

Article

Hydraulic Planning in Insular Urban Territories: The Case of Madeira Island—Ribeira Brava

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Abstract: The main objective of this study was to analyze the flood propensity of the main water-course of Ribeira Brava drainage basin and, if necessary, to suggest two methodologies to mitigate the impacts, i.e., detention basin sizing and riverbed roughness coefficient adjustment. For the flood propensity assessment, geomorphological data obtained from the watershed characterization process was used through the SIG ArcGIS software and then used for the calculation of the expected peak flow rate for a return period of 100 years using the Gumbel Distribution. Finally, the drainage capacity of the river mouth was verified via the Manning-Strickler equation, with the purpose of establishing whether the river mouth of the watershed has the capacity to drain the entire volume of rainwater in an extreme flood event. In summary, the results indicate that the river mouth of the Ribeira Brava watershed does not have the capacity to drain the rain flow for the pre-established return period; therefore, the detention basin was sized using the Dutch Method and the Simplified Triangular Hydrograph Method, and the roughness coefficient was adjusted in order to ensure the increase in the river mouth's drainage capacity.

Keywords: hydraulics; hydrology; insular territories; spatial analysis; territorial management; urban planning

1. Introduction

Despite the economic and technological development after the Industrial Revolution, the Earth has constantly suffered from the effects of anthropogenic actions. As a result, extreme climatic phenomena have become one of the main problems faced by humanity, and floods are the most frequent of these events in regions with high urbanization [1–4]. Therefore, living with flood events has become increasingly intolerable over time, increasing the risk to human and economic loss [5].

As noted, floods tend to be more severe in urban areas because they occur during high intensity precipitation and where the soil seepage rate is insufficient to absorb the entire volume of rainwater, thus favoring surface drainage [6–9]. However, must be highlighted that this non-absorption of precipitation by the soil can occur both by the saturation process

and by the waterproofing of the land [10]. Nonetheless, in this context, due to the increase in the urban network in regions that were previously floodplains, there is an increase in the magnitude of the negative consequences in a densely urbanized region [11].

It should be noted that floods also occur when the drainage capacity of urban drainage systems is insufficient to cope with the flow that exceeds the absorption capacity of the soil. In such cases, this undrained volume can flood streets, businesses, houses, and industries [9,12,13]. Furthermore, mountainous regions are more prone to the occurrence of flash floods because steep slopes allow greater gravitational energy and, consequently, a greater destructive force for surface flows [12,14]. Therefore, it is possible to ascertain that high-altitude volcanic islands such as Hawaii, Gran Canaria, Reunion, and Madeira have geomorphological and lithological features that make them more susceptible and vulnerable to flooding [13–15].

As a result of the increase in this problem globally, and especially in tropical regions, it has become necessary to develop measures and devices to mitigate the impacts of floods [3–5]. Initially, the main guideline for urban drainage was the conventional principle of rapid removal of the cause of the problem, i.e., redirecting the excess flow from its origin to another location [5]. However, the principle of rapid removal of the problem, even if it is efficient for upstream regions, causes flooding to worsen in downstream areas. In this case, the problem is simply transferred from one area to another. Thus, the problem, which is caused by anthropic actions, and the geomorphological and hydrological characteristics of the watershed, is not actually solved at the source. Therefore, there is a need to use practices with the greatest capacity to mitigate the effects of floods; that is, to establish effective procedures at the source of the problem, without redirecting the destructive effects from one zone to another [4,5,14,16].

Based on this premise, this study aimed to undertake the hydrological analysis of the study region, in order to verify the expected peak flow rate for a recurrence time of 100 years, and then compare it with the drainage capacity of the stream mouth of the Ribeira Brava watershed. Following the assessment that the hydraulic characteristics of the mouth are not sufficient for the drainage of the estimated peak flow rate, a detention basin was sized as a means to regularize the flow downstream, allowing the river mouth to function within its current dimensional characteristics. Furthermore, this study also aimed to indicate a structural intervention for the stream mouth, with significantly reduced urban impacts. This intervention refers to changing the physical characteristics of the walls and riverbed of the stream, namely, the roughness coefficient. Therefore, the minimum characteristics of the stream are verified in order to increase the drainage capacity, without the need for dimensional changes.

2. Materials and Methods

2.1. Area of Study

The area of study—i.e., the Ribeira Brava watershed—is located on the southern slope of Madeira Island, between latitudes 30°01' N and 33°08' N and longitudes 15°51' W and 17°30' W [17,18]. This watershed belongs to the homonymous municipality, Ribeira Brava, and corresponds to the precipitation catchment area that supplies the main stream of the municipality, as shown in Figure 1.

In addition to Funchal, the main municipality of the island, the Ribeira Brava watershed suffers from severe flooding problems, as observed in 2010 and 2013, when the region was impacted by significant losses, both in material and human terms. Because it is located in a considerably urbanized area, the Ribeira Brava watershed has a significant rate of soil sealing from the buildings and pavements present [19,20]. In addition, as can be observed in Figure 2, the river mouth of the Ribeira Brava watershed has the presence of vegetation and sedimentation, which significantly reduces the drainage capacity of the channel.

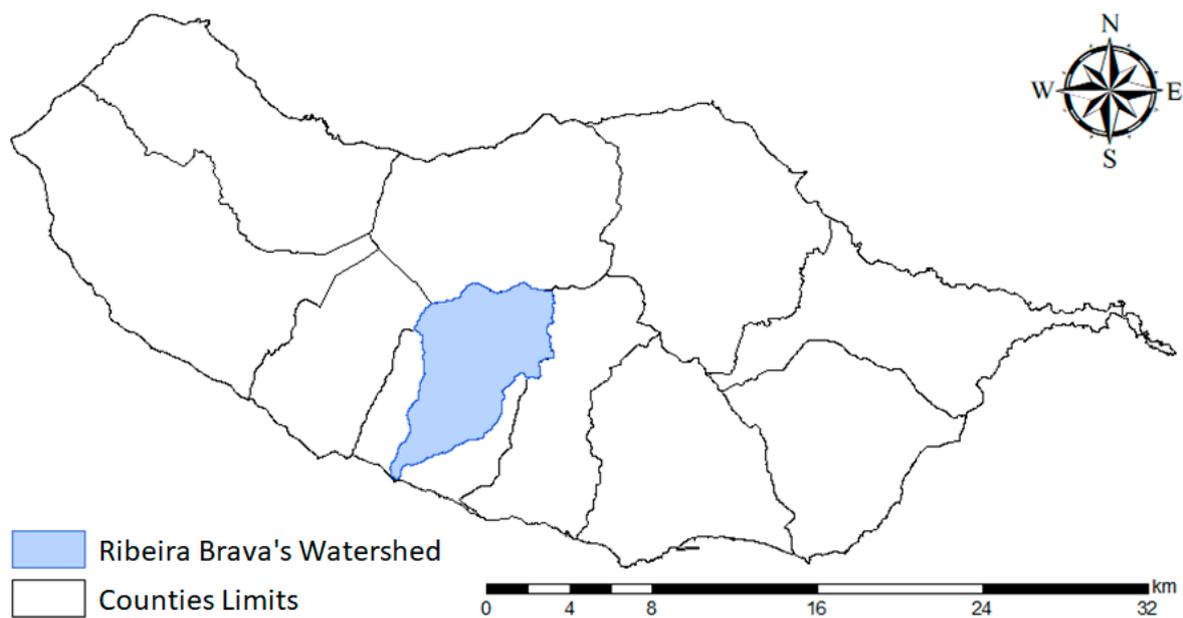


Figure 1. The Ribeira Brava watershed. (Source: Authors by ESRI ArcGIS, 2020).



Figure 2. State of conservation of Ribeira Brava main watercourse river mouth.

The state of conservation of the stream is practically the same throughout its length within the urban perimeter, which can be easily verified in situ. The excess of sedimentation and presence of vegetation is explained by the significantly reduced slope, which reduces the drainage velocity and the capacity to drag sediments with a larger grain size.

2.2. Schematic of the Methodology

The methodology adopted can be synthesized in 6 stages, as shown in Figure 3.

The methodology adopted started from a literature review, with the objective of gathering all the necessary information for the hydrological and morphometric characterization of Ribeira Brava basin. Therefore, through the aforementioned review, the methodologies recommended by several authors in the field were selected as a means to obtain a more reliable analysis of the flood propensity in the basin under study. Finally, the various steps of the adopted methodology presented in Figure 3 are described below.

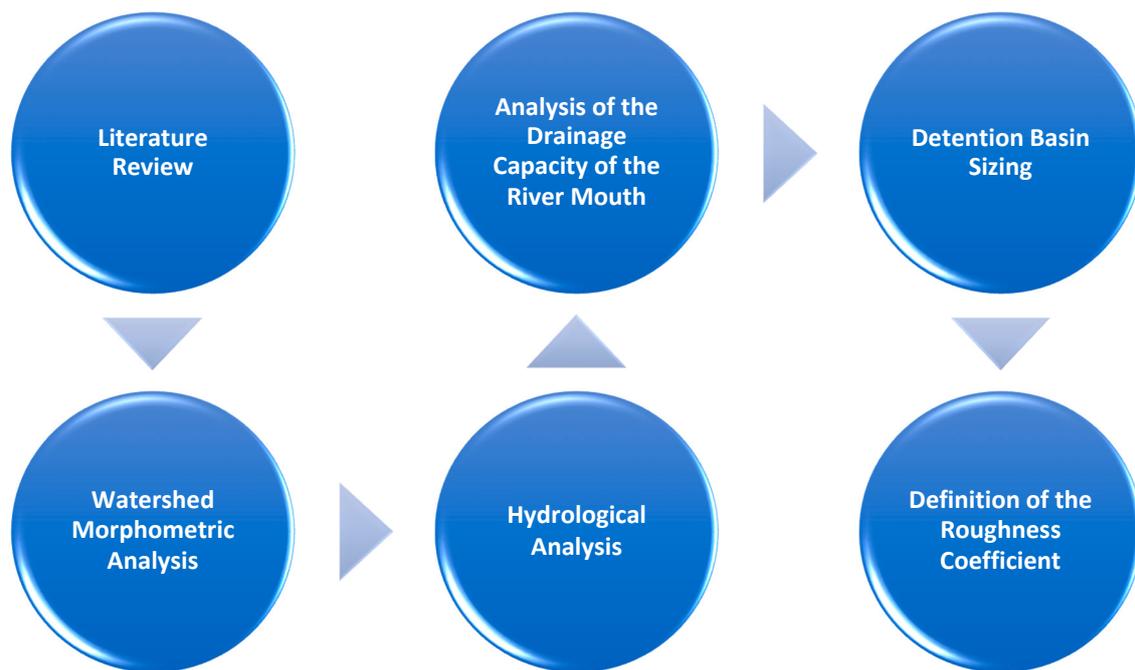


Figure 3. Organogram of the adopted methodology.

2.3. Morphometric Characterization of the Watershed

The main parameters used for the morphometric characterization of a watershed are [1,4,5,9,10,21–23]:

- **Gravelius Index— K_C** : Establishes the relationship between the actual perimeter of the basin under study and the perimeter of a perfectly circular basin, yet with the same area, thus establishing how close the geometric shape of the watershed is to a perfect circle [9,22]. It can be calculated using Equation (1). It is a dimensionless parameter, and the closer it is to “1”, the more rounded the watershed is and the greater its propensity to flood [22].

$$K_C = P/2 \times \sqrt{\pi \times A} \quad (1)$$

where:

P = Perimeter of the watershed, km;
 A = Watershed area, km².

- **Elongation Factor— K_L** : Establishes the relationship between the watershed under study with an equivalent rectangle of the same area. It can be calculated using Equation (2). This parameter is dimensionless and, if it is higher than “2”, it is an elongated watershed [22].

$$K_L = \frac{L_E}{l_E} = \frac{\frac{K_C \times \sqrt{A}}{1.128} \times \left| 1 + \sqrt{1 - \left(\frac{1.128}{K_C} \right)^2} \right|}{\frac{K_C \times \sqrt{A}}{1.128} \times \left| 1 - \sqrt{1 - \left(\frac{1.128}{K_C} \right)^2} \right|} \quad (2)$$

where:

L_E = Equivalent length, km;
 l_E = Equivalent width, km;
 K_C = Gravelius Index, dimensionless;
 A = Watershed area, km².

- **Shape Factor— K_F :** Establishes the relationship between the average width and length of the watershed under analysis. It can be calculated using Equation (3). This parameter is dimensionless and also indicates the degree of elongation of the watershed; however, the lower the value obtained, the greater the elongation and lower the propensity to flood. Values equal to the unit “1” indicate a square basin.

$$K_F = A/L_B^2 \quad (3)$$

where:

A = Watershed area, km^2 ;

L_B = Length of the watershed, km.

The length of the watershed can be determined by measuring the distance between the mouth of the stream and the furthest point. However, it is noted that the length of the watershed does not necessarily correspond to the length of its main watercourse, because the latter generally has a larger dimension due to its sinuosity. Based on the MDE file provided by LREC-RAM (the Regional Civil Engineering Laboratory of the Autonomous Region of Madeira), it was possible to perform the morphological characterization of the Ribeira Brava watershed and its respective main watercourse. The morphological data collected in this analysis were used in the empirical equations of several authors, as a means to avoid single method restrictions.

A priori, a morphometric analysis must have its origin in the establishment of a hierarchy of the watercourses—e.g., Strahler or Shreve—according to their order or magnitude, respectively [23]. The aforementioned classifications can be obtained by performing a hydrological analysis of the DEM file, obtaining the rasters “flow accumulation” and “flow direction”, from the “flow order” tool [21]. Furthermore, it is evidenced that the Strahler hierarchy is deeply associated with the branching or bifurcation ratio of a watershed, in which each degree of branching or bifurcation can be calculated using Equation (4) [1,10,16,21–24].

$$R_B = \frac{N_i}{N_{i+1}} \quad (4)$$

where:

N_i = Number of watercourses classified as “ i ”; dimensionless;

N_{i+1} = Number of watercourses classified as “ $i + 1$ ”, dimensionless.

This is a dimensionless coefficient, obtained by the ratio of the number of watercourses of a given order by the number of watercourses of the immediately higher order. The average bifurcation value can be calculated based on Equation (5).

$$\overline{R_B} = \sqrt[i-1]{\prod_{i=1}^{i-1} \frac{N_i}{N_{i+1}}} = \sqrt[i-1]{N_1} \quad (5)$$

where:

N_i = Number of watercourses classified as “ i ”; dimensionless;

N_{i+1} = Number of watercourses classified as “ $i + 1$ ”, dimensionless;

N_1 = Number of first-order watercourses.

Like the previous parameters, the average bifurcation ratio is a dimensionless parameter because it only represents an arithmetic mean of the bifurcation ratios. Moreover, one of the fundamental parameters for a good morphometric characterization of a watershed refers to the concentration time, which establishes the time required for the entire watershed area to contribute to the process of rainfall drainage to the river mouth [1,10,23,24].

Because the equations for determining the concentration time have an empirical character, each methodology indicates different values for the same parameter, where it is prudent to establish the arithmetic mean to avoid extreme values. For the present study,

the arithmetic mean was used for the values obtained by Kirpich (Equation (6)), Témez (Equation (7)), and Giandotti (Equation (8)) [21].

$$t_C = 57 \times (L^3 / (H_{MAX} - H_{MIN}))^{0.385} \quad (6)$$

where:

t_C = Time of concentration, minutes;
 L = Length of the main watercourse, km;
 H_{MAX} = Maximum height of the main watercourse, m;
 H_{MIN} = Minimum height of the main watercourse l , m.

$$t_C = \left(\frac{L}{i^{0.25}} \right)^{0.76} \quad (7)$$

where:

t_C = Time of concentration, hours;
 L = Length of the main watercourse, km;
 i = Slope of the main watercourse, m/m.

$$t_C = \frac{(4 + \sqrt{A}) + (1.5 \times L)}{0.8 \times \sqrt{H_M}} \quad (8)$$

where:

t_C = Time of concentration, hours;
 A = Watershed area, km²;
 L = Length of the main watercourse, km;
 H_M = Average height of the watershed, m.

2.4. Precipitation Analysis

The hydrological study of precipitation consisted of a probabilistic analysis of extreme events of short duration and high intensity that occurred in the Ribeira Brava watershed. The data used in this analysis were obtained through public sources such as the National Water Resources Information System (SNIRH), which provides precipitation data recorded automatically at precipitation stations. Regarding the probabilistic methodology adopted, the Gumbel Distribution was chosen because it showed a better fit with the data obtained and the expected projection for watersheds on the island of Madeira [25]. Therefore, the maximum annual daily precipitation can be calculated based on Equation (9).

$$P_{EST} = P_M + S' \times K_T \quad (9)$$

where:

P_{EST} = Estimated annual maximum daily precipitation, mm;
 P_M = Average annual precipitation, mm;
 S' = Sample standard deviation, mm;
 K_T = Frequency Factor, dimensionless.

where:

$$S' = \left(\frac{\sum (X_i - X_M)^2}{n'} \right)^{0.5} \quad (10)$$

where:

X_i = Sample value, mm;
 X_M = Sample mean, mm;
 n' = Number of samples.

$$K_T = -\frac{6^{0.5}}{\pi} \times \left\{ 0.577216 + \ln \left(\ln \left(\frac{T_R}{T_R - 1} \right) \right) \right\} \quad (11)$$

where:

T_R = Return period, years.

After determining the daily precipitation for an extreme phenomenon, the precipitation intensity with a specific duration can be obtained using Equation (12).

$$I = \frac{P_{EST} \times k}{t_C} \quad (12)$$

where:

I = Precipitation intensity, mm/h;

P_{EST} = Estimated annual maximum daily precipitation, mm;

t_C = Time of concentration, hours;

k = Time distribution coefficient, dimensionless.

where:

$$k = 0.181 \times \ln(t_C) + 0.4368 \quad (13)$$

where:

t_C = Time of concentration, hours.

The time distribution coefficient presents itself as a primary parameter because the annual maximum daily precipitation is only valid for events lasting 24 h. Therefore, because the duration of precipitation is equal to the time of concentration of the watershed, using the total amount of daily precipitation in the hydrologic analysis would cause oversizing of hydraulic structures [13,21]

2.5. Drainage Capacity of the River Mouth and Peak Flow Rate

The drainage capacity of the river mouth was calculated using the Manning–Strickler equation (Equation (14)), and then compared to the expected flow for an extreme event with a recurrence period of 100 years. To calculate the expected flow, empirical methodologies already widespread at the global level were used, namely: Forti (Equation (16)); Rational (Equation (17)); Giandotti (Equation (18)); and Mockus (Equation (19)).

$$Q_M = \left(\frac{1}{n}\right) \times A_M \times R^{\frac{2}{3}} \times \sqrt{i} \quad (14)$$

where:

Q_M = Drainage capacity of the river mouth, m³/s.

A_M = Area of the river mouth cross-section, m²;

R = Hydraulic radius, m;

i = Average slope of the river mouth region, m/m;

n = Roughness coefficient of the riverbed and walls, m^{-1/3} s, Appendix A Table A1.

where:

$$R = \frac{B + 2 \times h}{A_M} \quad (15)$$

where:

B = Width of the river mouth runoff section, m;

h = Height of the river mouth runoff section, m;

A_M = Area of the river mouth cross-section, m².

Both the width and height of the stream in the region of the mouth were obtained via previous studies in the region [21], and the first parameter was confirmed through the georeferencing process.

$$Q_{Forti} = A \times \left(b \times \frac{500}{125 + A}\right) + c \quad (16)$$

where:

Q_{Forti} = Peak flow rate by Forti, m³/s;

A = Watershed area, km²;

$b = 2.35$ for maximum daily precipitation below 200 mm and 3.25 for values above 200 mm;

$c = 0.5$ for maximum daily precipitation below 200 mm and 1 for values above 200 mm.

$$Q_{\text{Rational}} = \frac{C \times I \times A}{3.6} \quad (17)$$

where:

Q_{Rational} = Peak flow rate by the rational methodology, m^3/s ;

C = Surface runoff coefficient, Table A2;

I = Precipitation intensity, mm/h ;

A = Watershed area, km^2 .

$$Q_{\text{Giandotti}} = \frac{\lambda \times A \times P_{\text{MAX}}}{t_c} \quad (18)$$

where:

$Q_{\text{Giandotti}}$ = Peak flow rate by Giandotti, m^3/s ;

λ = Reduction coefficient, Table A3;

A = Watershed area, km^2 ;

P_{MAX} = Precipitation height for a duration equal to the concentration time, mm;

t_c = Concentration time, hours.

$$Q_{\text{Mockus}} = \frac{2.08 \times A \times P_{\text{EST}} \times C}{\sqrt{t_c} + 0.6 \times t_c} \quad (19)$$

where:

Q_{Mockus} = Peak flow rate by Mockus, m^3/s ;

A = Watershed area, km^2 ;

P_{EST} = Estimated precipitation, cm;

C = Surface runoff coefficient, Table A2;

t_c = Concentration time, hours.

When sizing hydraulic works, one of the main design criteria is to establish a Fill Rate value lower than 85%, as a means to determine a significant margin of error and thus ensure the safety of the population and their property [21,26]. Therefore, for the control of the flow to the river mouth, it is necessary to implement runoff regulation mechanisms, such as spillways.

As mentioned above, the Fill Rate value must be calculated using Equation (20) and, in case the runoff capacity of the river mouth is insufficient to drain the rain flow in the watershed and guarantee the established safety margin, it is necessary to size mitigation mechanisms, such as detention basins.

$$\text{FR} = \frac{Q_P}{Q_M} \times 100 \quad (20)$$

where:

FR = Fill Rate, %;

Q_P = Peak flow rate of each methodology, m^3/s ;

Q_M = Drainage capacity of the river mouth, m^3/s .

The Fill Rate parameter, in short, refers to the ability of a drainage section to drain a given flow. Thus, if the Fill Rate value is greater than 100%, it means that the section is no longer able to drain the entire volume of water without overflowing [21].

2.6. Detention Basin Sizing

As previously mentioned, if the runoff capacity of the river mouth is insufficient to drain all the rain and drained water to the mouth, it is necessary to size a spillway to regulate the flow that will reach the mouth, keeping the downstream flow below the

expected limit. For the present study, a Cipolletti type of spillway was chosen because this model has characteristics that reduce turbulence in the regions of contact with water and facilitate the runoff [4,5,13]. The sizing of this spillway can be determined using Equation (21).

After the definition and regularization of the flow that will be drained to the river mouth, it is possible to estimate the volume of water that will be retained along the detention basin. For this purpose, two methodologies were used, namely, the Dutch Method (Equation (22)) and the Simplified Triangular Hydrograph (STH; Equation (23)).

$$Q_S = 1.86 \times L_{SD} \times H_D^{1.5} \tag{21}$$

where:

Q_S = Flow drained by spillway, m^3/s ;

L_{SD} = Width of the sill, m^3/s ;

H_D = Height of the waterline above the sill, m .

$$V_A = (Q_P - Q_S) \times t_C \times 3600 \tag{22}$$

$$V_A = \frac{(Q_P - Q_S) \times (2 \times t_C - 2 \times [Q_S / \{Q_P / t_C\}])}{2} \tag{23}$$

where:

V_A = Storage Volume, m^3 ;

Q_P = Peak flow rate of each methodology, m^3/s ;

Q_S = Flow drained by the spillway, m^3/s ;

t_C = Concentration time, hours.

Note that Equation (23) was formulated based on the geometric analysis of the STH (Figure A1), considering an event with a total duration of at least twice the concentration time of the watershed under study. This consideration took into account that the last rainwater particle to arrive at the river mouth would occur at the last instant of precipitation and in the farthest region, which suggests that it would need a value equal to the time of concentration to be considered for the amount drained by the mouth [21].

The methodologies were chosen because the Dutch Method does not contemplate the delay and damping of the precipitation hydrograph, causing a certain oversizing of the structure [27], as shown in Figure 4, where q_s : runoff capacity of the spillway; t_c : concentration time; t_{MAX} : maximum precipitation duration (base); t_d : time delay until the beginning of water accumulation in the detention basin; $H_{a,MAX}$: maximum storage capacity; $i(t_{MAX})$: precipitation intensity corresponding to the maximum duration.

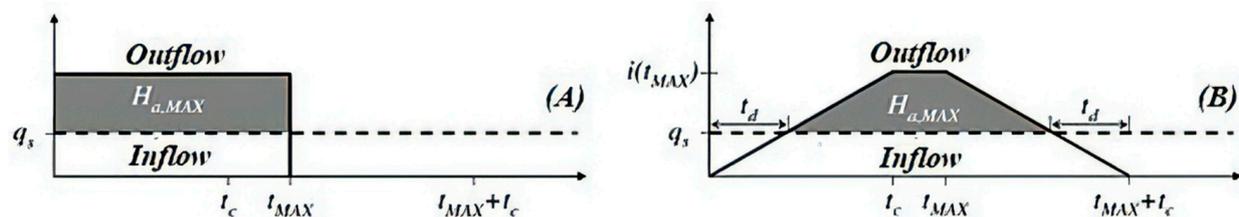


Figure 4. (A) Dutch method; (B) STH method (Source: [27]).

Therefore, it was verified that, in the Dutch Method, storage begins simultaneously with precipitation, which does not correspond to reality because storage will only begin when the flow drained downstream is greater than the spillway’s runoff capacity.

2.7. Modification of the Roughness Coefficient

In addition, a structural mitigation measure taken into consideration was the modification of the roughness coefficient of the walls and riverbed of the watercourse, thus avoiding the reduction of drainage capacity due to friction. This methodology consists

of changing the value of the parameter “n” in the Manning-Strickler equation in order to improve the flow of a given watercourse by considering another material for the coverage of the stream wall and riverbed [21].

3. Results

The results presented here correspond to the data obtained by applying the formulas described above. Therefore, to evaluate the morphometric characteristics of the main watercourse of Ribeira Brava, an individual analysis of each of the parameters listed in Table 1 was undertaken, correlating them with reference values suggested in several bibliographies.

Table 1. Parameters calculated or extracted from ArcGIS.

Parameter	Unit of Measurement	Value
Area	km ²	41.059
Perimeter	km	42.430
Length of Main Watercourse	km	14.619
Maximum Height of Main Watercourse	m	1316.030
Minimum Height of Main Watercourse	m	0.000
Average Concentration Time	hours	2.362
Gravelius Coefficient of Compactness	dimensionless	1.868
Elongation Factor	dimensionless	8.856
Shape Factor	dimensionless	0.298
Number of Watercourses	units	1668.000
Average Bifurcation Ratio	dimensionless	4.286
Strahler Classification	dimensionless	6.000

The first parameter to be analyzed concerns the watershed area, which has a primary role in the analysis of the volume of water drained to the river mouth. This parameter can be classified as: Very Large > 20 km²; Large > 10 km²; Medium > 1 km² and Small < 1 km² [28]. In this sense, as can be seen in the previous table, the watershed under study has a “Very Large” classification, indicating a higher propensity to flood compared to smaller watersheds. However, it is noted that the aforementioned reference values are arbitrary and may differ according to the type of analysis to be performed [28], and the propensity to flood.

As shown in Figure 5, the Ribeira Brava watershed has borders with significantly higher altitudes than the central region, which denotes a steep slope that tends to supply the main watercourse quickly and thus increase the volume of water present in the stream that, consequently, will be redirected to the river mouth.

Regarding the drainage system of the watershed under study, as shown in Figure 6, the high number of watercourses indicates a high drainage capacity, i.e., the basin has many low- and medium-order watercourses that supply the main watercourse. Moreover, this index represents the hydrographic behavior of a given area, where the fundamental aspect is its propensity to generate new watercourses. Therefore, basins with high hydric density have a tendency to present a greater number of ephemeral channels, precisely due to the ability to generate new watercourses [21,23].

The analysis of precipitation was based on data provided by the National Information System on Water Resources (SNIRH) [29], through sample data from a period of sixteen years, presented in Table A4 and Figure A2. Thus, through the probabilistic processing of the Gumbel Distribution, the values presented in Table 2 were obtained.

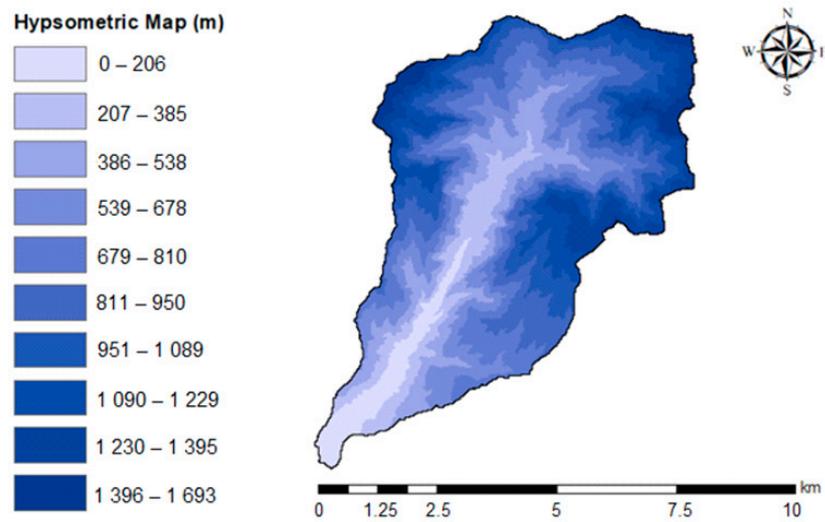


Figure 5. Hypsometric map—DEM file (Source: Authors by ESRI ArcGIS, 2020).

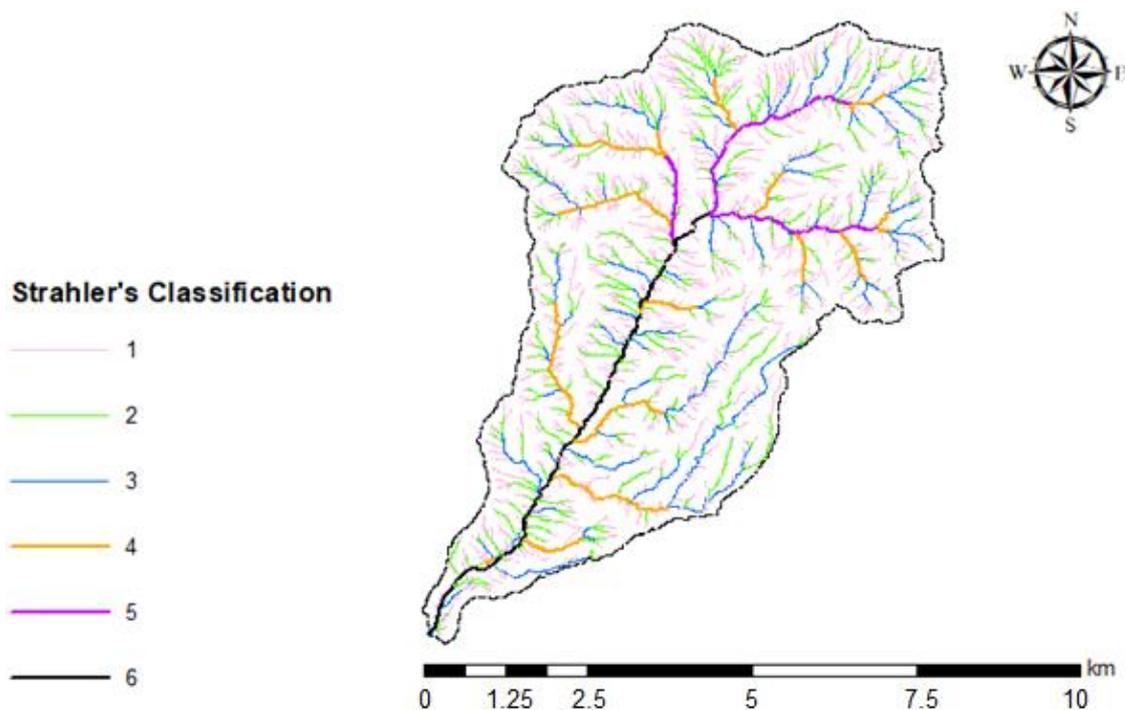


Figure 6. Strahler classification (Source: Authors by ESRI ArcGIS, 2020).

Table 2. Precipitation parameters.

Parameter	Symbol	Unit of Measurement	Value
Average Annual Precipitation	P_M	mm	164.443
Standard Deviation	S'	mm	64.424
Frequency Factor	K_T	dimensionless	3.136
Time Distribution Coefficient	k	dimensionless	0.592
Annual Maximum Daily Precipitation	P_{EST}	mm	366.521
Precipitation Intensity	I	mm/h	91.920

After determining the precipitation intensity estimated for a recurrence time of 100 years, the calculation of the peak flow rates was undertaken, as shown in Table 3,

through the formulas and methodologies indicated in the previous section. The surface drainage coefficient used in the rational methodology corresponds to the value of 0.500 (Table 4) because the region under study is a peripheral region with commercial buildings. This value corresponds, basically, to the percentage of water that tends to drain superficially, i.e., 50% of the total precipitation.

Table 3. Peak flow rate.

Methodology	Flow (m ³ /s)
Forti	442.849
Rational	531.782
Giandotti	560.848
Mockus	529.792

Table 4. Surface drainage coefficient adopted (Source: [30]).

Urban Areas		
Land Occupation		Surface Drainage Coefficient
Commercial Area	City Center	0.700–0.950
	Peripheral Areas	0.500–0.700

The value of the reduction coefficient (λ) used in the calculation of the flow using Giandotti's methodology is shown in Table 5.

Table 5. Adopted Giandotti's reduction coefficient (Source: [31]).

Area (km ²)	λ	Equivalent "C"
<300	0.346	1.250

Regarding the analysis of the drainage capacity of the river mouth, the Manning–Strickler equation was used to verify the need to implement a detention basin, where the values obtained are summarized in Table 6. However, it is necessary to emphasize that the stream walls and bed have different roughness coefficients. Therefore, the drainage capacity of the river mouth was calculated by means of the weighted mean of the respective coefficients, where the stone and mortar walls are in good condition ($n = 0.020$) and the riverbed consists of a stony and vegetated surface in poor condition ($n = 0.040$). Another important factor to note is the very low slope in the river mouth region, which tends to decrease the water flow velocity and, consequently, the drainage capacity of the section. As a means to simulate a critical situation, it was opted to take into account a slope of 0.01 m/m in the reference section.

Table 6. Assessment of the need for detention basin implementation.

Parameter	Unit of Measurement	Value
Width of the River Mouth	m	20.000
Height of the River Mouth	m	4.500
Drainage Capacity of the River Mouth	m ³ /s	566.645
Fill Rate—Forti (pre-regularization)	%	78
Fill Rate—Rational (pre-regularization)	%	94
Fill Rate—Giandotti (pre-regularization)	%	99
Fill Rate—Mockus (pre-regularization)	%	93

As shown in Table 6, the Fill Rate is higher than the established limit of 85% for the Rational, Giandotti, and Mockus methods, which indicates the need to implement mitigation and flow control measures for the river mouth region. Therefore, a detention

basin was sized with the flows found in the aforementioned methodologies, considering the urban and spatial limitations due to already existing infrastructures near the stream.

Because the sizing of the detention basin depends on the flow that exceeds the limit established for the river mouth, first, a Cipolletti trapezoid spillway was sized for the regularization and control of the flow that will drain downstream. The characteristics of the spillway are presented in Table 7.

Table 7. Application of the Cipolletti spillway.

Parameter	Unit of Measurement	Value
Width of the Spillway	m	18.000
Height of the Spillway Sill	m	4.500
Spillway Outflow	m ³ /s	319.598
Fill Rate—Rational (post-regularization)	%	56
Fill Rate—Giandotti (post-regularization)	%	56
Fill Rate—Mockus (post-regularization)	%	56

A posteriori, the detention basins were sized using the Dutch Method and the STH, which are simplified methodologies that neglect several factors and, thus, can result in an oversizing of the hydraulic structure. Additionally, the height and width of the detention basin were fixed with the same values of the existing cross-section, as a means to reduce the environmental and urban impacts of the implementation works of this measure. Therefore, the only geometric variable of the detention basin was its length, which is limited by the total length of the main watercourse.

After making the calculations with the aforementioned methodologies, the values shown in Table 8 were found.

Table 8. Detention basin sizing.

Parameter	Unit of Measurement	Value
Width	m	20.000
Height	m	4.500
Length—Dutch Method (Rational)	m	20,047.874
Length—STH Method (Rational)	m	7999.198
Length—Dutch Method (Giandotti)	m	22,794.149
Length—STH Method (Giandotti)	m	9804.940
Length—Dutch Method (Mockus)	m	19,859.934
Length—STH Method (Mockus)	m	7879.396

Finally, the modification of the roughness coefficient was used as an alternative measure to mitigate the impacts of the flood, while maintaining the characteristics of the riverbed vegetation. In this context, the values shown in Table 9 correspond, more precisely, to the improvement in the condition of conservation of the river bed, as a means to reduce the loss of drainage capacity caused by excessive friction between the fluid and the covering material.

Table 9. Modification of the roughness coefficient.

Parameter	Unit of Measurement	Value
Wall Roughness Coefficient—Modified	m ^{-1/3}	0.012
Riverbed Roughness Coefficient—Modified	m ^{-1/3}	0.030
Drainage Capacity of the River Mouth—Modified	m ³ /s	488.881
Fill Rate—Rational (post-modification)	%	68
Fill Rate—Giandotti (post-modification)	%	72
Fill Rate—Mockus (post-modification)	%	68

In short, the modified roughness coefficients of the walls correspond to the surface with concrete finishing in good condition, whereas the riverbed remains with the stony and vegetated characteristic, albeit in good condition. The values used for these coefficients are summarized in Table 10.

Table 10. Adopted roughness coefficient (Source: [31]).

Channel Typology	Very Good	Good	Regular	Bad
Channel with stony and vegetated slope	0.025	0.030	0.035	0.040
Surface with concrete finishing	0.011	0.012	0.013	0.015

4. Discussion

Because the primary objective of this research was to verify the need to implement simplified measures to mitigate the impacts of floods in the Ribeira Brava watershed, the use of the detention basin proved to be effective in controlling the flow at the river mouth, which can be characterized as a structural measure [32]. Initially, the Fill Rate values were 94%, 99%, and 93% for the Rational, Giandotti, and Mockus methodologies, respectively, and fell to 56% after the implementation of the measure. Therefore, it was evidenced that the chosen mitigation measure allows the river mouth to work below the 85% threshold, as indicated earlier. Furthermore, this study corroborates the flood risk analysis prepared by the Regional Directorate for Territorial Ordering and Environment (DROTA), as shown in Table 11, indicating an acceptable accuracy for the present study.

Table 11. Watersheds with high flood risk. (Source: [33]).

Municipality	Watershed
Ribeira Brava	Ribeira Brava Tabua

The proposal of this research aimed to cause the least possible impact on the existing waterway and its surroundings because the presence of natural elements and values in cities is now a fundamental condition for the environmental recovery of the urban territory [34]. Moreover, natural and urban systems are coexistent components in the same space, and their integrated management is a primary requirement of regional space and a condition for achieving the goal of sustainability in territories and cities [35,36]. Alternatively, urban growth or development can occur in a disorganized manner, creating urban voids [37].

Thus, it was decided not to change the dimensions of the cross-section of the streams, both in width and height, and the length was the only dimensional variable. Based on this assumption, the Dutch Method presented considerable oversizing, because the total length of the detention basin was found to be greater than the total length of the main waterway, which denotes the need to change one more of the dimensions, i.e., height or width. In this context, despite the efficiency in flow regularization, the Dutch Method is not applicable when considering the urban conditions previously imposed.

For the STH method, the same conditions were imposed. However, the methodology showed applicability because the total length of the detention basin is shorter than the length of the main watercourse.

With regard to the change in the roughness coefficient of the stream, it was decided to remain with the stony and vegetated characteristics of the riverbed, and to only improve the conditions of the state of conservation. This option was chosen because the total removal of stones, sediments, and vegetation from the riverbed would be a costly, time-consuming, and frequent process. For the walls, maintenance should not be frequent, because wear by abrasion would occur exclusively in an alluvial channel that tends to drain a high volume of water and large granular sediments.

Despite being a simple measure, the modification of the roughness coefficient of the stream presented considerable effectiveness, allowing the river mouth to work below the

pre-established filling limit. Furthermore, it should be highlighted that both applicable methodologies—i.e., the STH method and roughness coefficient modification—can be implemented together, in order to reduce the length required for the detention basin and, thus, optimizing the sizing performed.

It should be noted that the methodologies used are simplified in nature, i.e., they do not consider local peculiarities. Therefore, this causes the results to have a very high safety margin and, consequently, causes the oversizing of hydraulic structures.

5. Conclusions

The results obtained in this study indicate that the Ribeira Brava watershed is susceptible to floods during an extreme precipitation event, which is reinforced by the Flood Risk Report prepared by DROTA. This occurs because the stream bed has characteristics that are very unfavorable to surface runoff, because it has a surface with a significant presence of stones and vegetation. These characteristics of the riverbed reduce the velocity of the water flow and, consequently, reduce the drainage capacity, especially in areas with very low slope, such as the river mouth. Furthermore, the insufficient drainage capacity of the river mouth was demonstrated by 3/4 of the methodologies employed: Rational, Giandotti, and Mockus.

Regarding the proposed simplified solutions, the Dutch Method did not present an applicability coherent with the pre-established urban premises because it suggests very high lengths for the detention basin, i.e., higher than the length of the main watercourse. By comparison, the Simplified Triangular Hydrograph Method presents satisfactory results because it allows the implementation of the detention basin without changing the height or width of the stream.

Finally, the change in the roughness coefficient also presented satisfactory and effective results for the mitigation of flood impacts because it is a relatively simple measure to implement and still meets the demand of the watershed under study.

Due to the impossibility of considering all the aspects that make up a more complete and effective analysis in this study, other analyses can be carried out in order to complement or optimize the results presented here. These include the analysis of the drainage capacity of the implemented urban hydraulic system, in order to reduce the storage volume of the detention basins; analysis of sediment deposition according to the entrainment velocity present in the main watercourse [38]; verification of the deterioration of the artificial canal walls by abrasion, and the analysis of the maximum time to proceed with maintenance (silting and desilting processes); influence of the degradation of the artificial water channels according to the quality of the water discharged by its tributaries [39,40]; analysis of the urban growth perspective of the studied municipalities and its influence on the flow increase; budgeting for the implementation of the mitigation measures of this study; analysis of the influence of the tide level on the drainage in artificial water channels, and the direct relationship with the risk of downstream flooding; and characterizing the influence of artificial water channels in territorial planning processes—i.e., adaptation to rural watersheds.

The obtained findings corroborate the ideas and conclusions proposed in similar studies that used case study analysis and simulations as drivers for scientific development [41,42].

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Appendix A

Table A1. Manning-Strickler roughness coefficients (Source: [31]).

Type of Channel and Description	Very Good	Good	Regular	Bad
Mortared stone masonry	0.017	0.020	0.025	0.030
Rigged stone masonry	0.013	0.014	0.015	0.017
Dry stone masonry	0.025	0.033	0.033	0.035
Brick masonry	0.012	0.013	0.015	0.017
Smooth metal gutters (semicircular)	0.011	0.012	0.013	0.016
Open channels in rock (irregular)	0.035	0.040	0.045	-
Channels with bottom on land and slope with stones	0.028	0.030	0.033	0.035
Channels with stony bed and vegetated slope	0.025	0.030	0.035	0.040
Channels with concrete coating	0.012	0.014	0.016	0.018
Earth channels (rectilinear and uniform)	0.017	0.020	0.023	0.025
Dredged canals	0.025	0.028	0.030	0.033
Clay conduits (drainage)	0.011	0.012	0.014	0.017
Vitrified clay conduits (sewage)	0.011	0.013	0.015	0.017
Flattened wooden plank conduits	0.010	0.012	0.013	0.014
Gabion	0.022	0.030	0.035	-
Cement mortar surfaces	0.011	0.012	0.013	0.015
Smoothed cement surfaces	0.010	0.011	0.012	0.013
Cast iron coated tube with tar	0.011	0.012	0.013	-
Uncoated cast iron pipe	0.012	0.013	0.014	0.015
Brass or glass tubes	0.009	0.010	0.012	0.013
Concrete pipes	0.012	0.013	0.015	0.016
Galvanized iron pipes	0.013	0.014	0.015	0.017
Rectilinear and uniform clean streams and rivers	0.025	0.028	0.030	0.033
Streams and rivers cleared rectilinear and uniform with stones and vegetation	0.030	0.033	0.035	0.040
Streams and rivers cleared rectilinear and uniform with intricacies and wells	0.035	0.040	0.045	0.050
Spread margins with little vegetation	0.050	0.060	0.070	0.080
Spread margins with lots of vegetation	0.075	0.100	0.125	0.150

Table A2. Surface runoff coefficients (Source: [30]).

Urban Areas		
Land Occupation	Surface Runoff Coefficient	
Green Areas	Lawns in sandy soils	0.050–0.200
	Lawns on heavy soils	0.150–0.350
	Parks and cemeteries	0.100–0.350
	Sports fields	0.200–0.350
Commercial Areas	City district	0.700–0.950
	Periphery	0.500–0.700
Residential Areas	Town-center villas	0.300–0.500
	Villas on the outskirts	0.250–0.400
	Apartment buildings	0.500–0.700
Industrial Areas	Dispersed industry	0.500–0.800
	Concentrated industry	0.600–0.900
Railways		0.200–0.400
Streets and Roads	Paved	0.700–0.900
	Concrete	0.800–0.950
	In brick	0.700–0.850

Table A3. Giandotti reduction coefficients (Source: [31]).

A (km ²)	λ	"C" Equivalent
<300	0.346	1.250
300–500	0.277	1.000
500–1000	0.197	0.710
1000–8000	0.100	0.360
8000–20,000	0.076	0.270
20,000–70,000	0.055	0.200

Table A4. Precipitation historical data (Source: [29]).

n	Year	(mm)
1	1998/1999	170.000
2	1999/2000	180.700
3	2000/2001	135.000
4	2001/2002	190.000
5	2002/2003	195.400
6	2003/2004	141.000
7	2004/2005	103.200
8	2005/2006	91.400
9	2006/2007	141.400
10	2007/2008	104.600
11	2008/2009	155.000
12	2009/2010	257.800
13	2010/2011	148.400
14	2011/2012	288.600
15	2012/2013	267.400
16	2013/2014	61.200

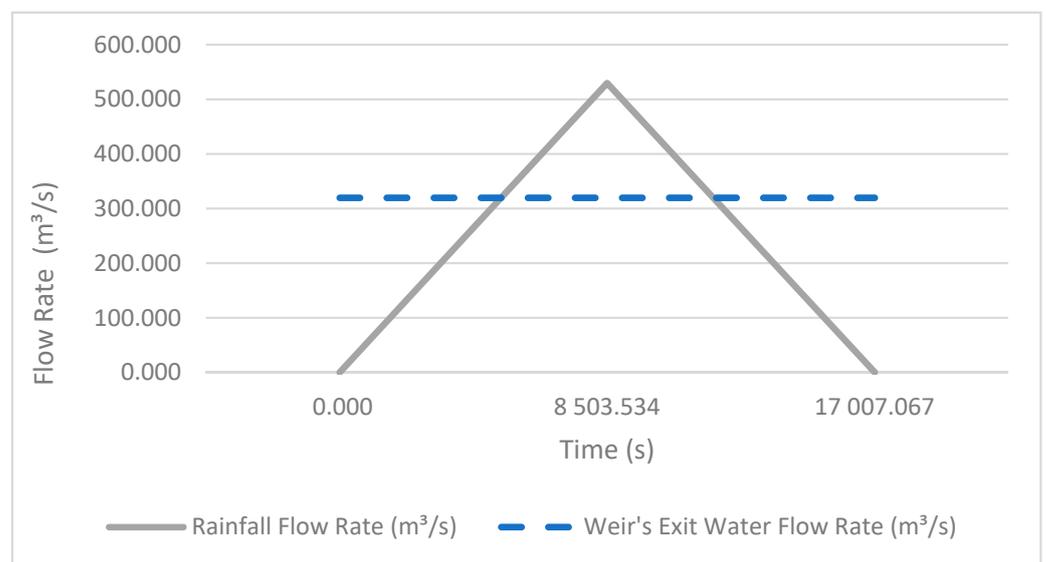


Figure A1. Ternary phase diagram.

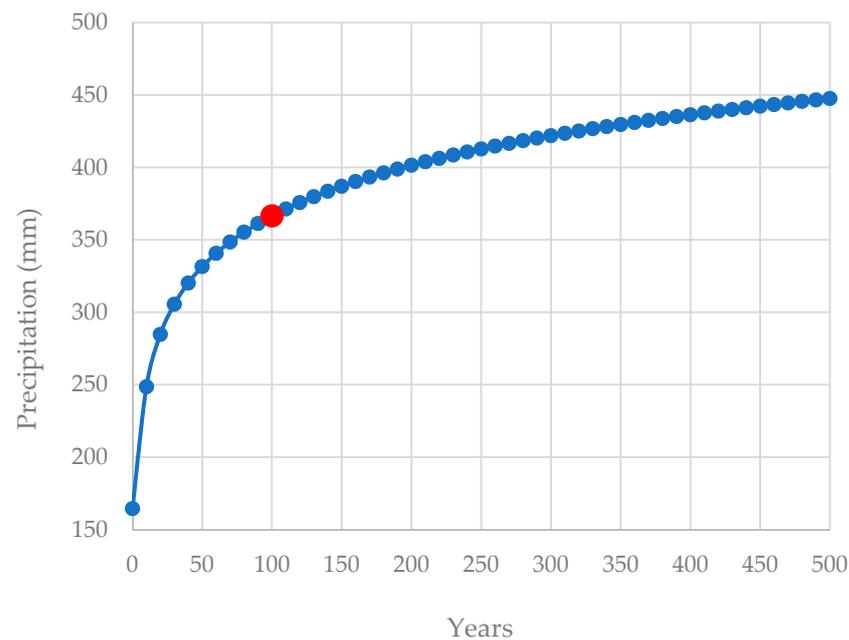


Figure A2. Expected rainfall for Ribeira Brava's watershed.

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