Hydraulic study for the railway line in the city of Durazno – Final Report

October 2017

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Acronyms

CN	Curve Number
DINAGUA	National Water Directorate
DINASA	National Water and Sanitation Directorate
IDF	Intensity – duration - frequency
NRCS	Natural Resources Conservation Service
SCS	Soil Conservation Service
Tr	Return period

1. Introduction

The aim of the study is to study the consequence of the increment of the level of the railway line in the city of Durazno and, if needed, propose a pre-design to avoid worsening the current situation of flooding.

To achieve this objective two study cases have been analysed. The first one corresponds to the current situation. The second one corresponds to the situation with the increment of the railway level. This study has been done for the 100 year return period event.

To study this, hydrological and hydraulic studies have been held. The hydrological study was done with the NRCS method and includes the characterization of the catchment, the computation of the time of concentration, effective rainfall and finally the design hydrograph.

The hydraulic studies were done with the HEC-RAS software form the US Corps of Engineers. As boundary conditions, was considered the water level at the Constitución dam (downstream) and the hydrographs computed in the hydrological study (upstream).

2. Hydrologic study

2.1. Catchment delimitation

The catchment considered in this study corresponds to the catchment with closing point in the railway bridge in the City of Durazno (544168.30 m E and 6308702.66 m S in UTM84-21S coordinate system). The following figure and table present the catchment delimitation and its characteristics

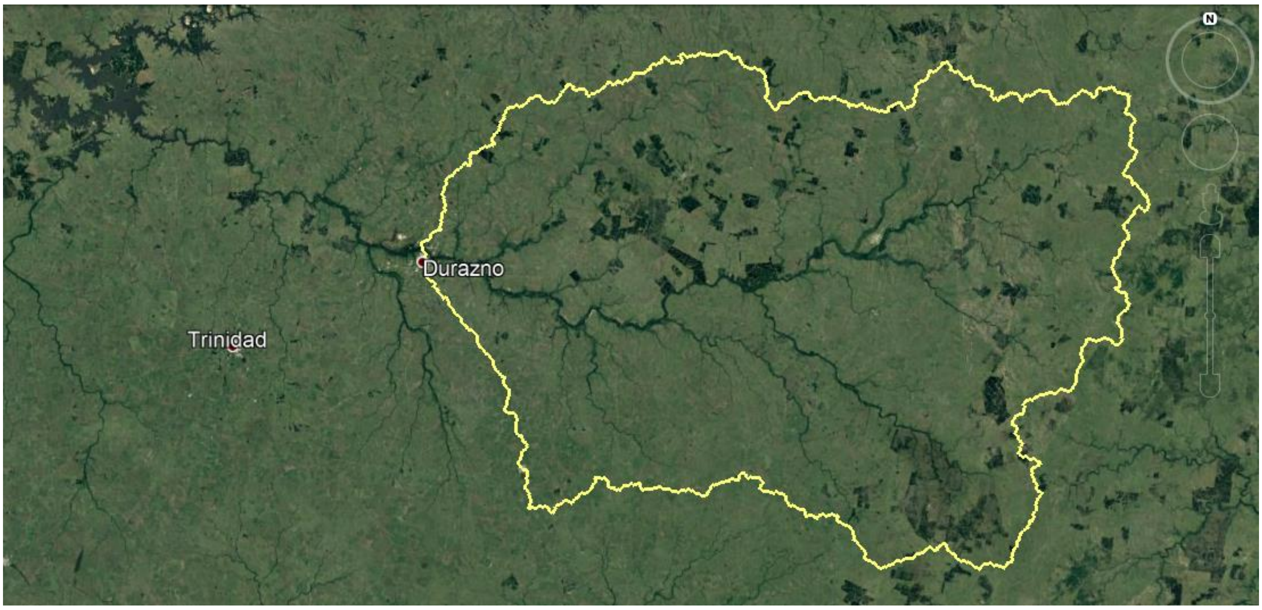

Figure 2–1 Catchment delimitation in the city of Durazno

Area (km^2)	Main length (km)	ΔH (m)	Slope (m/m)
8883	205	195	0.001

Table 2–1 Physical characteristics of the catchment in the city of Durazno

2.1.1. Input flow

The method of the Natural Resources Conservation Service (NRCS)[1] of the United States was used to define the hydrograph that corresponds 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.

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

The method consists of three stages:

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

2.1.1.1. Design Storm

The storm was built for 100-year return period and 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 these hypothetical storms, 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:

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

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

S: is the average slope of the longest hydraulic path

The following table presents the time of concentration of the catchment.

	Time of Concentration (hours)
Durazno catchment	58.24

Table 2–2 Time of concentration of the catchment

The numbers of blocks used to create the design storms were such that they cover at least twice the estimated time of concentration. We considered 31 blocks of 230 minutes for the catchment of the city of Durazno.

The precipitation intensity is the average rainfall rate, usually expressed in millimetres per unit of time. The value assumed is closely linked to the period of return 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:

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

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

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 the location of the Durazno catchment NC takes a value of 85 mm.

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

Duration	a	b
Less than 3.5 hours	75.5226	-0.547
More than 3.5 hours	95.4831	-0.725

Table 2–3 Coefficients a and b to compute rainfall intensity

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

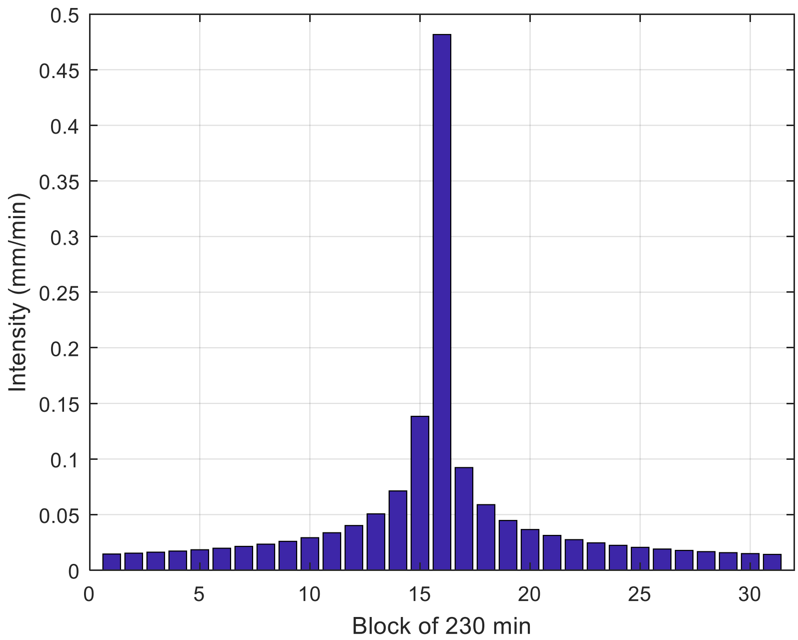

Figure 2–2 Synthetic design storm with 100 year return period for Durazno catchment

2.1.1.2. 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 2.1.1.1. 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:

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

The CN have been tabulated by the NRCS based on the type of soil, its use, coverage and hydrological condition. The soil type of the catchment was defined using the Soil Recognition Map of Uruguay. 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 soil type in the catchment, its percentage and the associated hydrology group.

Soil unity	Code	Hydrology group	Percentage (%)
Bacacuá	Ba	B	1.06
Capilla de Farruco	CF	B/D	0.56
Carpinteri	Cpt	D	7.07
Cerro Chato	CCh	B	9.33
Curtina	Cu	D	2.01
Isla Mala	IM	C	0.50
Itapebi – Tres Árboles	I-TA	D	0.39
La Carolina	LC	C/D	6.46
Montecoral	Mc	D	15.52
Punta de Herrera	PH	C	4.63
San Gabriel – Guaycurú	SG-G	B	24.63
Santa Clara	SCl	B	7.65
Sarandí de Tejera	SdT	B/C	6.39
Sierra de Polanco	SP	B/C	4.56
Trinidad	Tr	C/D	2.24
Yi	Yi	B/C	7.00

Table 2–4 Soil type of Durazno catchment

The following table shows the percentage that each hydrology group presents in the Durazno catchment.

Hydrology gropu	Percentage (%)
B	42.67
B/C	17.95
B/D	0.56
C	5.13
C/D	8.70
D	24.99

Table 2–5 Percentages of hydrology groups in the Durazno catchment

The information of the type of soil in the catchment was combined with the soil use to determine the curve number associate with the Durazno catchment. Finally, the adopted CN was 75.

2.1.1.3. Computed hydrograph

For each catchment, a Unit Hydrograph was constructed using the time of concentration and the area 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.

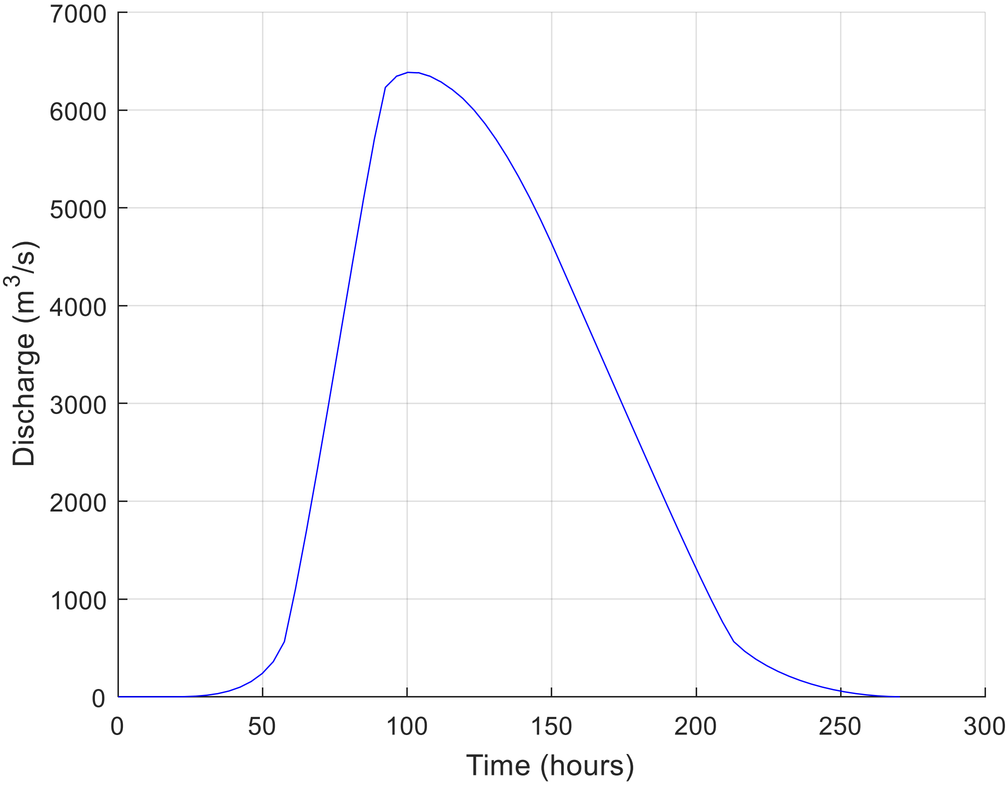

Figure 2–3 Hydrograph for Durazno catchment

3. Hydraulic study

3.1. Description of the HEC-RAS modelling system

The HEC-RAS is a hydrodynamic modelling system designed to simulate one-dimensional free surface flow in networks and natural or artificial channels. The model is developed by the Hydrologic Engineering Centre 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 modelling 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 considered for the hydrodynamic models was obtained the Digital Terrain Model 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 of the hydrodynamic model

3.3. Bridges

Two bridges were considered in the hydraulic model:

- Submersible bridge "Ing. Federico Capurro". It is located approximately 170 m downstream of the railway bridge of the city of Durazno. Although the exact geometry of the bridge was not available for this study, the geometry was estimated with images of Google Earth and the fact that the bridge is submersible.
- Railway Bridge in the city of Durazno. Two scenarios were considered:
  - Bridge with the current design of the railway towards the city of Durazno.
  - Bridge considering that railway towards the city of Durazno is at the same level than the bride.

The following figures present the considered scenarios.

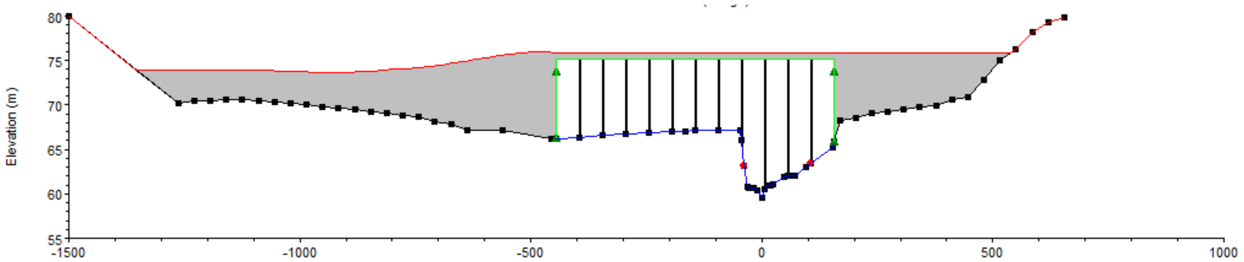

Figure 3–2 Railway bridge with the current design of the railway towards the city of Durazno

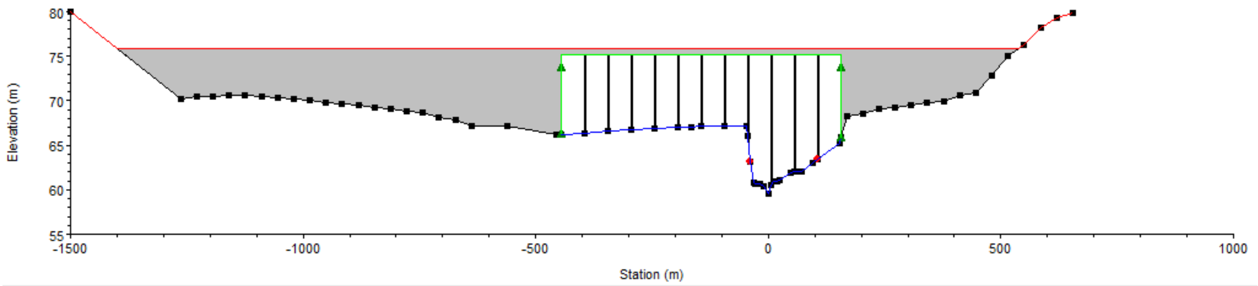

Figure 3–3 Railway bridge considering that the railway towards the city is at the same level than the bridge

The bridge located in Route 5 in the city of Durazno was not considered in this study. As it is located upstream of the railway bridge its influence is not significant for this study.

3.4. Construction of the model

The model was constructed in two steps:

1. Model from the city of Durazno until the Paso del Bote station
2. Model extended until the Constitución reservoir

In the following items these points are presented.

3.4.1. Model from the city of Durazno until the Paso del Bote measure station

This step was used to validate the Manning coefficient of the model. As the consultant has time series of measured water level data in Paso del Bote and measured water level and discharge in Durazno city for the 2007 flood (one of the largest recorded to date) this event was used.

Due to the time provided for the accomplishment of this study, the typical calibration-validation procedure recommended for hydrodynamic models was not performed. The adopted Manning coefficients were chosen from the ranges presented in bibliography for natural watercourses. The selected values were 0.07 for the main channel and 0.4 for the flood plains.

The validation of the roughness coefficients consisted in the comparison of the maximum water level given by the model of the current situation with the water level measured in the bridge “Ing. Federico Capurro”. The following table presents the obtained water levels.

Measured water level (m, Official zero)	Model water level (m, Official zero)
74.01	74.13

Table 3–1 Measured and model maximum water level for the 2007 flood event

3.4.2. Model extended until the Constitución reservoir

In a previous study held by the consultant in the Negro river catchment for the state company Administración Nacional de Usinas y Trasmisiones Eléctricas (UTE) showed that the water level in the Constitución dam reservoir does not affect the water level in the city of Durazno. So, the purpose of extending the model until the reservoir is to avoid that the downstream water level condition affects the results in the railway bridge. For all runs was adopted a water level of 44 m, referred to the Official zero, as downstream boundary condition that is the maximum water level considered by the study done for UTE.

3.5. Results

The following figure presents the maximum water levels profile focused on the Railway bridge area with the current and future designs.

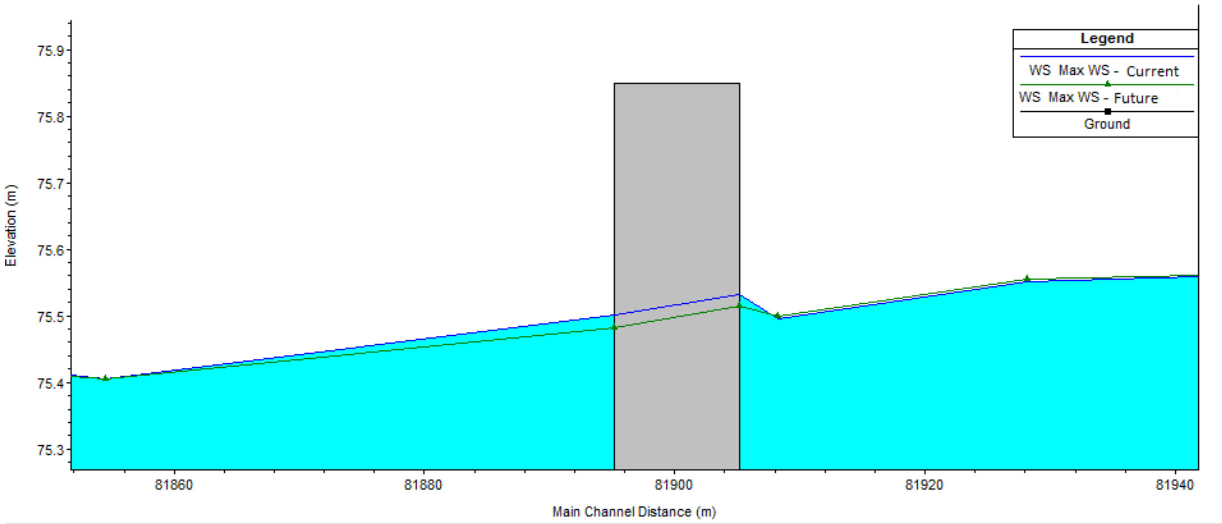

Figure 3–4 Maximum water level profile for the actual and future situation

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

	Current design		Future design	
	Downstream	Upstream	Downstream	Upstream
Maximum water level Official Zero (m)	75.41	75.50	75.41	75.50
Maximum velocity under the bridge (m/s)	0,88		1,06	
Discharge under the bridge (m^3/s)	5779		5978	
Discharge over the railway in Durazno (m^3/s)	198		0	

Table 3–2 Maximum water level and velocity in the main channel downstream and upstream the bridge for the current and future design

Several things can be seen in Figure 3-4 and Table 3-1:

- The maximum water level does not present a significant variation between the current and future situations.
- There is a different situation exactly under the bridge, where the velocity in the future design is bigger, but it doesn’t affect the upstream cross section.
- The discharge under the bridge is higher in the future scenario

The overtoping of the railway line in the present situation represents only 198 m^3/s (3% of the total discharge). The results of the simulation with the future design show that the section under the bridge can absorb that 3% without affecting the water level upstream.

It can be concluded that no further measures are needed. However, to better illustrate this conclusion, it was calculated the culvert needed to let cross the 198 m^3/s that currently cross over the railway line. For the design the following data was considered:

- Water level in the upstream cross-section of the railway bridge in the future situation: 75.50 m
- Water level in the downstream cross-section of the railway bridge in the future situation: 75.41 m
- Cd = 0.84
- Location: km 199+400 of the railway profile provided
- Downstream invert level = 69 m
- Length = 35 m

Under these conditions the culvert is submerged. If it is consider for the computation a square culvert with 2 m side (typical size used by the Ministry of Transportation and Public Works), 49 culverts are needed to achieve a discharge of 200 m^3/s. If a square culvert with 3 m side is used, 21 culverts are needed. This numbers clearly show that really big culverts are needed to let pass a flow that the bridge can convey without affecting the upstream levels.

4. Limitations and conclusions

4.1. Limitations

Some limitations of this study are worth to be mentioned:

- The cross-sections used to create the hydraulic model were obtained from a DTM.
- The hydraulic model was validated using the 2007 flood event but no calibration has been performed.

4.2. Conclusions

The study concludes that, with the precision of the model used, the future design does not produce a worse flood situation in the city of Durazno.

The discharge that currently crosses over the railway is 198 m^3/s and represents the 3% of the total discharge associated with the 100 year return period event. With the new design of the railway, this discharge will flow under the bridge not producing significant differences with current situation.

The number of culverts needed to transport a discharge of 198 m^3/s is really high and technically they are not necessary. Thus, the decision of locating them under the bridge and their design is a political decision due to the sensitivity of the topic in the Durazno community.