Hydraulical Feasibility Study of Margat Railway Bridge over Arroyo Canelón Grande

September 2017

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September, 2017.

Acronyms

DINASA	National Water and Sanitation Directorate
DINAGUA	National Water Directorate
NRCS	Natural Resources Conservation Service
SCS	Soil Conservation Service
Tr	Return period
IDF	Intensity – duration - frequency
CN	Curve Number
IMFIA	Institute of Fluid Mechanics and Environmental Engineering

1. Introduction

The final objective of this study is to determine the feasibility of the new design for the Margat Bridge, located in the Canelón Grande River, Canelones department, Uruguay.

For this, hydrological and hydraulic studies were held. The hydrological study was done with the NRCS method and includes the characterization of the catchment, the computation of its time of concentration, effective rainfall and finally the design hydrograph. This study was held for 100 year return period.

The hydraulic study was done with the HEC-RAS software from the US Corps of Engineers. The model simulates the behavior of the Canelón Grande River from approximately the Route 62 until its mouth in the Santa Lucía River. For the current and future designs of the bridge two cases have been evaluated. The first one represents the worst scenario for the head loss and velocity magnitude in the bridge. The second one considers the worst scenario regarding water levels. Both depends on the downstream boundary condition levels in Santa Lucia River.

2. Hydrologic study

2.1. Catchment delimitation

The relevant catchment for this study is the one that defines the flow at the Margat Bridge. For this reason, a single catchment was considered, with closing point at that bridge. The coordinates of this point are 560967.06 m E and 6183816.27 m S in UTM84-21S coordinate system.

The following figure and table present the catchment delimitation and its characteristics.

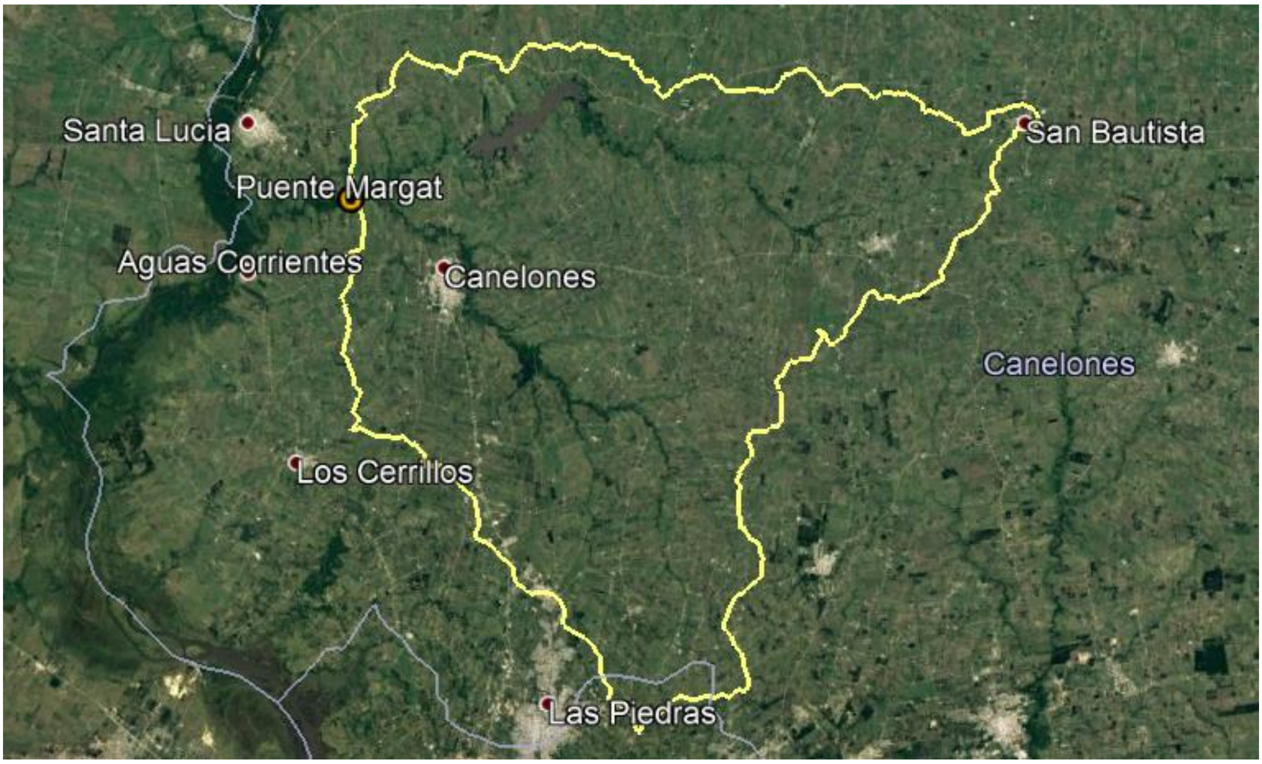

Figure 2–1 Catchment delimitation

Area (km^2)	Main length (km)	ΔH (m)	Slope (m/m)
688	58.7	75	0.0013

Table 2–1 Physical characteristics of the catchment

2.2. Input flow

The method of the Natural Resources Conservation Service (NRCS)[1] of the United States was used to define the input flow to the catchment. This method calculates the runoff for extreme events, given the precipitation, soil characteristics and catchment cover. In addition, it proposes the use of a Triangular Unit Hydrograph to estimate the maximum flow and its associated hydrograph, from the effective rainfall.

The method consists of three stages:

- Synthetic Storm (Alternating Block Method).
- Effective rainfall (SCS Curve Number Method)
- Unit Hydrograph (SCS triangular hydrograph).

2.2.1. Design Storm

The storm was built for 100-year-return period and it was constructed using the Alternating Block Method, recommended in Chapter 7.3.3 of the Urban Storm Water Design Manual of the National Water and Sanitation Directorate (DINASA[2], for its name in Spanish). For the construction of this hypothetical storm, the available information of intensity-duration-frequency curves presented in Chapter 7.3.2 of DINASA's manual was used.

In the Alternating Block Method, rainfall intensity is divided into time intervals, where rainfall intensity remains constant. To determine the size of each interval, first the time of concentration of the catchment was computed using the Kirpich equation:

$$t_c = 0{,}066 \times \frac{L^{0{,}77}}{S^{0{,}385}} = 19.9 \text{ hours}$$

where,

t_c: is the time of concentration in hours

L: is the hydraulic length of the catchment (km), and corresponds to the largest flow path (see Table 2-1)

S: is the average slope of the longest hydraulic path (see Table 2-1)

Once the concentration time of the catchment was calculated, the interval of the Alternating Block Method was selected. The chosen value was 80 minutes.

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[1] Formerly known as the Soil Conservation Service (SCS)

[2] Formerly known as National Water Directorate (DINAGUA, for its name in Spanish)

The number of blocks used to create the design storm was such that they cover at least twice the estimated time of concentration. We considered 30 blocks of 80 minutes each.

The precipitation intensity is the average rainfall rate, usually expressed in millimeters per unit of time. The value assumed is closely linked to the return period of the storm (Tr) and the duration of the rainfall. The intensity – duration – frequency curves (IDF) and the Montana Law were used for the computation of the rainfall intensity. According to the Montana Law:

$$i = a \times t^b$$

where,

i: is the rainfall intensity in mm/h

t: is the duration of the storm in hours

a and b: are coefficients that depend of the duration and return period of the storm; they can be calculated using the following expressions:

- If the duration is smaller than 3.5 hours:

  $$a = P(3,10,p) \times (0{,}1241 \times ln\,(Tr)\; + 0{,}317)$$

  $$b = -0{,}547$$

- If the duration is bigger than 3.5 hours:

  $$a = P(3,10,p) \times (0{,}1567 \times ln\,(Tr)\; + 0{,}4017)$$

  $$b = -0{,}725$$

where,

Tr: is the return peridod in years

$P(3,10,p)$: is the height, in mm, of precipitation for a storm with duration of 3 hours and 10 years of return period. It is obtained from the map of isohyets of extreme rainfall in Uruguay. For this catchment location it takes a value of 80 mm.

The following table presents the coefficients a and b for this study.

Duration	$\boldsymbol{a}$	$\boldsymbol{b}$
Less than 3.5 hours	71.080	-0.547
More than 3.5 hours	89.866	-0.725

Table 2–2 Coefficients a and b to compute rainfall intensity

30 blocks of 80 minutes each were used to make the design storm, with the peak in the center of the hyetograph. The precipitation associated to each block was calculated with the Alternating Block Method. In this way, it is always assumed that the maximum precipitation in any period multiple of 80 minutes corresponds to the equation presented above.

The following figure presents the design storm for 100 year return period.

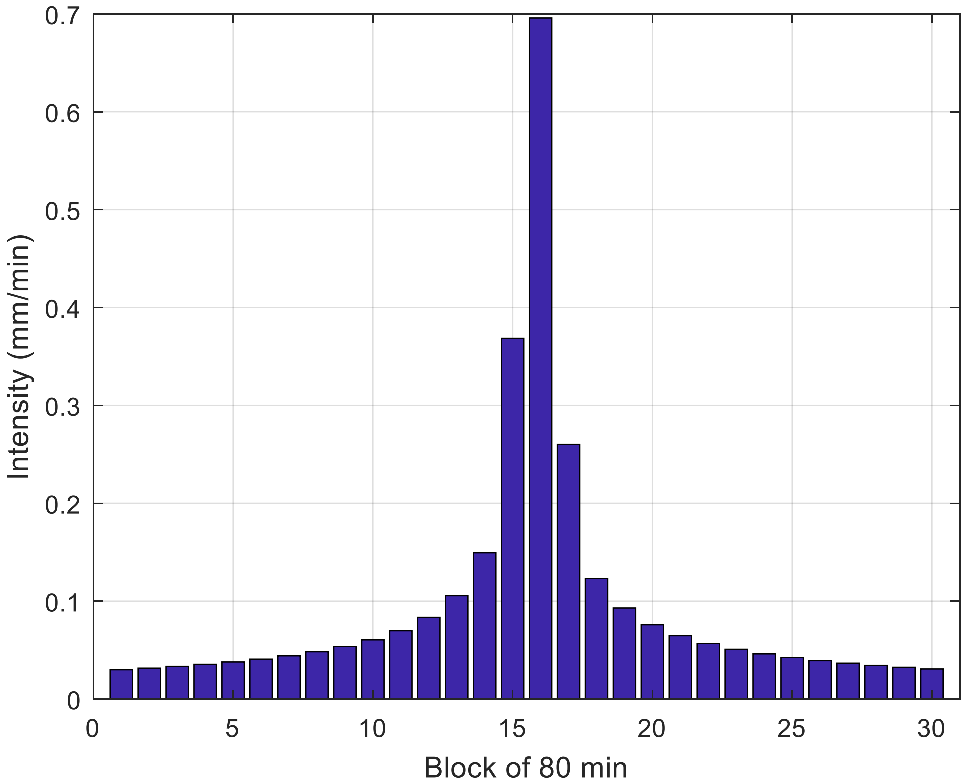


Figure 2–2 Synthetic design storm for 100 year return period

2.2.2. Effective Rainfall

The effective rainfall is the part of the total rainfall that falls on a given area that generates direct runoff. It is computed from the design storm, already determined in the previous item, and the soil unit.

The effective rainfall is calculated for each interval of the design storm presented in item 2.2.1. From the cumulative volume of the storm, the runoff was calculated using the Curve Number Method (hereinafter CN), following the equations shown below.

- If P < 0,2 S

$$P_e = 0$$

- If P > 0,2 S

$$P_e = \frac{(P - 0{,}2S)^2}{(P + 0{,}8S)}$$

where,

P_e: is the effective rainfall

P: is the total rainfall

S: is the potential maximum retention of the soil, which depends on the CN, which in turn depends on the hydrological groups of the geological formations and their coverage. It is calculated as:

$$S = 25{,}4 \times (\frac{1000}{NC} - 10)$$

The CN have been tabulated by the NRCS based on the type of soil, its use, coverage and hydrological condition.

In order to define the soil type of the catchment, the Soil Recognition Map of Uruguay was used. The whole catchment belongs to Tala-Rodriguez soil unit, which has associated the hydrology group C/D.

The soil use was identified with the Land Used Map of the Ministry of Livestock, Agriculture and Fisheries of Uruguay. The following Table presents the Land Uses in the catchment and its area.

Land Use	Area (km^2)
Aerodromes	0.36
Urban area	9.86
Industrial areas	0.47
Dry crop > 2 ha	40.05
Wet crop (>2ha)	31.09
Wet crop < 2 ha y dry crop < 2 ha	17.10
Watercourses	0.11
Seasonally Flooded Herbaceous	0.21
Herbaceous Permanently Flooded (Pajonal)	0.18
Sports facilities	0.11
Lagoons and reservoir	7.52
Bushes < 5 ha,	6.64
Native forest	13.78
Riverside native forest	4.99
Urban parks	0.27
Eucalyptus tree plantation> 5 ha,	0.28
Fruit tree plantation	31.36
Forest plantation > 5 ha,	5.02
Forest plantation of Pine > 5ha,	1.26
Natural meadow	351.65
Natural or improved meadow or arable crop	147.21
Uncovered soil	0.06
Uncovered soil associated to agriculture or forest plantation	7.35
Disperse urban areas and crops	2.68
Disperse urban areas and natural meadow	8.64

Table 2–3 Land use of the catchment

This land uses were associated with the categories of land use presented in the Urban Storm Water Design Manual of DINASA, which have associated the NC. The following table presents the final land uses adopted, the NC associated to land use C and D, their average, and the associated area.

Land use	NC of C	NC of D	Average between C and D	Area (km^2)
Commercial areas	94	95	94.5	0.46
Forest	77	83	80	63.33
Crops cultivated in rows	84	88	86	88.62
Industrial areas	91	93	92	0.47
Low density grass and bushes	71	78	74.5	7.40
Water bodies	100	100	100	7.62
Meadows	85	89	87	499.12
Residential	81	86	83.5	21.19

Table 2–4 Adopted land use of the catchment

The final NC of the catchment was computed by calculating the weighted average of the NC associated to the different land uses. The final NC adopted value was 86.

2.2.3. Computed hydrograph

A Unit Hydrograph was constructed using the time of concentration and the area of the catchment according to the SCS methodology presented in the Urban Stormwater Design Manual of DINASA. The Unit Hydrograph consists of a triangle that has the following shape:

$$t_p = \frac{D}{2} + 0{,}6 \times t_c$$

$$t_b = 2{,}667 \times t_p$$

$$q_p = \frac{0{,}208 \times A}{t_p}$$

where,

t_p: is the time to peak of the hydrograph (hours)

D: is the duration of the block of rainfall (hours)

t_c: is the time of concentration (hours)

t_b: is the base time of the hydrograph (hours)

A: is the area of the catchment (km^2)

q_p: is the maximum discharge of the hydrograph (m^3/s)

Subsequently the properties of linearity and overlap were applied by multiplying the Unit Hydrograph by each increment of runoff and adding these hydrographs by displacing them over time. In this way, a hydrograph corresponding to the design storm is obtained, whose integral in time is equal to the water drained volume.

The following figure presents the obtained hydrograph.

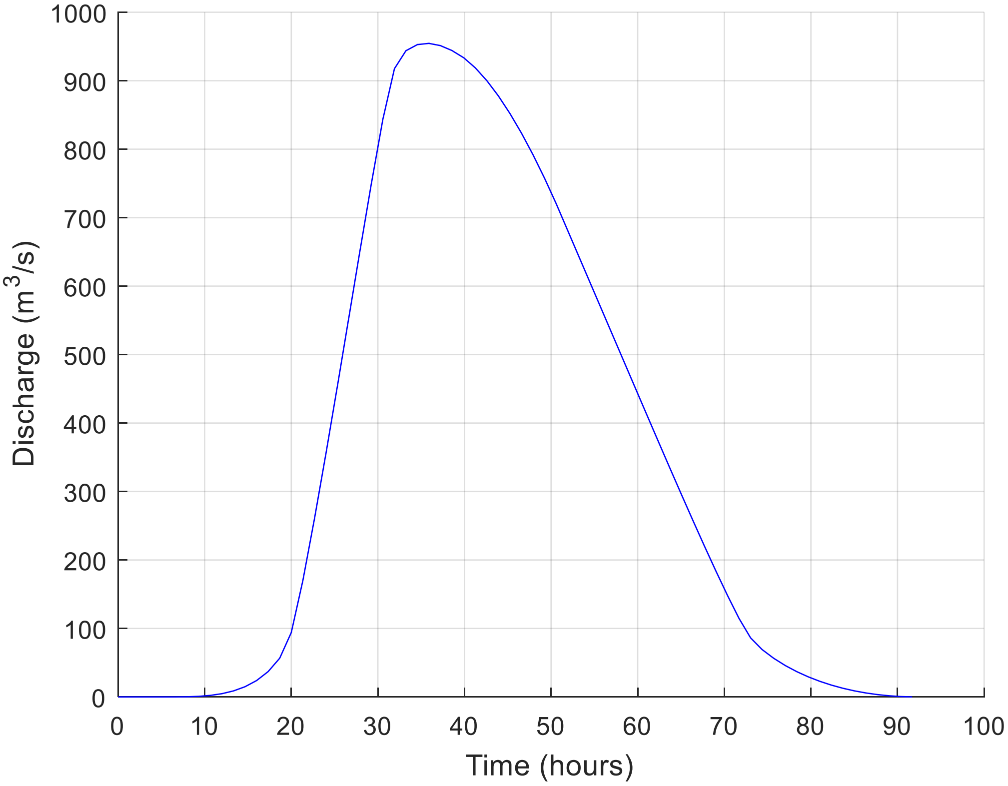


Figure 2–3 Computed hydrograph

3. Hydraulic study

3.1. Description of the HEC-RAS modeling system

The HEC-RAS is a hydrodynamic modeling system designed to simulate one-dimensional free surface flow in networks and natural or artificial channels. The model is developed by the Hydrologic Engineering Center of U.S. Army Corps of Engineers and has been extensively tested.

The system contains four main components for the hydraulic analysis of the pipes:

- Calculation of the free surface profile for steady flow.
- Non-stationary flow simulation.
- Calculation of sediment transport with moving bed.
- Analysis of water quality

The key element of the modeling system is that the four components use the same physical model and routines for the hydraulic and geometric calculation. In addition, the system contains several utilities for designing hydraulic structures, which can be invoked once the basic profiles of the free surface have been calculated.

3.2. Cross-sections

The bathymetry of the Canelón Grande River was the result of the combination of the following sources:

- Cross-sections taken by the company Sigma Plus for the hydrodynamic study of the Santa Lucia River for low flows
- Digital terrain model obtained from the Ministry of Livestock, Agriculture and Fisheries of Uruguay (MGAP, for its name in Spanish)

The following image presents the location of the considered cross-sections.

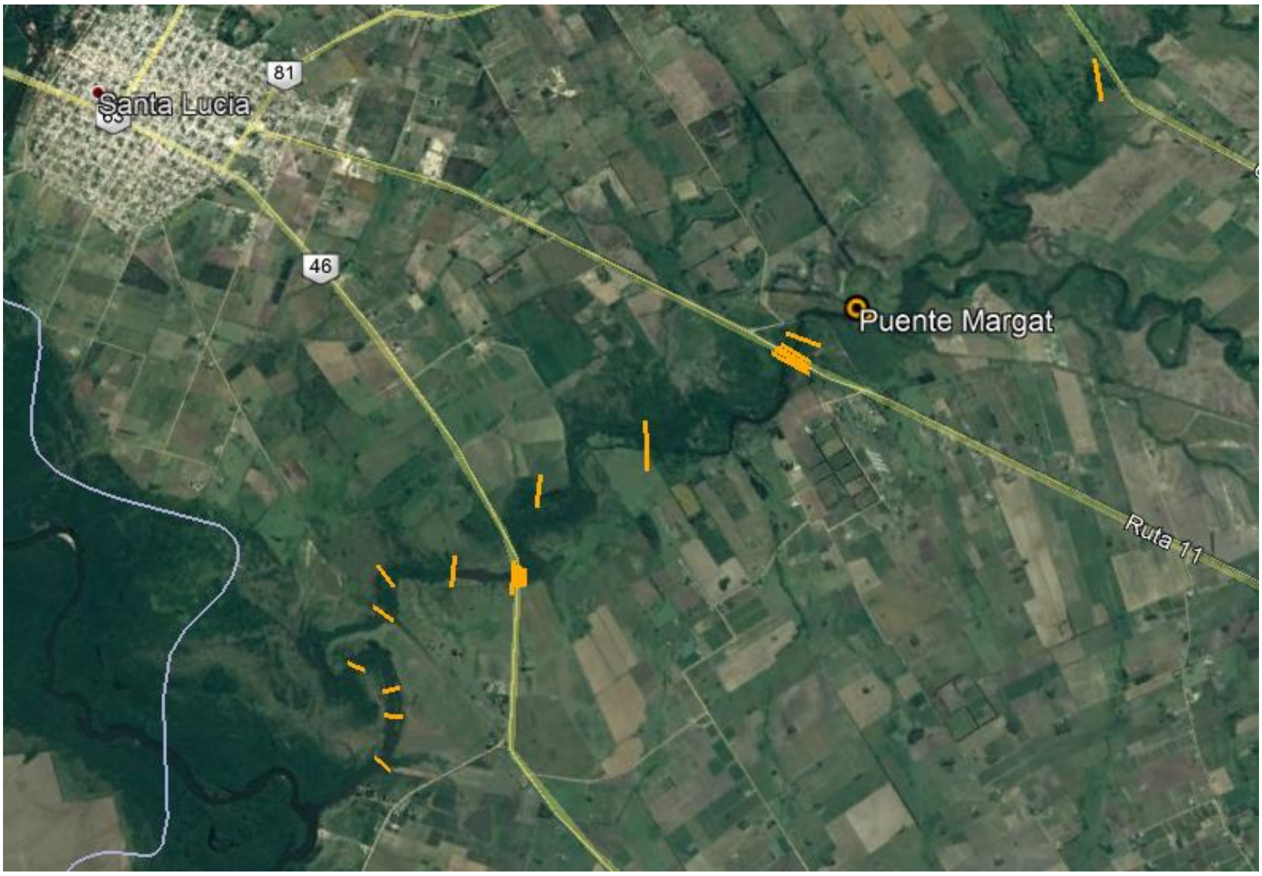


Figure 3–1 Cross-sections considered for the hydrodynamic model

3.3. Bridges

The following bridges at the Canelón Grande River were considered in the model:

- Route 46
- Route 11
- Margat Bridge

The geometry of the bridges was taken from the hydrodynamic model created by the Institute of Fluid Mechanics and Environmental Engineering (IMFIA, for its name in Spanish) for the hydrodynamic study of the Santa Lucia River for low flows. All this geometric data was reviewed with satellite images from Google Earth. In particular, it was found that the bridge located in Route 11 has piers with a separation of 15 meters between each other. It was supposed that these have diameter of 1 m.

3.3.1. Existing Margat Bridge

According to the information provided by to the consultant and the data taken from the IMFIA model, the Margat Bridge has the following geometry:

- Bottom deck level at +12.1 Official Zero
- Upper deck level at +12.9 Official Zero
- Length of the deck: 54 m

- No piers

3.3.2. Future design of the Margat Bridge

According to the information provided to the consultant the future bridge will have the following characteristics:

- Bottom deck level at +12.1 Official Zero
- Length of the deck: 86.6 m
- Two piers with diameter equal to 1 m separated 32 from each other
- Lateral slope: 1V:2H

3.4. Boundary conditions and roughness

The model presents the following boundary conditions:

- Upstream: hydrograph presented in item 2.2.3.
- Downstream: constant water level.

Regarding the water level two cases were considered:

- **Case 1:** Represents the worst case scenario for head loss and velocity magnitude and it consists in a water level where the backwater curve from the intersection of the Canelón Grande and the Santa Lucia Rivers does not reach the Margat Bridge. To select this water level a sensitivity analysis was performed. The final water level selected was 5 m referred to the Official Zero.
- **Case 2:** Represents the worst case scenario for the water levels at the bridge and considers that the water level at the confluence is the water level associated to the 100 year return period flood for the Santa Lucia River. A previous study held by the consultant for the state company Obras Sanitarias del Estado (OSE) estimated that the water level at Aguas Corrientes and the Santa Lucia City for the 100 year return period flood is 11.72 m and 13.72 m refer to the Official Zero respectively. Considering linear interpolation, the water level at the confluence is 11.90 m.

Three sources were considered for the selection of the roughness (Manning coefficient) of the model:

- Calibrated hydrodynamic model created by the IMFIA for the Santa Lucia River for low flows
- Handbook of applied hydrology: a compendium of water-resources technology, Chow, V. T. (1964).
- Urban Storm Water Design Manual of DINASA

Finally, the adopted Manning coefficients were 0.06 for the main channel and 0.09 for the flood plains. These values were the ones adopted by the IMFIA and they are inside the recommended ranges for natural rivers.

3.5. Results

3.5.1.1. Case 1

The following figure presents the maximum water levels profile focused on the Margat Bridge area with its current and future designs. The vertical lines at 9600 m and 9620 m approximately mark the location of the Margat Bridge.

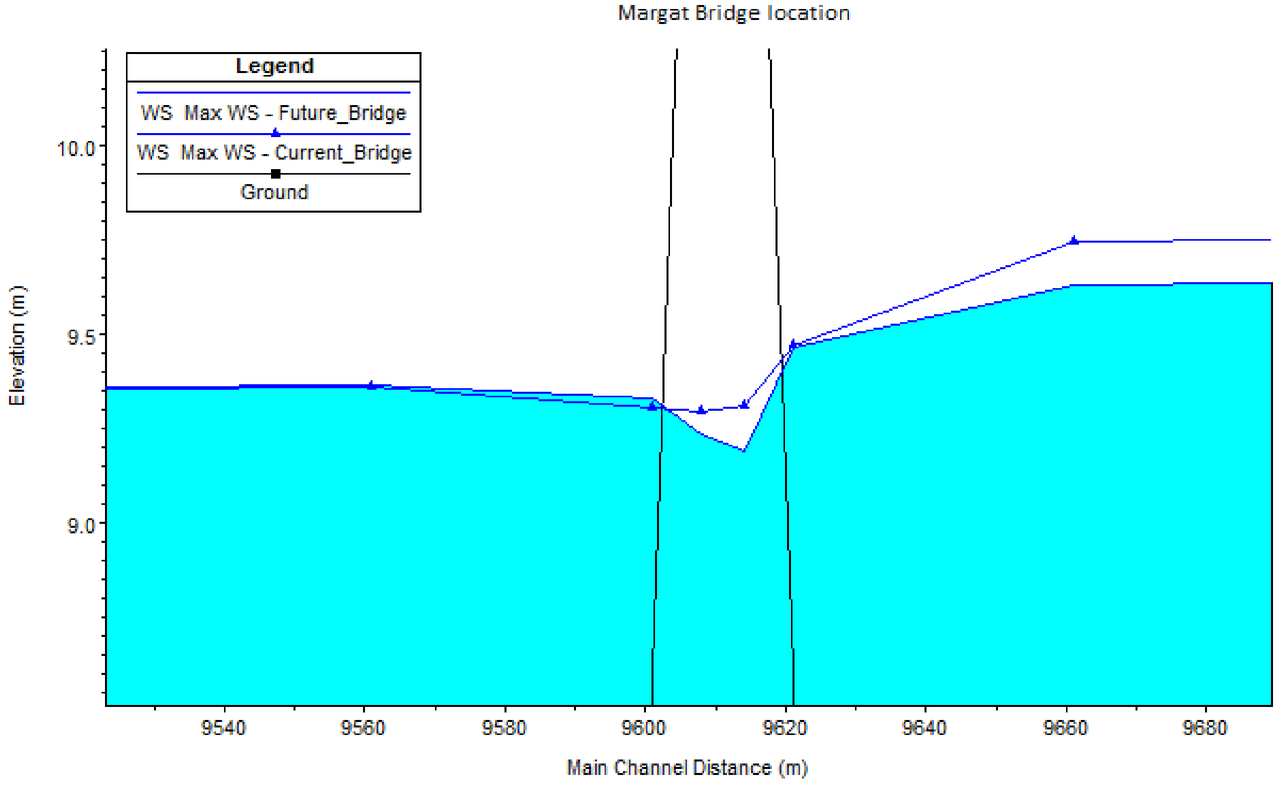


Figure 3–2 Maximum water level profile focused on the Margat Bridge area with its current and future design for Case 1

The following figure presents the water levels immediately upstream (Stage HW, for headwater) and downstream (Stage TW, for tailwater) of the Margat Bridge with its current and future designs.

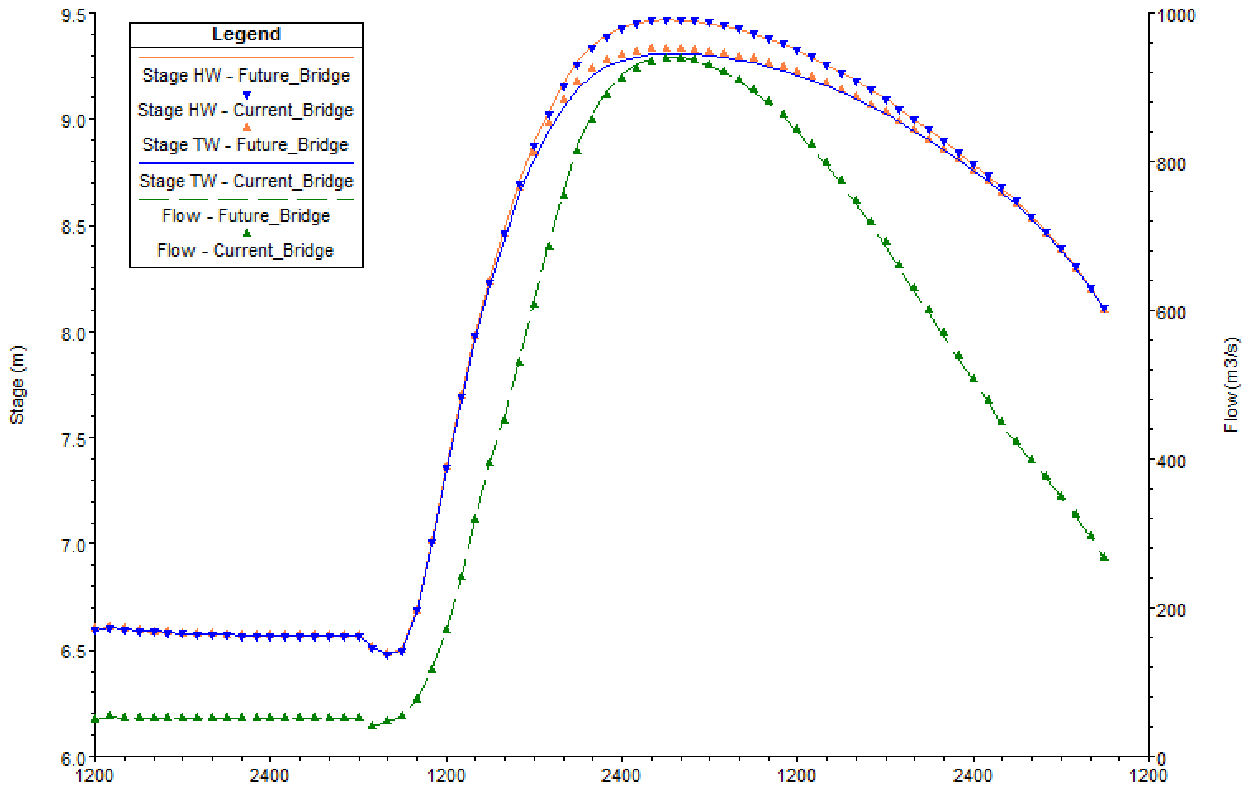


Figure 3–3 Water levels, referred to the Official Zero, upstream and downstream of the current and future Margat Bridge for Case 1

The following table presents the maximum water level and velocity immediately upstream and downstream of the bridge for both scenarios.

	Current design		Future design	
	Downstream	Upstream	Downstream	Upstream
Maximum water level Official Zero (m)	9.31	9.47	9.33	9.47
Maximum velocity (m/s)	3.35	3.39	2.31	2.58

Table 3–1 Maximum water level and velocity in the main channel upstream and downstream of the current and future Margat Bridge for Case 1

It can be seen that despite of the fact that the maximum water level in both designs remains almost the same, the velocity in the main channel decreases with the future design of the bridge. This is a consequence of the fact that the future bridge is longer than the current one, thus the total area available for the flow under the bridge increases.

3.5.1.2. Case 2

The following figure presents the water levels upstream and downstream of the bridge with its current and future design.

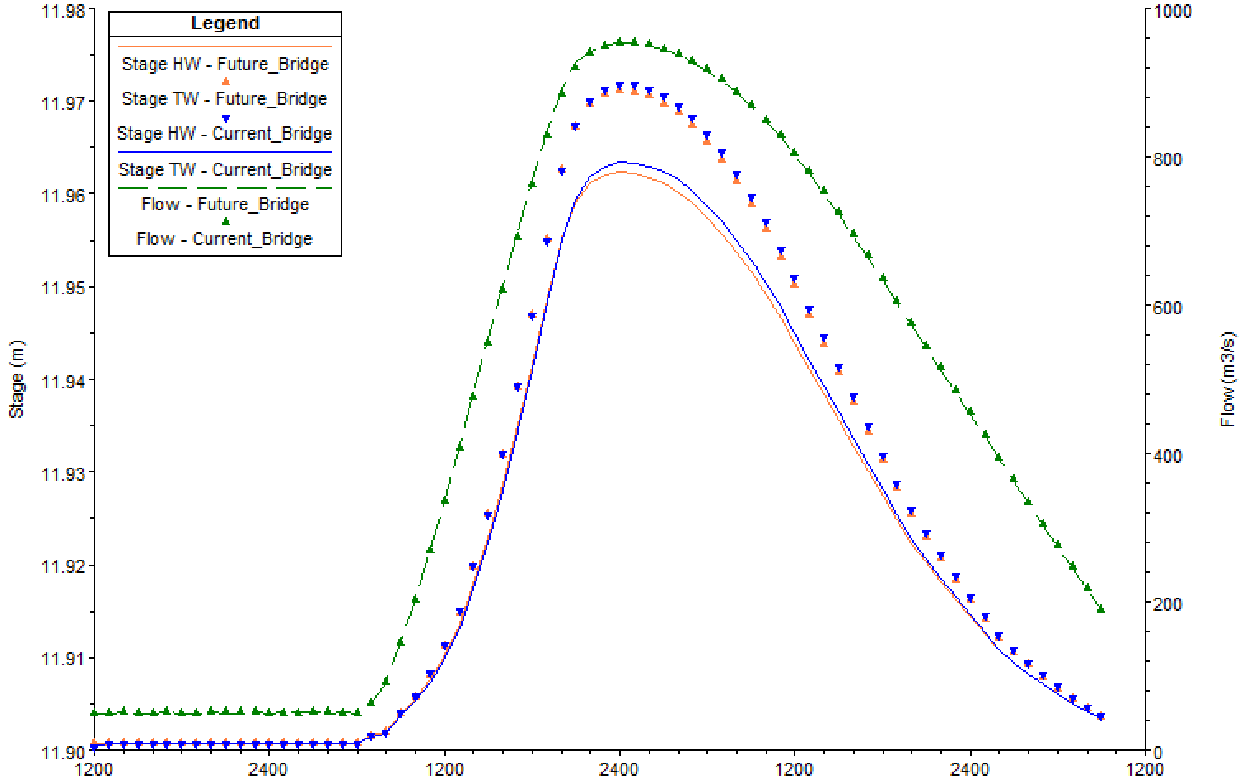


Figure 3–4 Water levels, referred to the Official Zero, upstream and downstream of the current and future Margat Bridge for Case 2

The following table presents the maximum water level and velocity for this case scenario.

	Current design		Future design	
	Downstream	Upstream	Downstream	Upstream
Maximum water level Official Zero (m)	11.97	11.96	11.96	11.97
Maximum velocity (m/s)	2.39	2.43	1.65	1.79

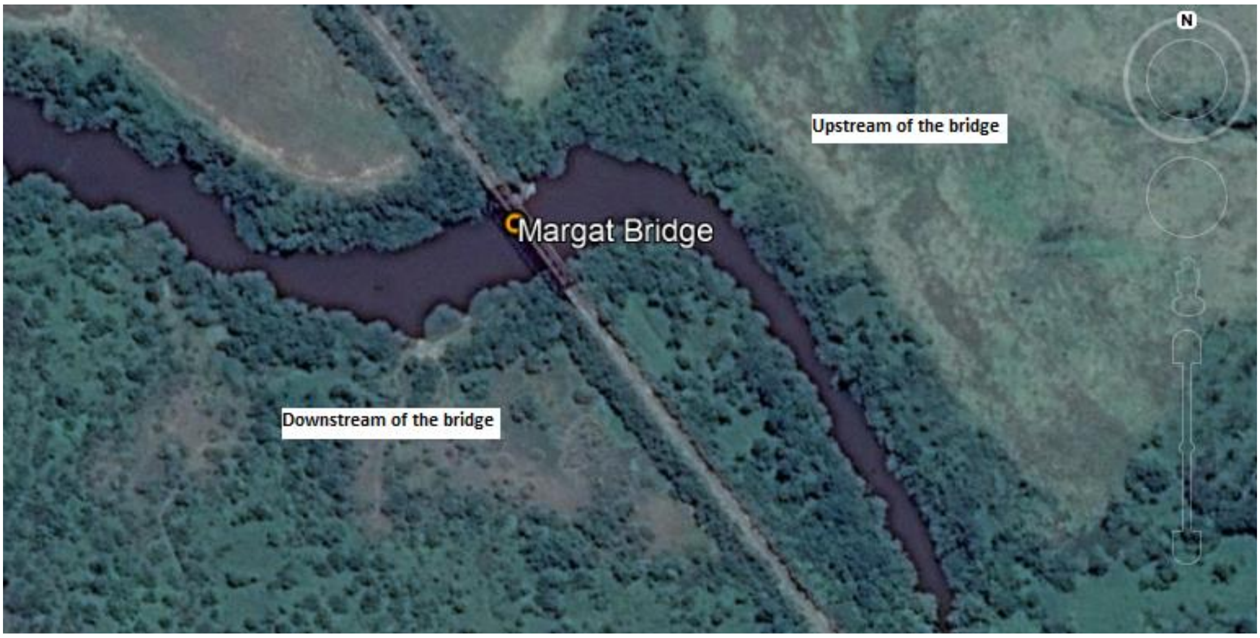
Table 3–2 Water levels, referred to the Official Zero, upstream and downstream of the current and future Margat Bridge for Case 2

We can see that with both designs the water levels are practically the same (small variations due to numerical approximations), given that the section of the Margat Bridge is inside the backwater curve induced by the boundary condition of the Santa Lucia river.

As we saw in the evaluation of Case 1 (see item 3.5.1.1) the velocity decreases with the new design of the bridge. It is worth noticing that these water levels are extremely close to the bottom level of the deck of the Margat Bridge (bottom level at +12.1 Official Zero).

3.6. Comments regarding the location of the bridge

The following figure presents a satellite image from Google Earth of the location of the Margat Bridge.



In the image it can be seem that the river presents a sharp bend immediately upstream of the bridge. According to the paper "Flow separation in sharp meander bends"[3] abrupt changes in the direction of the flow may result in flow separation from boundaries. This, combined with vortex bar formation, results in a decrease of the river width and conveyance capacity. The sharp bend modifies patterns of bed and bank erosion and can lead to localized and focus bank erosion, which differs from the flow structure and morphodynamics in less sharp meander bends.

As a consequence of this phenomenon, an area of turbulence may appear in the inner side of the meander that may cause the scouring of the left bank and the reduction of the conveyance capacity of the river.

This may indicate that the location of the bridge is not the optimum and that a detail scouring study should be held for the executive design of the bridge. This study is out of the scope of the present study.

[3] Kleinhans, M. G., Blanckaert, K., Mc Lelland, S. J., Uijttewaal, W. S., Murphy, B. J., van de Kruijs, A., & Parsons, D. (2010). Flow separation in sharp meander bends. In Proceedings of the HYDRALAB III Joint User Meeting (No. EPFL-CONF-161943).

3.7. Limitations

Some limitations of this study are worth to be mentioned:

- Hydrological modelling of the catchment is a simplified one as there was not discretization between Canelon Grande and Canelon Chico subbasins. Furthermore, Canelon Grande Reservoir was not modelled, hence its hydrograph attenuation is not considered. It is a conservative approach in order to obtain results with the available information and project deadlines. Nevertheless, as it can be seen in results, the hydrograph is not as restrictive, from the maximum water elevation point of view, as downstream boundary condition.
- Downstream boundary condition from Santa Lucia River was taken from a model of that River from the company and interpolated at the confluence with Canelon Grande. Moreover an assumption of return periods simultaneity was done, which might be arguable. Nevertheless, catchment areas are just one order of difference, and again these assumptions were done in order to achieve projects deadlines with available information. Further improvements should be done in future stages of the project.
- The manning coefficients adopted were chosen from a model calibrated for low flows. Thus, the model used for this study is not calibrated.
- The cross-sections used for the creation of the model present measured data only in the main channel
- The hydraulic effects caused by the sharp bend located immediately upstream of the bridge are not considered

3.8. Conclusions and recommendations

As a result of the comparison of scenarios, the implementation of the future bridge reduces the velocity in the Canelón Grande River, and consequently its head loss. However, the maximum water levels obtained for Case 2, are extremely close to the bottom level of the deck of the Margat Bridge. Consequently, it is recommendable to increase the elevation of the bridge.

A more detailed study should be done for the executive design of the bridge. The present study only evaluates the affectation in the velocity (it increases or it decreases) due to the change in the design of the bridge. However, the velocity magnitudes given by the model cannot be used for the scouring study, where a detailed study is needed.