires the knowledge of the flow characteristics at the outlet of the culvert and those of the flow in the canal downstream of the basin.

The following table presents a summary of the different design parameters

Table 34 Design parameters for basin BI-01

Symbol	Explanation	Unit	Value
L	Drop width	m	3.00
Q	Discharge	m^3/s	16.15
q	Discharge per unit width	m^2/s	5.38
h	Drop height	m	2.50

Manning's formula is used to determine the characteristics of the flow in the canal downstream of the basin.

Table 35 Flow properties in the canal downstream of basin BI-01

Symbol	Explanation	Unit	Value
B_3	Canal width	m	3.00
S	Bed slope	m/m	0.010
n	Roughness coefficient	-	0.015
m	Canal wall slope	m/m	0.00
y_3	Water level	m	1.17

The flow geometry of the hydraulic jump is determined using the equations presented in the book "Open Channel Hydraulics". These equations describe the geometry as functions of the drop number, which is defined as:

$$D = \frac{q^3}{gh^3}$$

The properties of the hydraulic jump in the basin are thus the following:

Table 36 Properties of the hydraulic jump in basin BI-01

Symbol	Explanation	Unit	Value
L_d	Drop length	m	6.86
y_p	Pool depth under the nappe	m	1.73
y_1	Depth at the beginning of the hydraulic jump	m	0.67
y_2	Tailwater depth	m	2.65
L	Length of the hydraulic jump	m	15.88

The length thus recommended for the basin is given by the length of the drop and the length of the jump:

Lb = Ld + L = 6.86 + 15.88 = 23 m

The dimensions selected for the basin are as follows:

Table 37 Geometry of basin BI-01

Symbol	Explanation	Unit	Value
L_b	Basin length	m	23.00
D	Sill height	m	1.50
h_w	Basin wall height	m	2.50

4.5.2 Basin walls

The basin walls are gravity masonry walls. They must be designed to resist overturning, sliding and bearing pressure failure. The dimensions of the basin walls are given in Table 38.

Table 38 Basin BI-01 wall dimensions

Symbol	Element	Unit	Value
h_w	Height	m	2.50
$b_{w,top}$	Top width	m	0.30
$b_{w,bottom}$	Bottom width	m	0.80

4.6 Structural design of BI-01

4.6.1 Canal

Manning's formula is used to verify the properties of the canals and their corresponding theoretical and nominal flows.

$$Q = V \text{ x } S$$
$$V = K\ R^{2/3} I^{1/2}$$
(Manning Strickler formula)
$$S = Y\ u \text{ x} B$$
$$K = \frac{1}{n}$$
$$R = \frac{S}{\mathrm{P}}$$

Q : Flow rate in m^3/s
V : Flow velocity in m/s
S : Flow cross-section in m^2
n : Roughness coefficient
R : Hydraulic radius in m
P : Wetted perimeter in m
I : Bottom slope along the canal in m/m
Yu : Water level in m
B : Bed width in m

The roughness coefficient depends on the lining of the canal bed and walls: for a canal with a concrete bed and stone masonry walls, n is taken as 0.015.

In order that the velocity does not exceed 4 m/s, the slope of the canal should not exceed 1.0%. The below table gives the properties of the canal and the corresponding flows and velocities for the minimum and 25-year return period flows:

Table 39 Flow properties in the canal downstream of BI-01

Q	i	n	b	Yu	Rev	Htot	V
[l/s]	[m/m]	-	[m]	[m]	[m]	[m]	[m/s]
32	0.010	0.015	2.80	0.022	0.98	1.00	0.52
8880	0.010	0.015	2.80	0.762	0.248	1.00	4.16

4.6.2 Intake structure

Figure 26: BI-001 current state at the existing intake structure

The water level in the canal upstream of the structure must be maintained at 20 cm year-round in order to feed the irrigation channels.

Given that the hydraulic modelling did not show any choking of flow for the 100-year return period, it is proposed to raise the step on the structure by 0.20 m in order to maintain the water level.

4.6.3 Stilling basin

Hydraulic design

The following table presents a summary of the different design parameters

Table 40 Design parameters for basin BI-001

Symbol	Explanation	Unit	Value
L	Drop width	m	3.50
Q	Discharge	m^3/s	8.88
q	Discharge per unit width	m^2/s	2.54
h	Drop height	m	2.20

Manning's formula is used to determine the characteristics of the flow in the canal downstream of the basin.

Table 41 Flow properties in the canal downstream of basin BI-001

Symbol	Explanation	Unit	Value
B_3	Canal width	m	1.80
S	Bed slope	m/m	0.010
n	Roughness coefficient	-	0.033
m	Canal wall slope	m/m	0.67
y_3	Water level	m	1.35

The flow geometry of the hydraulic jump is determined using the equations presented in the book "Open Channel Hydraulics". These equations describe the geometry as functions of the drop number, which is defined as:

$$D = \frac{q^2}{gh^3}$$

The properties of the hydraulic jump in the basin are thus the following:

Table 42 Properties of the hydraulic jump in basin BI-001

Symbol	Explanation	Unit	Value
L_d	Drop length	m	4.46
y_p	Pool depth under the nappe	m	1.19
y_1	Depth at the beginning of the hydraulic jump	m	0.36
y_2	Tailwater depth	m	1.72
L	Length of the hydraulic jump	m	10.33

The length thus recommended for the basin is given by the length of the drop and the length of the jump:

Lb = Ld + L = 4.46 + 10.33 = 14.80 m

Table 43 Geometry of basin BI-001

Symbol	Explanation	Unit	Value
L_b	Basin length	m	15.00
D	Sill height	m	0.40
h_w	Basin wall height	m	2.50

4.6.4 Basin walls

The basin walls are gravity masonry walls. They must be designed to resist overturning, sliding and bearing pressure failure. The dimensions of the basin walls are given in

Table 44 Basin BI-001 wall dimensions

Symbol	Element	Unit	Value
h_w	Height	m	2.50
$b_{w,top}$	Top width	m	0.30
$b_{w,bottom}$	Bottom width	m	0.80

4.6.5 Sheet piles

When small structures are constructed on a loose foundation, the risk of piping mut be prevented. It is prevented by providing sheet piles on the buried face of sufficient depth to lengthen the flow path.

Lane's formula is used to verify the security of the structure with regards to piping.

$$L_v = + \frac{1}{3} L_n \geq C\ H$$

with
L_v the length of vertical flow paths
L_h the length of horizontal flow paths
H the water level upstream of the structure
The coefficient C depends on the soil: for fine silts and sands, C is taken as 8.5

In the case of structure BI-001

$$3.10 + \frac{1}{3} \times 15.00 = 8.1 \geq 7 \times 0.76 = 5.3$$

Thus, the stability of the structure is ensured without the need for sheet piles.

4.7 Structural design of BI-04

4.7.1 Canal

Manning's formula is used to verify the properties of the canals and their corresponding theoretical and nominal flows.

$$Q = V \text{ x } S$$

$$V = K\ R^{2/3} I^{1/2}$$

(Manning Strickler formula)

$$S = Y\ u \text{ x} B$$

$$K = \frac{1}{n}$$

$$R = \frac{S}{\text{P}}$$

Q	: Flow rate in m^3/s
V	: Flow velocity in m/s
S	: Flow cross-section in m^2
n	: Roughness coefficient
R	: Hydraulic radius in m
P	: Wetted perimeter in m
I	: Bottom slope along the canal in m/m
Yu	: Water level in m
B	: Bed width in m

The roughness coefficient depends on the lining of the canal bed and walls: for a canal with a concrete bed and stone masonry walls, n is taken as 0.015.

In order that the velocity does not exceed 4 m/s, the slope of the canal should not exceed 0.5%.

The below tables give the properties of the canal and the corresponding flows and velocities for the minimum and 25-year return period discharges:

Table 45 Flow properties in the canal upstream of BI-04 – Qmin

MP		Q	i	n	b	Yu	Rev	Htot	V
Start	End	[l/s]	[m/m]	-	[m]	[m]	[m]	[m]	[m/s]
0+000	0+005	102	0.005	0.015	6.62	0.03	1.24	2.00	0.48
0+005	0+010	102	0.005	0.015	5.54	0.04	1.61	2.46	0.51
0+010	0+015	102	0.005	0.015	5.69	0.04	1.66	2.50	0.50
0+015	0+020	102	0.005	0.015	3.80	0.05	1.51	2.50	0.59
0+020	0+030	102	0.005	0.015	2.45	0.06	0.28	1.60	0.67
0+030	0+035	102	0.005	0.015	2.18	0.07	0.70	2.00	0.69
0+035	0+040	102	0.005	0.015	2.03	0.07	1.05	2.25	0.70
0+040	0+045	102	0.005	0.015	2.26	0.06	1.25	2.42	0.71
0+045	0+050	102	0.005	0.015	1.47	0.08	0.37	1.90	0.82

Table 46 Flow properties in the canal upstream of BI-04 – Q25

MP		Q	i	n	b	Yu	Rev	Htot	V
Start	End	[l/s]	[m/m]	-	[m]	[m]	[m]	[m]	[m/s]
0+000	0+005	18 600	0.005	0.015	6.62	0.76	1.97	2.00	3.50
0+005	0+010	18 600	0.005	0.015	5.54	0.85	2.42	2.46	3.65
0+010	0+015	18 600	0.005	0.015	5.69	0.84	2.46	2.50	3.63
0+015	0+020	18 600	0.005	0.015	3.80	0.99	2.45	2.50	3.77
0+020	0+030	18 600	0.005	0.015	2.45	1.32	1.54	1.60	3.82
0+030	0+035	18 600	0.005	0.015	2.18	1.30	1.93	2.00	3.74
0+035	0+040	18 600	0.005	0.015	2.03	1.20	2.18	2.25	3.55
0+040	0+045	18 600	0.005	0.015	2.26	1.17	2.36	2.42	3.79
0+045	0+050	18 600	0.005	0.015	1.47	1.53	1.82	1.90	4.04

4.7.2 Intake structure

Figure 27: BI-04 current state

The water level in the canal upstream of the intake structure must be maintained at 1.60 m year-round in order to feed the irrigation channels. Following the redesign of the intake structure, the depth of water for the minimum flow is only 7 cm.

The redesign of the intake structure must therefore be completed by the installation of water level control gates (for example, sluice gates or flap gates) in order to maintain the minimum water level. Depending on the model chosen, this allows either manual regulation, or a constant upstream level regulated automatically (by means of a float sector valve balanced by a counterweight).

4.7.3 Stilling basin

Hydraulic design

The following table presents a summary of the different design parameters

Table 47 Design parameters for basin BI-04

Symbol	Explanation	Unit	Value
L	Drop width	m	6.50
Q	Discharge	m^3/s	18.60
q	Discharge per unit width	m^2/s	2.86
h	Drop height	m	2.50

Manning's formula is used to determine the characteristics of the flow in the canal downstream of the basin.

Table 48 Flow properties in the canal downstream of basin BI-04

Symbol	Explanation	Unit	Value
B_3	Canal width	m	6.50
S	Bed slope	m/m	0.015
n	Roughness coefficient	-	0.033
m	Canal wall slope	m/m	0.42
y_3	Water level	m	0.90

The flow geometry of the hydraulic jump is determined using the equations presented in the book "Open Channel Hydraulics". These equations describe the geometry as functions of the drop number, which is defined as:

$$D = \frac{q^2}{g\ \mathrm{H}^3}$$

The properties of the hydraulic jump in the basin are thus the following:

Table 49 Properties of the hydraulic jump in basin BI-001

Symbol	Explanation	Unit	Value
L_d	Drop length	m	5.25
y_p	Pool depth under the nappe	m	1.50
y_1	Depth at the beginning of the hydraulic jump	m	0.35
y_2	Tailwater depth	m	2.03
L	Length of the hydraulic jump	m	12.16

The length thus recommended for the basin is given by the length of the drop and the length of the jump:

Lb = Ld + L = 5.25 + 12.16 = 17.50 m

The dimensions selected for the basin are as follows:

Table 50 Geometry of basin BI-04

Symbol	Explanation	Unit	Value
L_b	Basin length	m	17.50
D	Sill height	m	1.20
h_w	Basin wall height	m	3.50

Basin walls

The basin walls are cantilever reinforced concrete walls. They must be designed to resist overturning, sliding and bearing pressure failure. The dimensions of the basin walls are given in Table 51.

Table 51 Basin BI-001 wall dimensions

Symbol	Element	Unit	Value
	Wall		
h_w	Height	m	3.50
b_w	Width	m	0.30
	Base footing		
B	Length	m	
D	Thickness	m	0.30

4.7.4 Sheet piles

Lane's formula is used to verify the security of the structure with regards to piping.

$$L_V + \frac{1}{3} L_H \geq C\ H$$

with

Lv the length of vertical flow paths

Lh the length of horizontal flow paths

H the water level upstream of the structure

The coefficient C depends on the soil: for fine sands, C is taken as 7

In the case of structure BI-04

$$4.90 + \frac{1}{3} \times 17.50 = 10.7 \geq 7 \times 1.53 = 10.7$$

Thus, the stability of the structure is ensured without the need for sheet piles.

5

RWANDEX-MAGERWA STRUCTURAL CALCULATIONS

5.1 Introduction

Five hydraulic structures in the Rwandex-Magerwa site are the object of design:

1. Adaptation of RM-01 culvert on KK 567 St.;
2. Design of RM-02 culvert on KK 46 Ave.;
3. Redesign of RM-07 culvert on KK 6 Ave.;
4. New pedestrian culvert between RM-01 and RM-02 (after KK 628 St.);
5. Drainage channel between RM-01 and RM-02.

We adopt concrete box culverts throughout since the spans are smaller than 6m and do not call for complex bridges. We adopt rectangular section for the culverts for economy of space and current common practice.

The width of the road and hence the width of the culvert is based on the width of the road/street not per status quo but as projected in Kigali master plan 2050.

5.2 General hypotheses

5.2.1 Reference documents

Rwanda Transport Development Agency, Drainage Manual, 2014

The U.S. Department of Transportation, Hydraulic design of highway culverts, 2012

AASHTO Drainage Manual Volume 1, 2014.

5.2.2 Hydraulic design

The design of the structures is based on the Manning formula, expressed as:

$$Q = V \text{ x } S$$

$$V = K\ R^{2/3} I^{1/2}$$

(Manning Strickler formula)

$$S = Yu \text{ x} B$$

$$K = \frac{1}{n}$$

$$R = \frac{S}{\text{P}}$$

Q	: Flow rate in m^3/s
V	: Flow velocity in m/s
S	: Flow cross-section in m^2
n	: Roughness coefficient
R	: Hydraulic radius in m
P	: Wetted perimeter in m
I	: Bottom slope along the canal in m/m
Yu	: Water level in m
B	: Bed width in m

The design has been run through HY-8 Culvert Hydraulic Analysis Program, produced by the U.S. Department of Transportation. The maximum depths of flow have been systematically defined to have minimum freeboard of 15cm. The maximum flow velocity has been limited to 2 m/s. For existing structures, this has been checked to not exceed 4.5 m/s as per RTDA guidelines.

The longitudinal slopes have been fixed at 0.5% for the culverts. For the drain, the natural average slope of 1.5% has been followed. 50 cm maximum drops in elevation have been provided where the slope was above the one specified.

Below is a recapitulative table of the dimensions of the calculated structures:

Table 52 Recapitulative table of dimensions of the structures

Structure	Q [m^3/s]	Existing internal dimensions (approx.)			New internal dimensions			Comment
		Base [m]	Height [m]	Width [m]	Base [m]	Height [m]	Width [m]	
RM-01	10.59	2.2	1.9	5.4	2.2	1.9	15	Only extend the culvert width as per the master plan
RM-02	11.02	2.3	1.5	5.0	3.0	2.0	6.0	New RC culvert to build in lieu of an old wooden bridge
RM-07	7.58	1.6	1.8	15	2.0	2.0	30	Change alignment to break perpendicularity
RM-01/02	11.02	-	-	-	3.0	2.0	1.5	New structure to allow pedestrians to cross channel DR-01/02
DR-01/02	11.02	-	-	-	2.0	2.0	300	Natural drain to line for safety and to avoid further gullying

Culverts are in reinforced concrete with 25 cm wall thickness. Angles are chamfered at 45� for more structural support.

The drainage channel is in stone masonry.

The location of the structures is presented on drawing no. RW114-IR3-RM-001.

5.3 Description of the structures

5.3.1 RM-01

RM-01 existing RC culvert crosses street no. KK 567 St. in Rebero village, Karambo cell, Gatenga sector of Kicukiro district. It consists of an existing RC culvert spanning 5m of the current road width.

Figure 28 RM-01 existing culvert

The existing culvert with 2.2m base and 1.9m height has been found to successfully evacuate the design discharge. However, the width of the road according the Kigali City masterplan will have to increase to 15 m, and therefore the culvert is adapted accordingly.

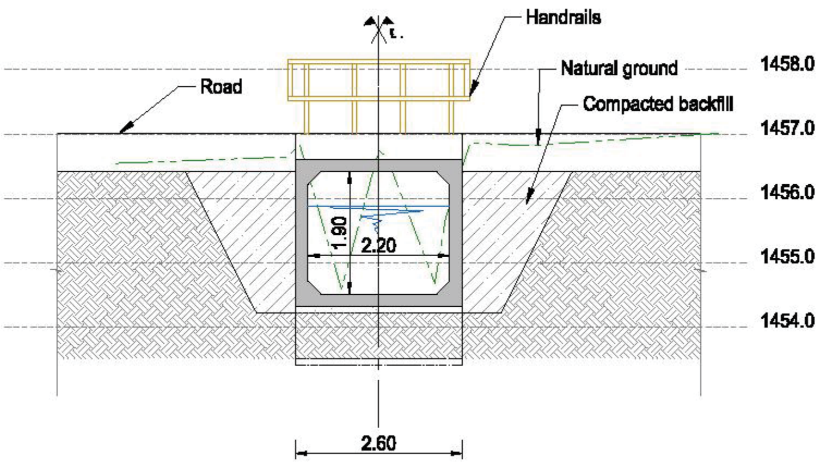


Figure 29 RM-01 proposed culvert in cross-section

5.3.2 RM-02

RM-02 existing RC culvert crosses street no. KK 46 Ave. in Gwiza village, Karambo cell, Gatenga sector of Kicukiro district. The existing structure is a rudimentary wooden bridge at an angle of approximately 60^0 with the drainage channel.

Figure 30 RM-02 existing wooden bridge

The proposed structure will be a RC box culvert and will keep the same angle of approximately 60^0 with the road.

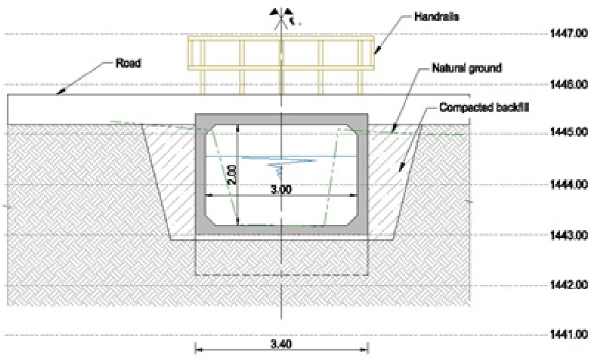


Figure 31 RM-02 proposed RC culvert in cross-section

5.3.3 RM-07

RM-07 existing RC culvert crosses street no. KK 6 Ave. in Marembo I village, Kanserege cell, Gikondo sector of Kicukiro district near the entrance to NAEB. The existing stone masonry drain covered with a RC slab is under-sized to evacuate the design discharge. Moreover, the alignment makes a perpendicular angle to cross the road, which causes an obstruction in the flow and creates considerable inundations around the structure.

Figure 32 RM-07 existing culvert

The proposed structure is a RC culvert at an average angle of approximately 150° with the upstream drainage channel and 170° with the downstream channel crossing the road KN 6 Ave.

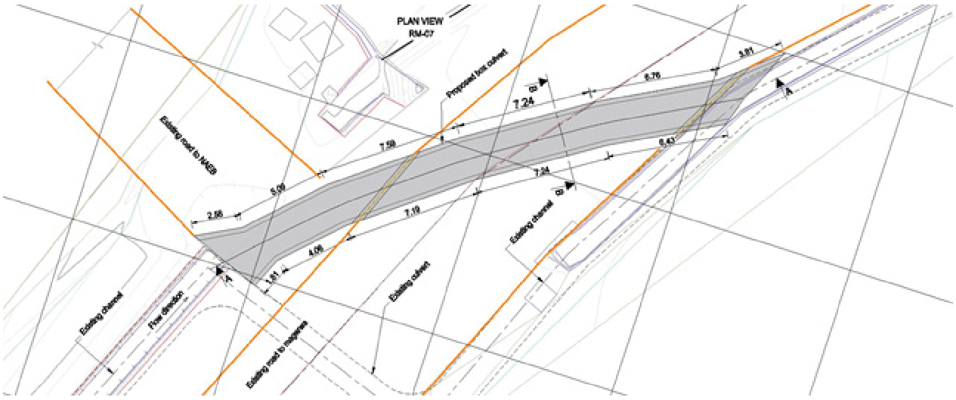


Figure 33 RM-07 corrected alignment

5.3.4 RM-01/02

The new proposed culvert between RM-01 and RM-02 will serve as a pedestrian crossing point over the proposed new masonry drainage channel (see Section 5.3.5). The current crossing consists of simple logs and planks, and the pedestrian traffic in that section of the street in continuation of KK 628 St. seems rather significant.

Figure 34 RM-07 Existing pedestrian crossing

5.3.5 DR-01/02

The 300m long drainage channel that runs from culvert RM-01 to culvert RM-02 is proposed to be lined with stone masonry.

The natural channel has experienced extensive gullying over time especially in the first 100 m. It poses a hazard to the neighboring houses, noting that accidents and deaths have been reported at deep sections of the channel. The proposed masonry channel will be rock filled at the bottom to reach the required elevation, and will be equipped with handrails for safety.

A series of 50 cm drops are proposed along the profile to maintain a consistent channel slope, and thereby reduce flow velocities, maintaining them within the design limits.

Figure 35 Location of proposed stone masonry channel and pedestrian crossing RM-01/02

5.4 Structural design of culverts

The RC culvert structure is structurally considered as a retaining wall box, with fixity on all junctions, and with water retention inside. The worst load conditions are when it is empty, so water pressure and earth pressure don't coexist.
Seismic action is considered conservatively at this stage, assuming entire mass of concrete and water is activated, with the peak ground acceleration "pga", as the structure is buried.

The critical section with maximum width and height is considered for detailed structural calculation.

5.4.1 Design assumptions

Concrete Grade:	25 N/mm^2
High yielding strength deformed bars	B 500
Slab bed rammed and leveled with ~75mm layer of lean concrete or similar	
Design Traffic LL	145 kN
Design Lane LL	9.3 kN/m
Min geometrical % reinforcement, per Ac element ρ	0.3 %
Soil friction angle	30 ^{0}
Granular backfill angle	30 ^{0}
Unit weight of wet soil	19.22 kN/m^3
Ka	0.296
Linear Pressure acting on bottom slab from sandy soil w*h*Ka	15.9 kN/m^2
Linear Pressure acting from water gw*h	24 kN/m^2
Constant Pressure acting from granular backfill soil, p w*h*Ka	15.9 kN/m^2
Constant Vertical line load from surcharge LL F/L	57.6 kN/m
Constant Horizontal line load from surcharge LL	77.2 kN/m
h	2.8 m
b	3.4 m
Net pressure, full water	8.1 kN/m^2

5.4.2 Design of wall

1 m run of wall is considered, and the critical condition is when the culvert is empty of water, and only soil pressure is applied.

Triangular soil load

Mmax for vertical reinforcement	
Mmax at top (LL, Seismic, Soil) ph^2/30	30.6 kNm
Mmax at bot (LL, Seismic, Soil) ph^2/20	31.7 kNm
Vmax ph/2	86.7 kN
Deflection max, immediate ph^4/764EI	0.94 mm

Triangular soil load, no water in basin	
Ec	30,000 Mpa
C25/30 assumed	
Moment of inertia I for 1m run bh^3/12	6.67x108 mm4

Initial dimensions:	
Top slab thickness	200 mm
Wall thickness	200 mm
Bottom slab thickness	200 mm

5.4.3 Verification of wall thickness

Stress ct, permissible tensile for C25/30 due to bending	1.8 Mpa
Mmax = Stressct*b*D^2/6, solve for Dmin	325 mm
D	200 mm
Cover d'	35 mm
Effective depth, deff	165 mm
L	15 m
h	2.0 m
b	3.0 m
Ratio for long wall L/h	10.0 > 2
Ratio for short wall b/h	1.50 <2

5.4.4 Vertical walls

Mmax, soil, outer	31.7 kNm
Outer face Ast, Mmax/Stress,st*gc*d	903 mm^2
Spacing s, (ast/Ast)*1000	120 mm
Bar diameter	12 mm

Provide vertical outer face reinforcement 10mm dia at S mm c/c horizontal spacing

Mmax, water, inner	3.0 kNm
Mmax, inner - Mmax, outer	28.7 kNm
Inner face Ast, Mmax/Stress, st*gc*d	817 mm^2
Spacing s, (ast/Ast)*1000	130 mm
Bar diameter	12 mm

Provide vertical inner face reinforcement 10mm dia. at S mm c/c horizontal spacing

Horizontal, secondary

Min r=0.3% 600 mm^2	600 mm^2
Spacing s, (ast/Ast)*1000 180 mm	180 mm
Bar diameter12 mm	12 mm

Provide vertical inner face reinforcement 16mm dia. at S mm c/c horizontal spacing

Same for both faces

Concrete grade	C25/30
Permissible design values, water retaining, cracking critical	
Tension due to bending (concrete)	1.8 N/mm^2
Compression due to bending (concrete)	7 N/mm^2
Steel reinforcement for strength	250 N/mm^2
Uplift 1.35*Gw*H	0 kPa

5.4.5 Bottom slab

Assumed fixed supports boundary conditions from long walls

Mmax support ph^2/12	13.6 kNm
Mmax span ph^2/24	6.8 kNm
Effective span, clear span + h/2 + h/2	3.2 m
Ast, Mmax/Stress,st*gc*d	388 mm^2
Spacing s, (ast/Ast)*1000	290 mm
Bar diameter	12 mm

Both faces

Horizontal reinforcement 3%	600 mm^2
Spacing s, (ast/Ast)*1000	130 mm
Bar diameter	10 mm

5.4.6 Top slab

Ratio Ly/Lx	5.0 >2
Ly	15 m
Lx	3 m
M ql^2/24	27.59 kNm
M ql^2/12	55.17 kNm
Ast, Mmax/Stress,st*gc*d	1623 mm^2
Spacing s, (ast/Ast)*1000	120 mm
Bar diameter	16 mm
Secondary distribution bar 0.3%	600 mm^2
Spacing s, (ast/Ast)*1000	130 mm
Bar diameter	10 mm
Crack width targeted for water retaining long term structure	0.2 mm

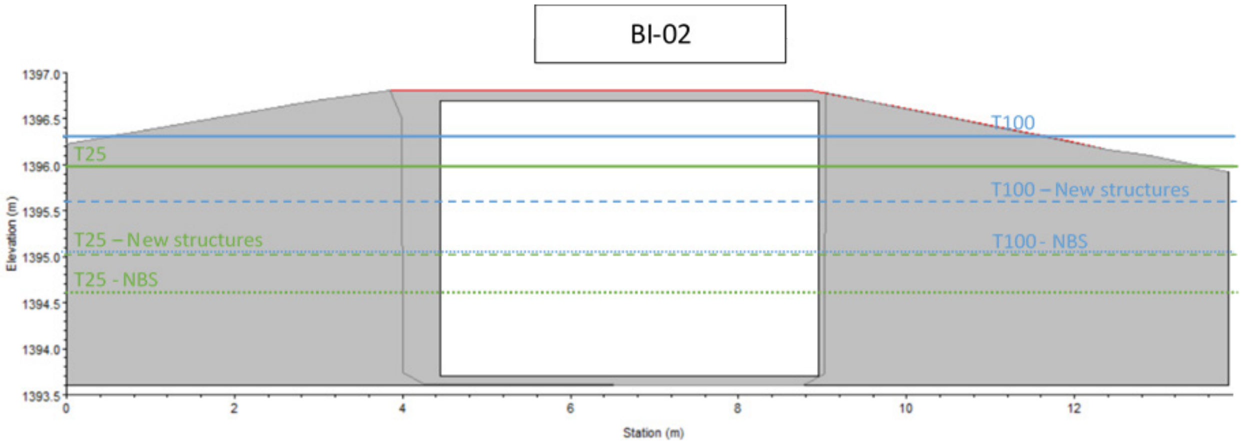
Effort to be less

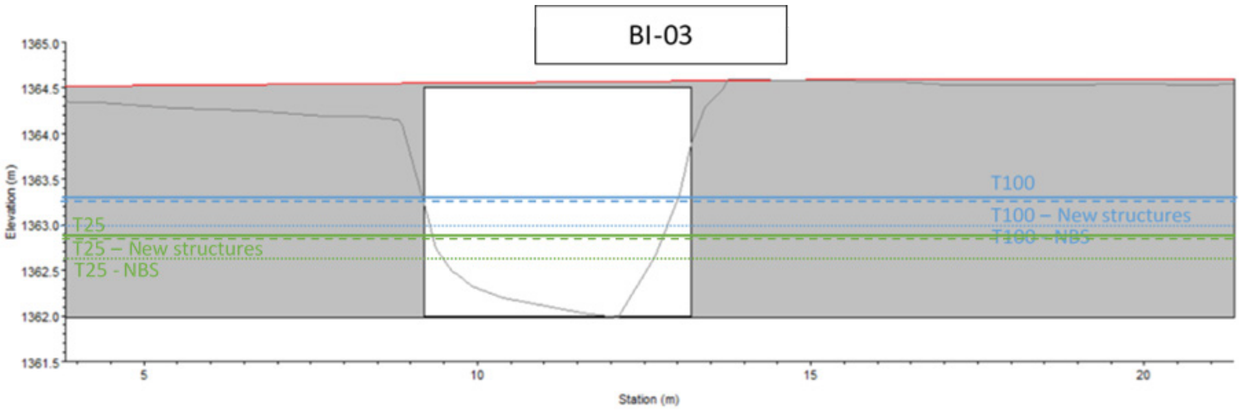
Approach, limit tensile concrete stress, limit rebar stress, close spacing, low diameter bars

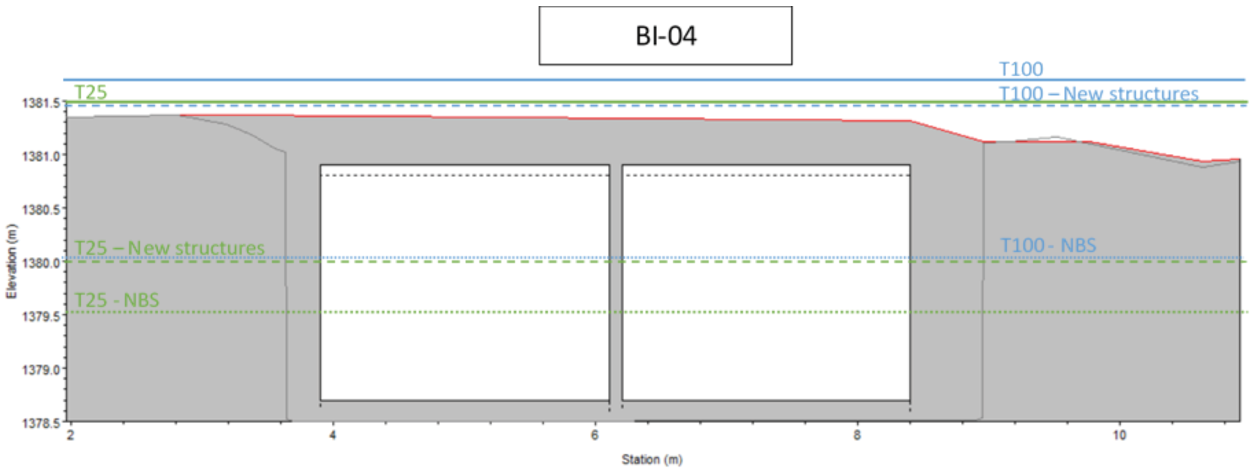
Final proposed dimensions: the initial dimensions are OK but on the limit side.

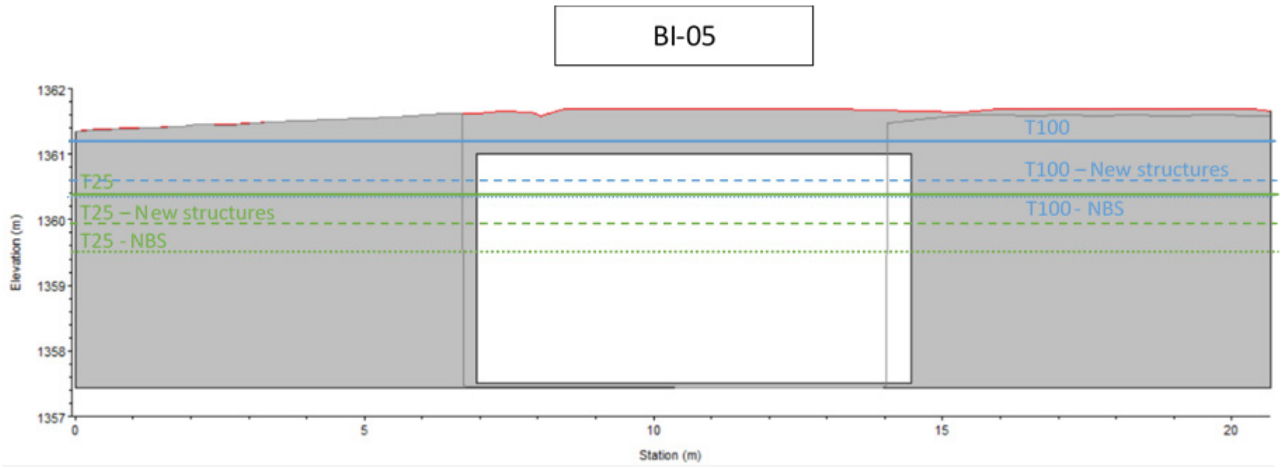
Top slab thickness	250 mm
Wall thickness	250 mm
Bottom slab thickness	200 mm

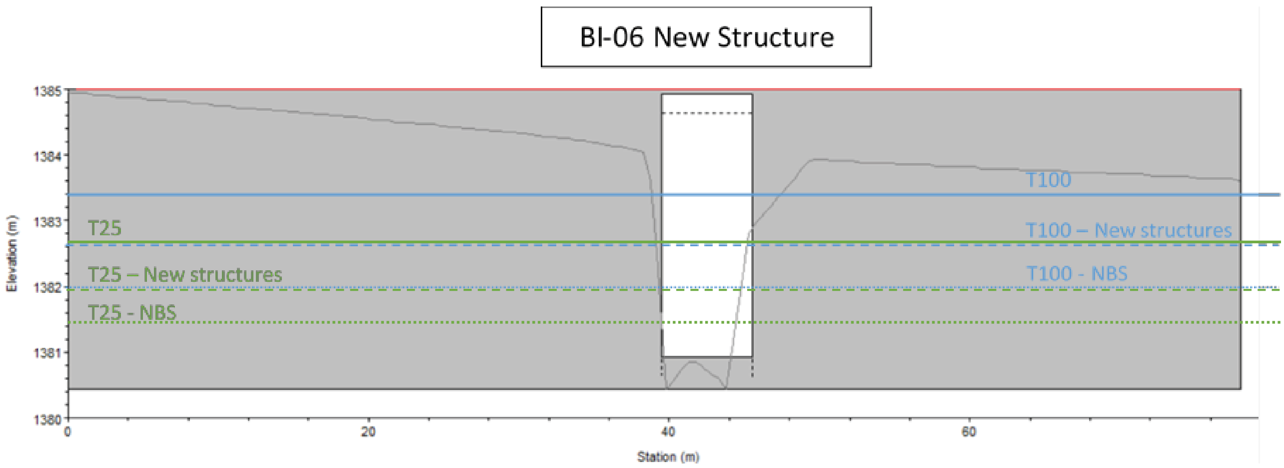
APPENDIX – HEC-RAS cross section of structures with indication of water levelss

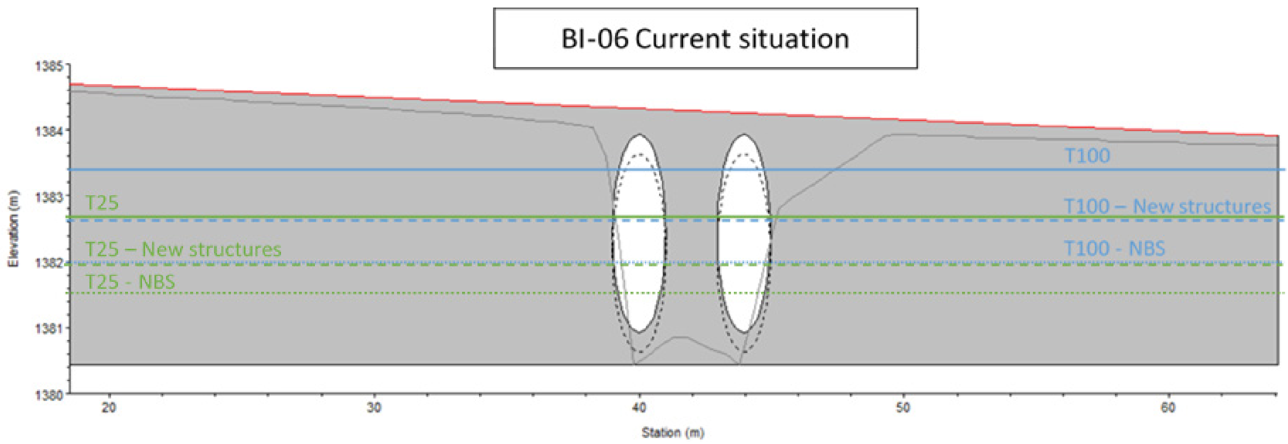


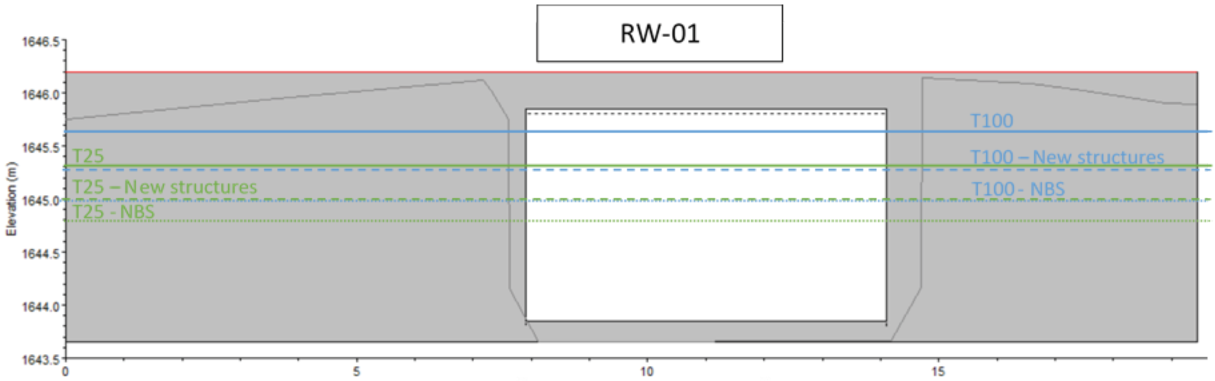


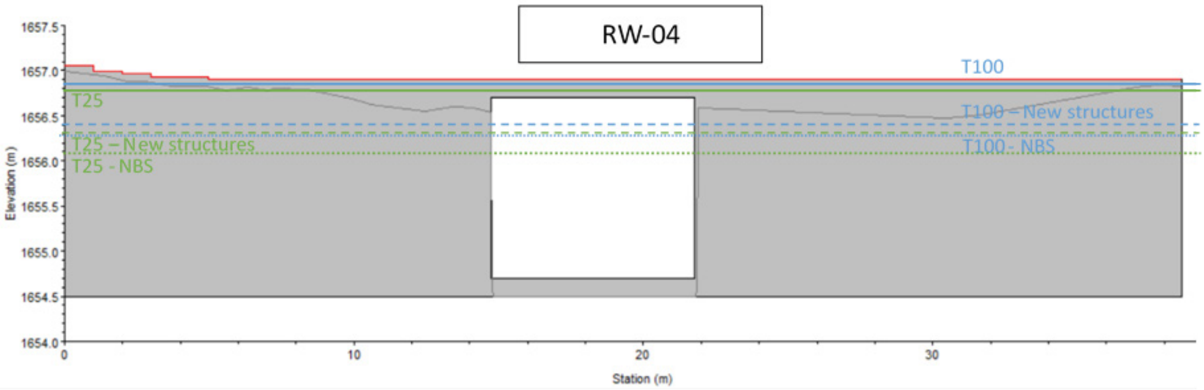


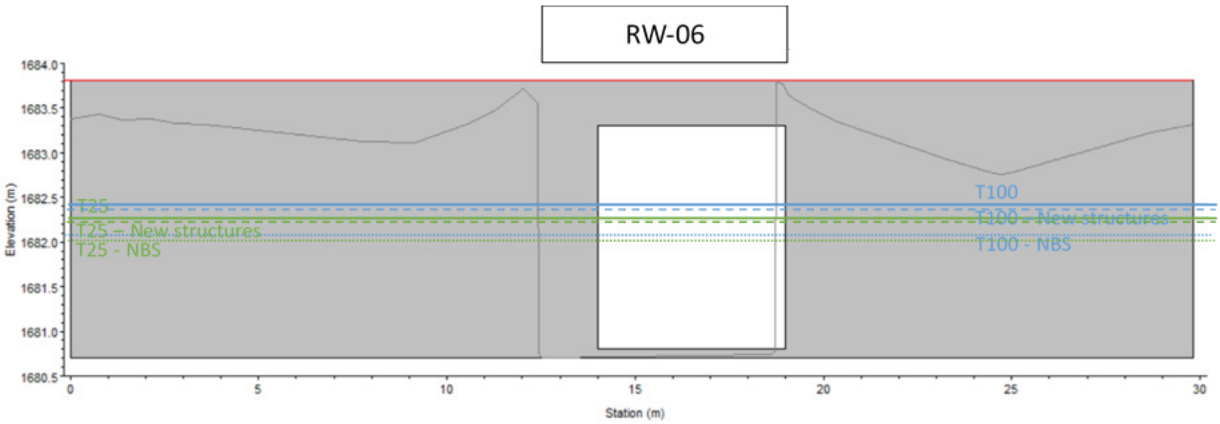

BI-05
1362
1361
1360
1359
1358
1357
Elevation (m)
0
5
10
15
20
Station (m)
T100
T100 – New structures
T100 - NBS
T25
T25 – New structures
T25 - NBS

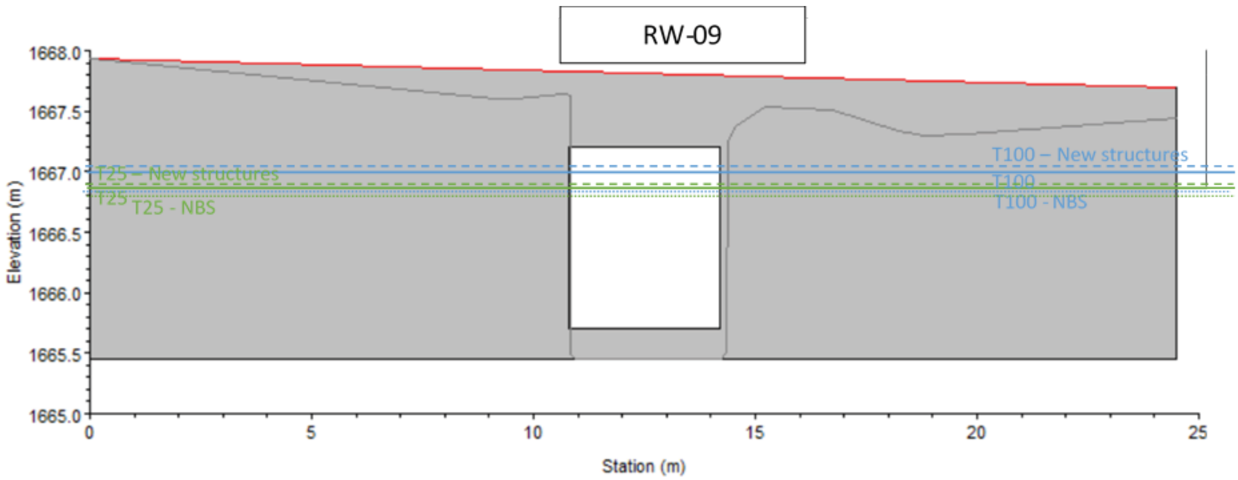

BI-06 New Structure
1385
1384
1383
1382
1381
1380
Elevation (m)
0
20
40
60
Station (m)
T100
T100 – New structures
T100 - NBS
T25
T25 – New structures
T25 - NBS

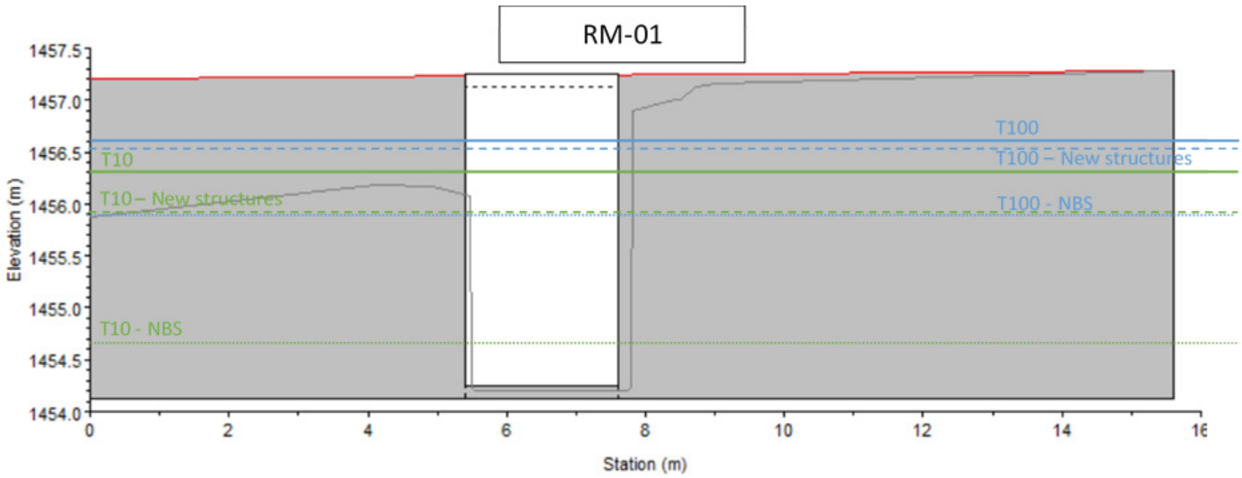

BI-06 Current situation
1385
1384
1383
1382
1381
1380
Elevation (m)
20
30
40
50
60
Station (m)
T100
T100 – New structures
T100 - NBS
T25
T25 – New structures
T25 - NBS

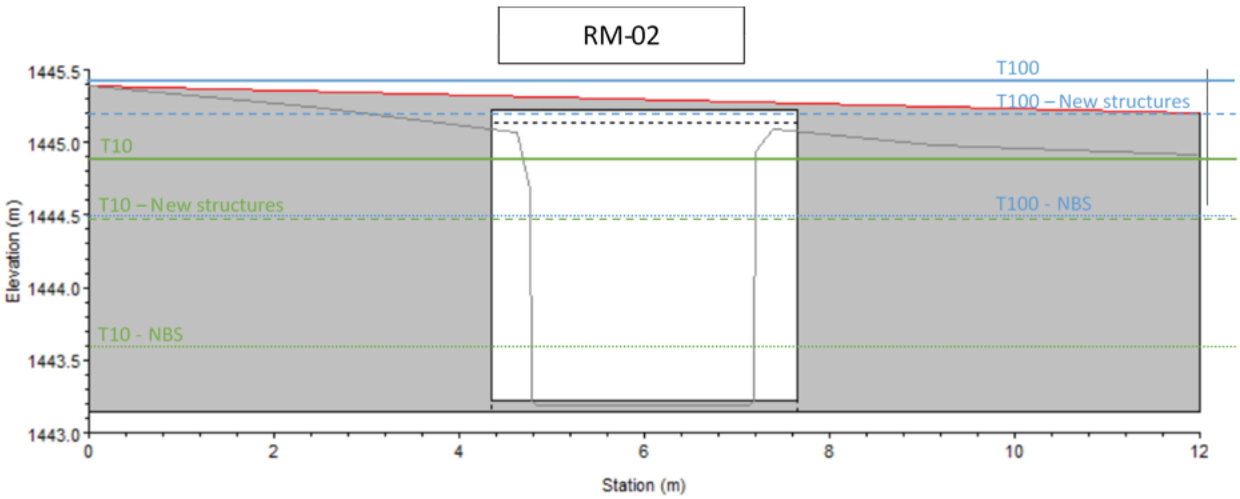

RW-01
Elevation (m)
T25
T25 – New structures
T25 - NBS
T100
T100 – New structures
T100 - NBS

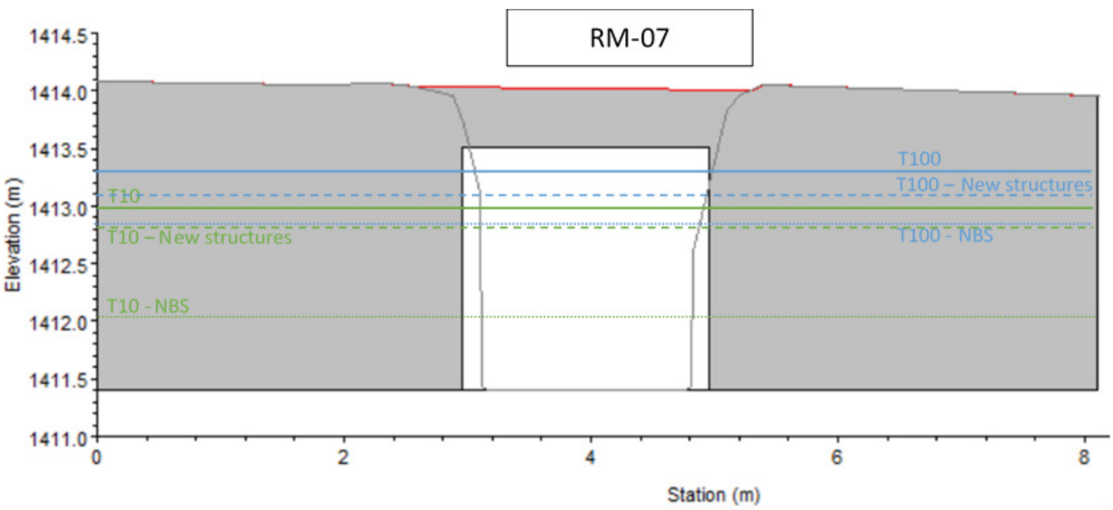

RW-04
Elevation (m)
T25
T25 – New structures
T25 - NBS
T100
T100 – New structures
T100 - NBS
Station (m)


RW-06
Elevation (m)
T25
T25 – New structures
T25 - NBS
T100
T100 – New structures
T100 - NBS
Station (m)


RW-09
Elevation (m)
T25 – New structures
T25
T25 - NBS
T100 – New structures
T100
T100 - NBS
Station (m)


RM-01
1457.5
1457.0
1456.5
1456.0
1455.5
1455.0
1454.5
1454.0
Elevation (m)
T10
T10 – New structures
T10 - NBS
T100
T100 – New structures
T100 - NBS
0
2
4
6
8
10
12
14
16
Station (m)


RM-02
1445.5
1445.0
1444.5
1444.0
1443.5
1443.0
Elevation (m)
T10
T10 – New structures
T10 - NBS
T100
T100 – New structures
T100 - NBS
0
2
4
6
8
10
12
Station (m)


RM-07
1414.5
1414.0
1413.5
1413.0
1412.5
1412.0
1411.5
1411.0
Elevation (m)
T10
T10 – New structures
T10 - NBS
T100
T100 – New structures
T100 - NBS
0
2
4
6
8
Station (m)

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