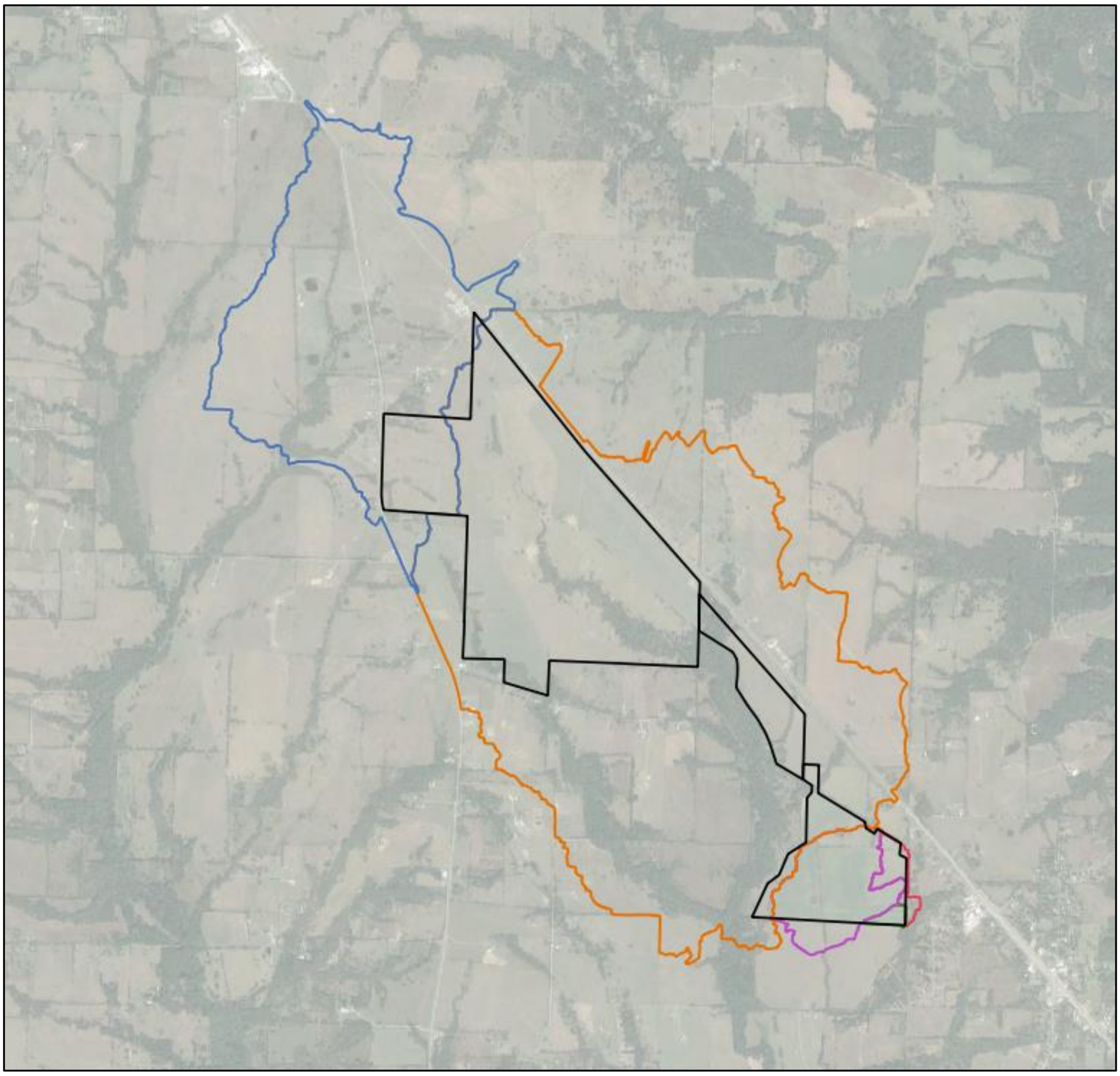
HYDROLOGICAL AND FLOOD STUDY

PV BELLTOWN - BARRETT SOLAR, TEXAS (USA)



INDEX

1. INTRODUCTION AND SCOPE

This hydrological and flood study was developed by ZUBÍA INGENIEROS as a request from COBRA for the construction of "PV BELLTOWN - BARRETT SOLAR" plant, located 84.9 km west of the city of Dallas (USA).

The scope of this study is to determine the basins that directly affect the plots and the expected flow for a return period of 100 years. In this way, we proceed to develop a qualitative and quantitative estimate of the flood risk presented by the photovoltaic plant during its service life.

2. DESCRIPTION OF THE AREA AND ITS HYDROGRAPHIC NETWORK

The Barrett Solar Photovoltaic Plant consists of an area of 369.19 ha distributed in three plots located in Rains County, in the east part of the state of Texas, in southern USA. The plots are located in a rural area 84.9 km west of the city of Dallas.

The geographical coordinates of Barrett Solar are as follows:

- Latitude: 32°57'16.0"N
- Length: 95°54'18.3"W

Below is an image of the location of the plant in relation to the city of Dallas.

Figure 1. Location of Barrett Solar.

As for the hydrographic network present in the Barrett Solar area, the northern plot is crossed by the Cedar Creek. This watercourse and the rest of the existing water runoff in the study area belong to the watershed of the Sabine River.

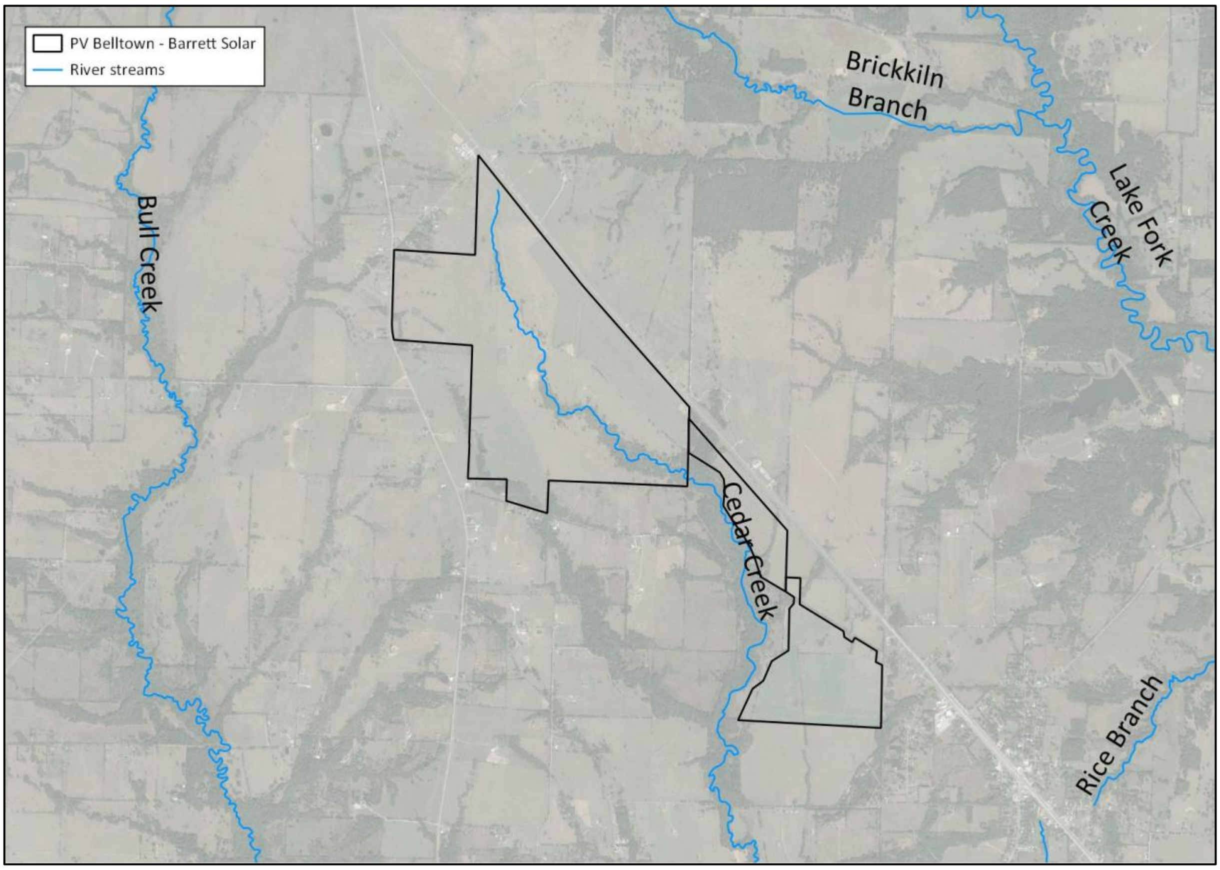


Figure 2. Hydrographic network in the area of Barrett Solar.

The plots are located on a gentle slope that descends in a south direction. The altitude range in which the plots are located varies between 147 and 169 m approximately.

3. HYDROLOGICAL STUDY

The "*Annex 1 Climatological and Hydrological Study*" is included to provide a general description of the climatology of the plant area and to determine the hydrological parameters of the study basins.

Once the water network has been identified, we proceed to calculate the flow rates of the watercourses intercepted by the Photovoltaic Park. For this purpose, the external and internal basins are defined, obtaining the flows for the different return periods. In this case, the Barrett Solar plots are under the influence of five watersheds, which are shown below:

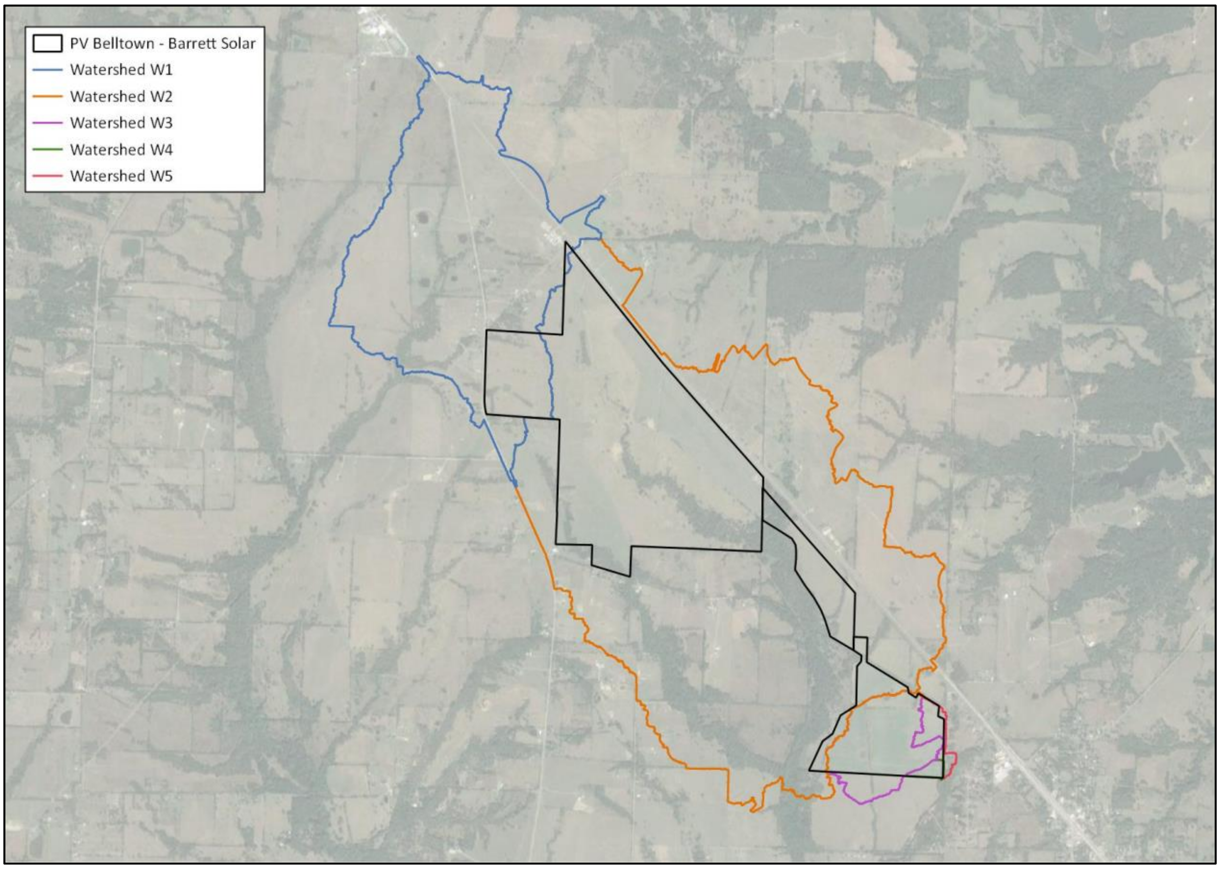


Figure 3. Study basins and study area.

The physical characteristics of the study basins are as follows:

Table 1. Characteristics of the study basins.

Name	Area (km^2)	Hmax (m)	Hmin (m)	Difference in height (m)	Channel length (km)	Slope %	Tc (h)
Watershed W1	3.34	171.38	148.14	23.23	4.34	0.53%	2.48
Watershed W2	8.43	170.56	142.73	27.83	9.50	0.29%	5.03
Watershed W3	0.49	155.22	146.18	9.05	2.17	0.42%	1.53
Watershed W4	0.03	150.57	148.74	1.83	0.39	0.47%	0.41
Watershed W5	0.08	155.29	147.64	7.66	0.88	0.87%	0.67

3.1. SCS METHOD METHODOLOGY

In this case, to obtain the hydrological parameters of each study basin, the method of superposition of unit hydrographs of the SCS of the United States was applied. In order to use this method, a methodology described in detail in "*Annex 1 Climatological and Hydrological Study*" has been applied.

The return period of study is 100 years.

3.2. PEAK FLOW RESULTS

The peak flows obtained, as detailed in "*Annex 1 Climatological and Hydrological Study*", are the following, obtained with the method of superposition of unit hydrographs of the SCS of the United States:

Table 2. Summary of peak flows for each basin and return period using the U.S. SCS unit hydrograph superposition method.

Watershed	Q_p (m^3/s) for T-100 years
Watershed W1	14.886
Watershed W2	24.922
Watershed W3	2.766
Watershed W4	0.288
Watershed W5	0.713

4. FLOOD STUDY

The "*Annex 2. Flood Study*" is included to determine the variation of flood risks in the area of the Barrett Solar Plant and downstream of it.

The process followed, as described in the annex, is as follows:

- Generation of a 2D flood analysis model, carried out with the IBER program, version 3.2.2. This model will analyze the scope of the study plots in a sufficient area, both upstream, so that there is no influence caused by the concentration of inflow at a single point, and downstream, so that the modeled sinkholes do not affect the flood zone near the plots or can generate a backwater. The results obtained from the model are depths, velocities, elevations and specific discharge.
- The scope of the study plots is modeled in the program, and their flood risk is obtained by introducing a hyetograph of alternating blocks of precipitation with a total duration of 24 hours subdivided into blocks of 1 hour. This hyetograph is obtained by multiplying the design intensity extracted from the IDF curves calculated in the "*Annex 1 Climatological and Hydrological Study*". In the IBER model itself, the curve numbers corresponding to the study area are input so that the program will obtain the corresponding runoff.

From the model made with the IBER software, it has been possible to determine the flood level of the study plots for the return period of 100 years.

In terms of depths, the greatest impact is on the watercourse that crosses the central part of the northern plot, where depths up to 2.78 m occur. In the rest of the area there are depths that generally remain below 0.50 m and are distributed along its surface.

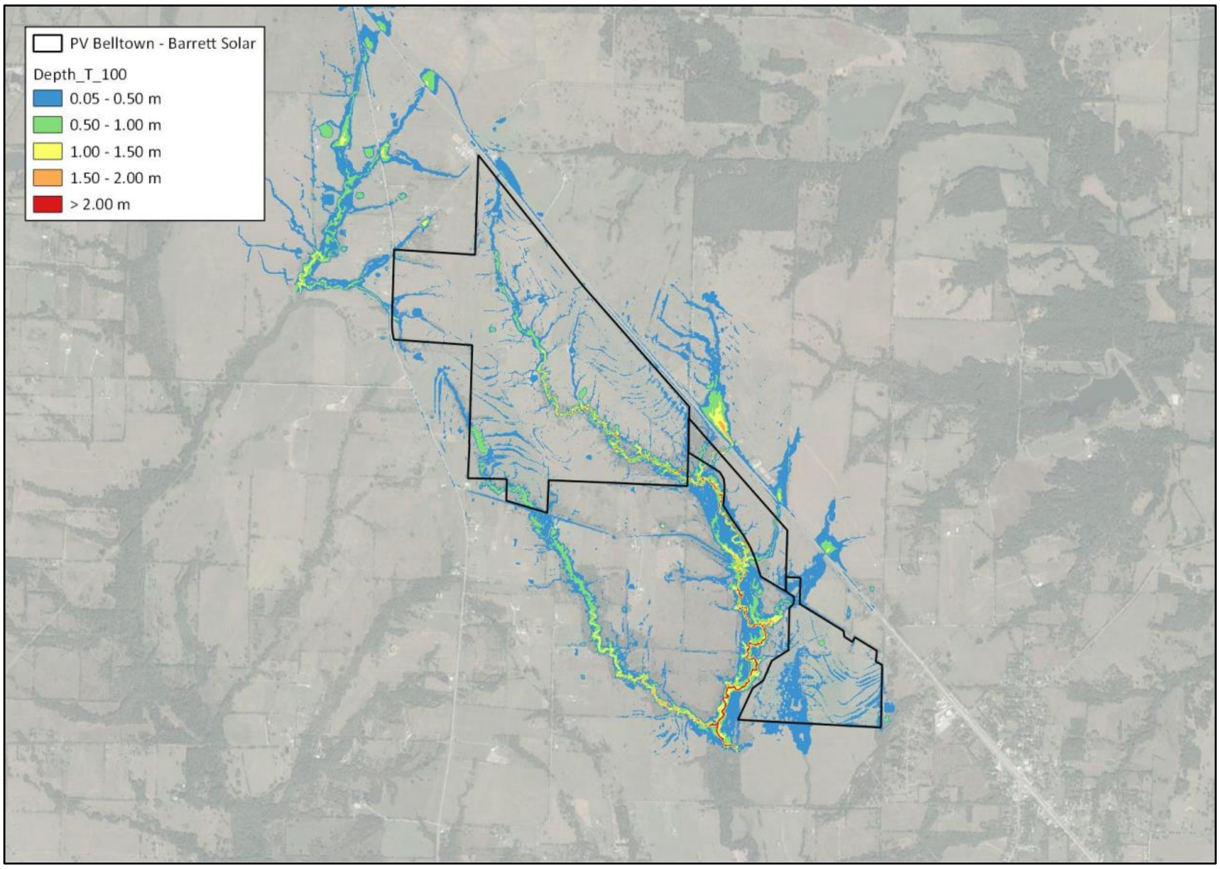


Figure 4. Maximum flooding depths obtained in each cell for a return period of 100 years.

Regarding flood velocities, the most affected areas are around the watercourse that crosses the central part of the northern plot, where maximum velocities of 3.95 m/s are obtained. Despite this maximum value, the most common velocity values are low, generally below 0.50 m/s in the case of streams and 0.25 m/s outside them.

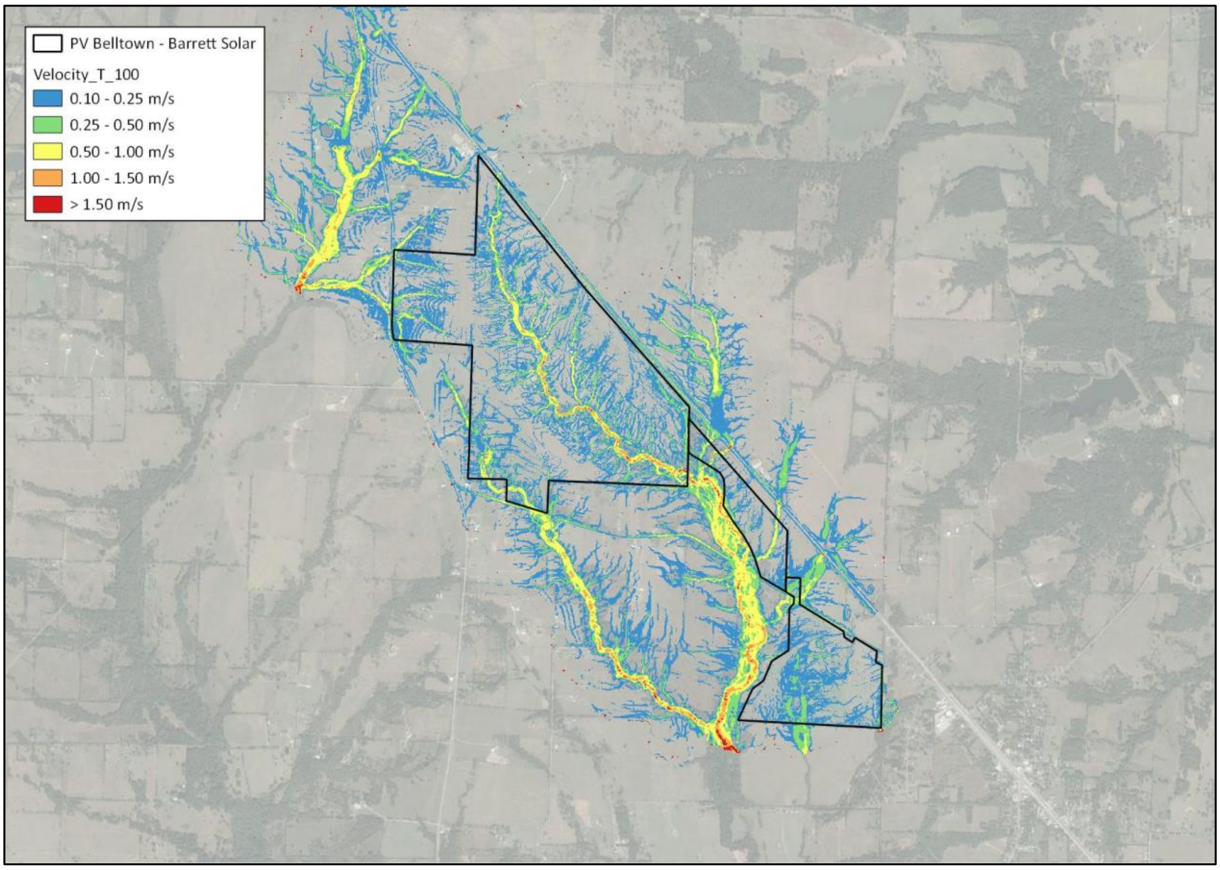


Figure 5. Maximum flood velocities obtained in each cell for a return period of 100 years.

5. CONCLUSIONS

This study was conducted in two phases:

- Hydrological study.
- Flood study.

Each of the phases is developed in an Annex and the drawings corresponding to the results obtained in each document are included.

The calculations are made for an unaltered land configuration, as it is before any action related to the photovoltaic park is carried out. Any alteration of the land must be carried out respecting basins and watercourses, so as not to generate variations in them.

Vigo, July 6, 2023

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ANNEX 1. CLIMATOLOGICAL AND HYDROLOGICAL STUDY

INDEX

1. INTRODUCTION

This "*Annex 1 Climatological and Hydrological Study*" is made at the request of COBRA and is intended to develop in detail the calculation of the flow calculation of the external and internal basins of the "PV BELLTOWN - BARRETT SOLAR" plant, located 84.9 km west of the city of Dallas (USA).

2. STUDY AREA

The Barrett Solar Photovoltaic Plant consists of an area of 369.19 ha distributed in three plots located in Rains County, in the east part of the state of Texas, in southern USA. The plots are located in a rural area 84.9 km west of the city of Dallas.

The geographical coordinates of Barrett Solar are as follows:

- Latitude: 32°57'16.0"N
- Length: 95°54'18.3"W

Below is an image of the location of the plant in relation to the city of Dallas.

Figure 1. Location of Barrett Solar.

3. CLIMATOLOGICAL ANALYSIS

The study plots have an *Cfa* climate according to the Köppen-Geiger climate classification. This climate is known as hot humid subtropical climate and tends to be located on the southeast side of all continents (except Antarctica), generally between latitudes 25° and 40° and are located poleward from adjacent tropical climates. It is also known as warm temperate climate in some climate classifications.

Average monthly summer temperatures typically range between 24 and 27 °C and average monthly winter temperatures are mild, typically averaging 7.5 to 16 °C.

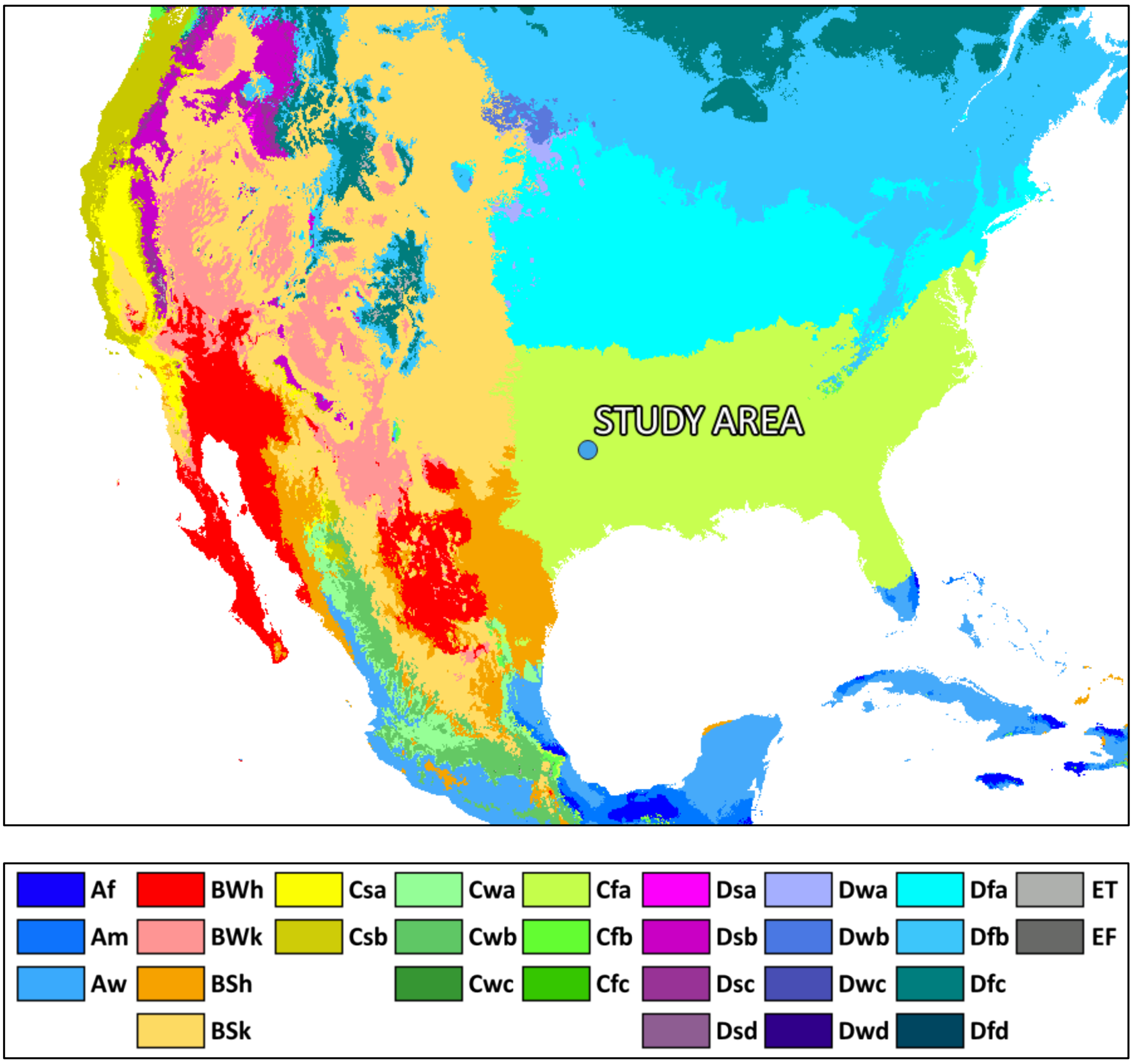

Figure 2. Köppen-Geiger climate classification.

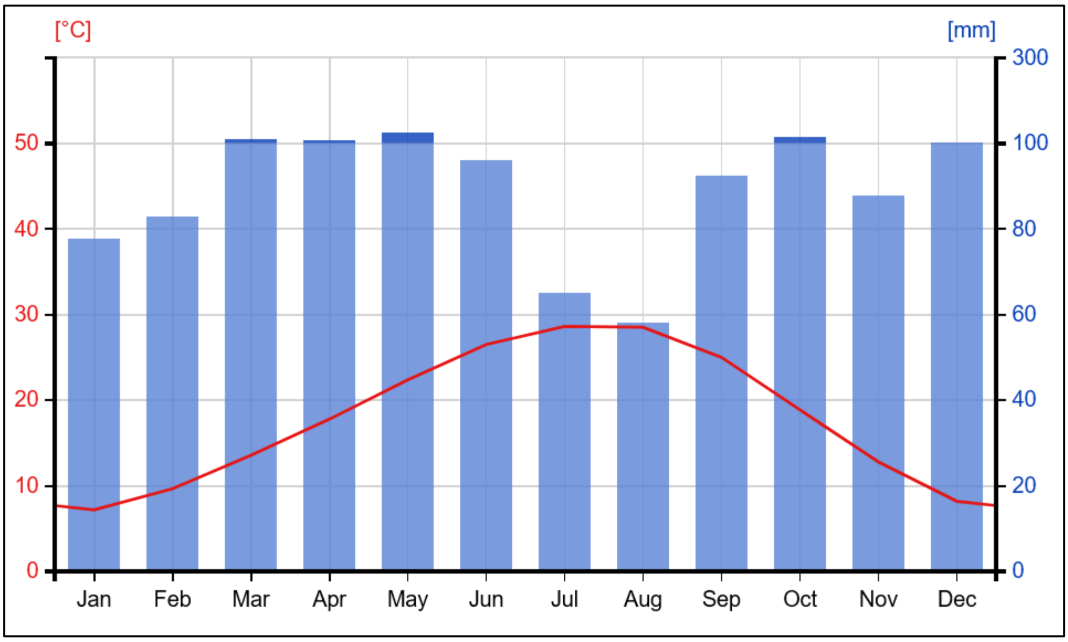

Figure 3. Climograph of the study area. Source: https://climatecharts.net/.

In the case of the study area, as can be seen in the climograph in Figure 3, the distribution of temperatures is within what is expected for the *Cfa* climate with average monthly temperatures around 30° in summer and 10° in winter.

4. METHODOLOGY FOR HYDROLOGICAL CALCULATIONS

In order to develop the methodology for determining the maximum flood flows in the basins, the Soil Conservation Service (SCS) unit hydrograph superposition method was used. This method uses the SCS Dimensionless Unit Hydrograph based on the analysis of large number of watersheds. The X-axis consists of dimensionless time units and Y-axis consists of dimensionless discharge units. The dimensionless UH is very useful for constructing a synthetic unit hydrograph for a wide variety of watersheds. Dimensionless unit hydrographs based on a study of a large number of unit hydrographs are recommended by various agencies to facilitate construction of synthetic unit hydrographs.

The return period studied in this hydrological study is 100 years.

5. DELIMITATION AND PHYSICAL CHARACTERISTICS OF THE WATERSHEDS

The watersheds are delimited with software that allows them to be extracted from terrain elevation files. Thanks to QGIS and GRASS software tools, the limits of the basins that affect the study area can be obtained. For this purpose, the following Digital Elevation Model (DEM) has been used as a basis:

- Digital Elevation Model with 1x1 m pixel resolution extracted from the East Texas Lidar project. This lidar collection covers portions of east Texas. This project was managed by the Texas Strategic Mapping (StratMap) Program and utilized the StratMap contracts held by the Department of Information Resources (DIR). Collection took place December 29, 2016, through April 14, 2017. Data was acquired and processed by Fugro EarthData, Inc. with third party quality assurance/quality control by AECOM.

Once the basins have been represented, their physical characteristics are deduced: surface area, length, extreme elevations and slope of the main course. As for the time of concentration, the formula proposed by Témez has been used, which is also used in the Spanish Instruction 5.2-IC. The use of another formulation would produce small differences in the time of concentration, and the effect on the results would be very limited for practical purposes.

The expression to be used is as follows:

$$T_C = 0{,}3\left[\frac{L}{J^{\frac{1}{4}}}\right]^{0{,}76}$$

being:

- L = length of main channel, in km.
- J = slope, in tenths.

In the case of the present study, the Barrett Solar plots are under the influence of five watersheds (W1 to W5), which are shown below:

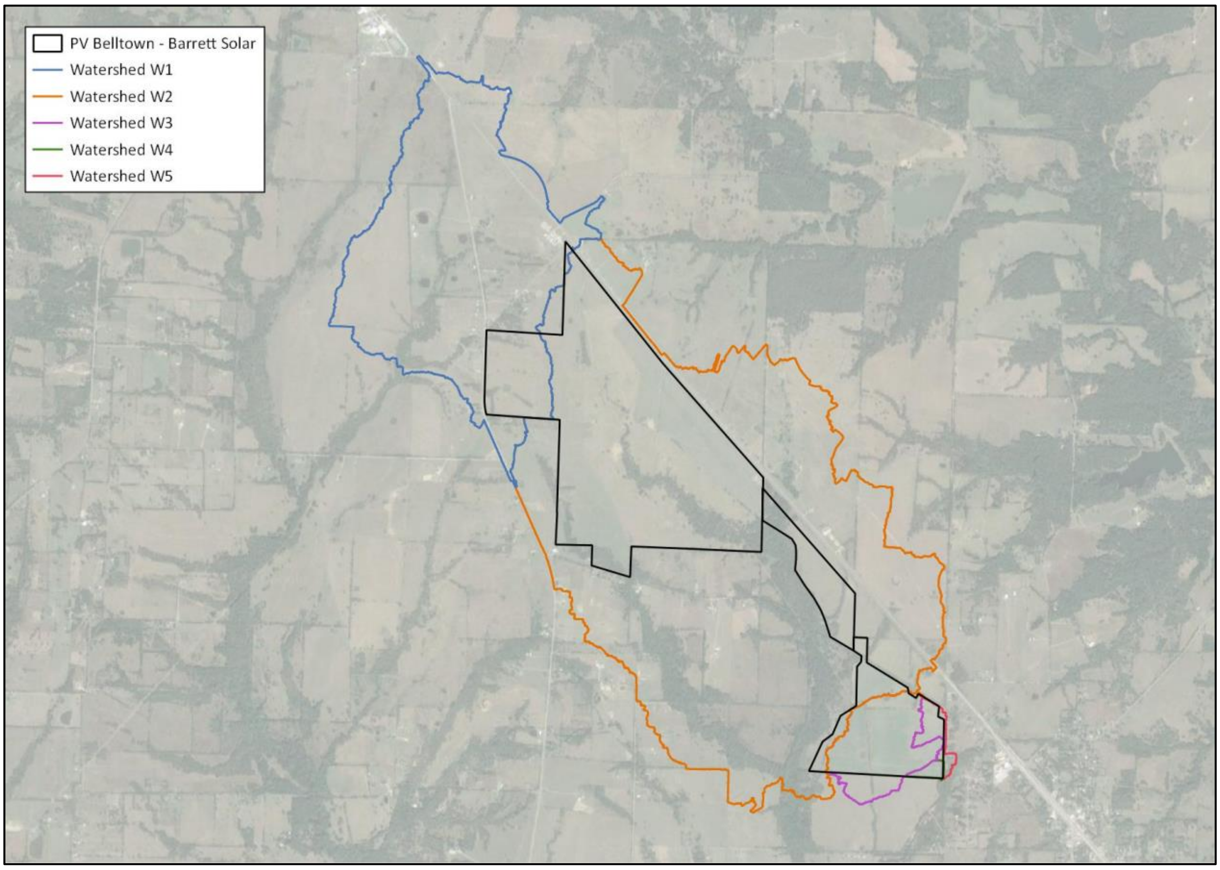

Figure 4. Watersheds and study area.

Table 1. Characteristics of the study basins.

Name	Area (km²)	Hmax (m)	Hmin (m)	Difference in height (m)	Channel length (km)	Slope %	Tc (h)
Watershed W1	3.34	171.38	148.14	23.23	4.34	0.53%	2.48
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Watershed W5	0.08	155.29	147.64	7.66	0.88	0.87%	0.67

6. CALCULATION OF MAXIMUM DAILY PRECIPITATION

In order to obtain the corresponding maximum daily precipitation (P_d) for a return period of 100 years, data from several nearby meteorological stations were used. This procedure is explained in the following section.

6.1. CALCULATION OF MAXIMUM DAILY PRECIPITATION FROM WEATHER STATIONS DATA

The National Oceanic and Atmospheric Administration (NOAA) https://www.ncei.noaa.gov/cdo-web/datatools/findstation is used to collect meteorological data, in the form of daily rainfall time series that will be fitted according to Gumbel, SQRT-ETmax and Log Pearson III functions.

The nearest meteorological stations that have a sufficient number of data to be able to perform the necessary statistical analysis are as follows:

Table 2. Information from the meteorological stations studied.

Code	Name	Time series of precipitation data	Years of records	Distance to plots (km)
USC00413734	GREENVILLE KGVL RADIO, TX US	1900 - 2023	123	30.1
USC00418743	SULPHUR SPRINGS, TX US	1893 - 2023	119	33.6
USC00412902	EMORY, TX US	1897 - 2012	70	14.5

Figure 5. Location of the chosen meteorological stations with respect to the study area.

The calculation of the maximum daily rainfall for different return periods from station rainfall data was performed using the Gumbel maximum, SQRT-ET_{max} and Log Pearson III methods.

Gumbel Distribution

The mathematical expression of the Gumbel function with the maximum likelihood method of adjustment is as follows:

$$F(x) = e^{\left[-e^{-a(x-x_0)}\right]}$$

Where:

- X= value of the calculation variable
- F(x)= probability that the value of x will not be exceeded
- a, xo = function parameters

The parameters of the function can be related to the mean (Xm) and the standard deviation (σ) of the sample by means of the expressions:

$$\frac{1}{a} = \frac{1}{\sigma_n} \cdot \sigma \qquad x_0 = x_m - \frac{y_n}{\sigma_n} \cdot \sigma$$

Depending on yn and σ n of the sample size.

Operating on the expression of Gumbel's Law and considering that F(x) represents the probability of not exceeding a certain value of precipitation in a year, we obtain the following expression that allows us to calculate the precipitation associated with the desired return period.

$$x = x_0 - \frac{1}{a} \cdot \ln\left[\ln\left(\frac{T}{T-1}\right)\right]$$

By ordering the values from smallest to largest and calculating the value of the mean and the standard deviation, a frequency value given by:

$$x'_i = x_0 - \frac{1}{a} \cdot Y_i$$

Where Y_i is:

$$Y_i = \ln\left[\ln\left(\frac{n+1}{i}\right)\right]$$

Thus, the data corresponding to the Gumbel fit are plotted on the same graph. With this, the goodness of fit of the series to this function is verified.

In addition to the graphical verification, the value of the r^2 coefficient between the measured and estimated data will be calculated in each case. This coefficient is defined as:

$$r = \frac{n(\Sigma XY) - (\Sigma X)(\Sigma Y)}{\sqrt{\left[n\Sigma X^2 - (\Sigma X)^2\right]\left[n\Sigma Y^2 - (\Sigma Y)^2\right]}}$$

X and Y being the respective ranges of values. Logically, the closer the value of this coefficient is to 1, the better the fit of the function to the measured data.

Distribution of SQRT-ET_{max}

The SQRT-ET_{max} distribution function provides a theoretical scheme for the analysis of annual rainfall maxima based on the following assumptions:

- The intensity and duration of a storm event are independent phenomena.
- The duration of rainfall is distributed according to an exponential and the intensity according to a Gamma law.
- The total amount of rainfall of a storm event is proportional to the product of its duration and intensity.
- The occurrence of large showers follows the Poisson distribution. The mathematical expression of this function is as follows:

$$F(x) = e^{-k\cdot\left[\left(1+\sqrt{\alpha\cdot x}\cdot e^{-\sqrt{\alpha\cdot x}}\right)\right]}$$

Which is a function of two parameters, k, shape, and α, scale. Moments cannot be obtained analytically, so numerical methods must be used for fitting.

This distribution function is easy to use because it has only two parameters and raises the values of the maximum precipitation quantiles with respect to those obtained by Gumbel thanks to the square root of the second exponential term.

This method is based on the assumption that the available sample data should maximize the associated probability. Therefore, the parameters of the function are obtained by making the probability of obtaining the recorded values maximum.

We assume that a set of values following our distribution function is available. The joint probability of occurrence of these values depends on the parameters of the sample, and can be written as:

$$P(X, k, \alpha) = \prod_{i01}^{N} f(x_i, k, \alpha) \cdot dx_i$$

Which is the so-called likelihood function. The values of the parameters are those that maximize it.

Log Pearson III distribution

The Log Pearson III distribution function belongs to the "Pearson distribution" family of continuous probabilistic distributions. It is used for the analysis of series with maximum river flows. In this case it will be used for the maximum daily precipitation data for each year.

The procedure is as follows:

- The values of the given series must be transformed to their decimal logarithms, obtaining then the following parameters:

  - Arithmetic mean: $\overline{Q} = \dfrac{\sum \log Q_i}{N}$

  - Standard deviation: $o_{\log Q} = \sqrt{\dfrac{(\log Q_i - \overline{Q})^2}{N-1}}$

  - Asymmetry coefficient: $g = \dfrac{N \sum (\log Qi - \overline{Q})^3}{(N-1)(N-2)(\sigma_{\log Q})^3}$

- Once obtained, the tabulated values of parameter k are obtained, which depend on the asymmetry coefficient g and the return period.

- Finally, the values associated with each return period are obtained with the following formula:

$$\log Q = \overline{Q} + k \times \sigma$$

For the adjustment of the data series obtained from the meteorological station by means of the different statistical functions, the RETURN 2.0 application developed by FLUMEN, Institute of River Dynamics and Hydrological Engineering, has been used. This application allows us to know the maximum daily rainfall for different return periods by adjusting the data series by means of the different statistical distributions explained above.

The following is a screenshot of the calculations performed with the application for each weather station:

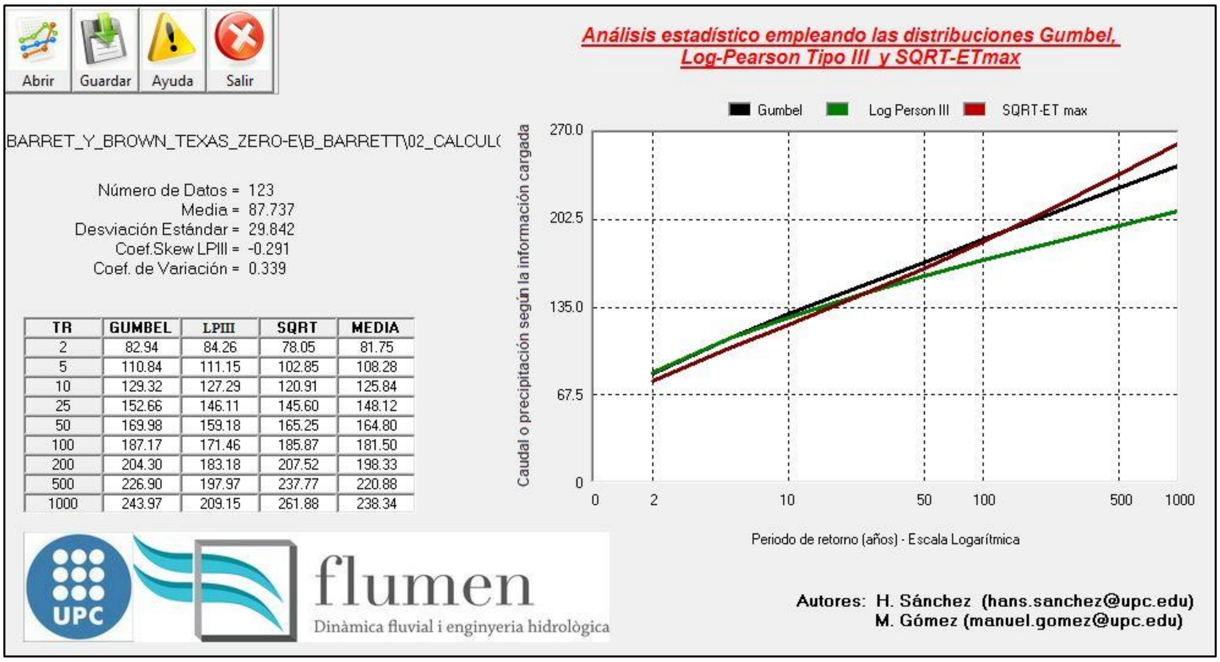
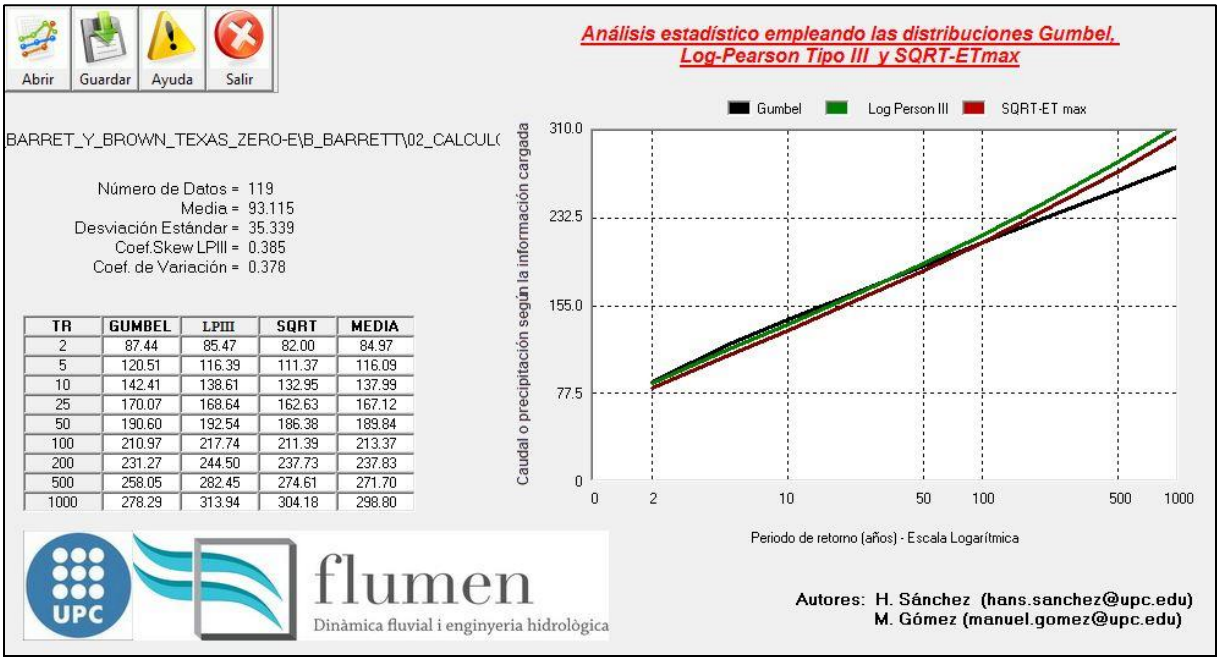
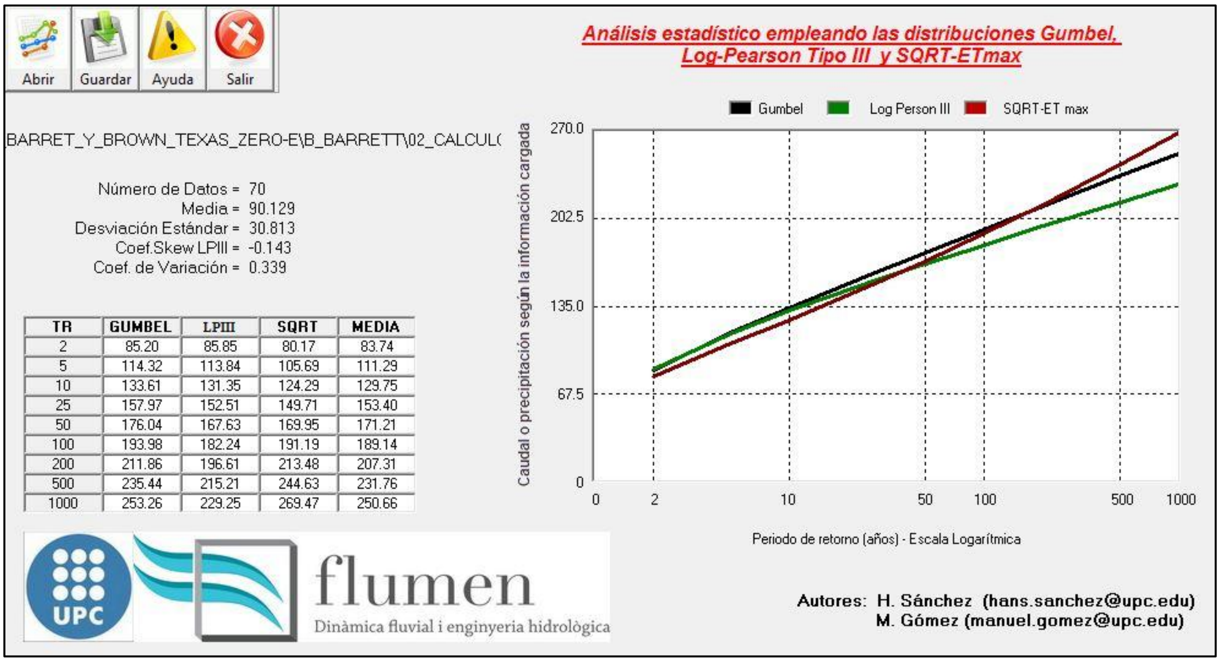


Figure 6. Capture of the RETORNO 2.0 application when obtaining the results associated with the "MS1 - GREENVILLE KGVL RADIO, TX US" meteorological station.



TR	GUMBEL	LPIII	SQRT	MEDIA
2	87.44	85.47	82.00	84.97
5	120.51	116.39	111.37	116.09
10	142.41	138.61	132.95	137.99
25	170.07	168.64	162.63	167.12
50	190.60	192.54	186.38	189.84
100	210.97	217.74	211.39	213.37
200	231.27	244.50	237.73	237.83
500	258.05	282.45	274.61	271.70
1000	278.29	313.94	304.18	298.80

Figure 7. Capture of the RETORNO 2.0 application when obtaining the results associated with the "MS2 - SULPHUR SPRINGS, TX US" meteorological station.



TR	GUMBEL	LPIII	SQRT	MEDIA
2	85.20	85.85	80.17	83.74
5	114.32	113.84	105.69	111.29
10	133.61	131.35	124.29	129.75
25	157.97	152.51	149.71	153.40
50	176.04	167.63	169.95	171.21
100	193.98	182.24	191.19	189.14
200	211.86	196.61	213.48	207.31
500	235.44	215.21	244.63	231.76
1000	253.26	229.25	269.47	250.66

Figure 8. Capture of the RETORNO 2.0 application when obtaining the results associated with the "MS3 - EMORY, TX US" meteorological station.

The results obtained with the RETORNO 2.0 application are shown below:

Table 3. Maximum precipitations resulting from the "MS1 - GREENVILLE KGVL RADIO, TX US" meteorological station.

	E1 - GREENVILLE KGVL RADIO, TX US			
Periodo de retorno T (años)	Gumbel	LPIII	SQRT	Average
100	187.17	171.46	185.87	181.5

Table 4. Maximum precipitations resulting from the "MS2 - SULPHUR SPRINGS, TX US" meteorological station.

	E2 - SULPHUR SPRINGS, TX US			
Periodo de retorno T (años)	Gumbel	LPIII	SQRT	Average
100	210.97	217.74	211.39	213.37

Table 5. Maximum precipitations resulting from the "MS3 - EMORY, TX US" meteorological station.

	E3 - EMORY, TX US			
Periodo de retorno T (años)	Gumbel	LPIII	SQRT	Average
100	193.98	182.24	191.19	189.14

6.2. CONCLUSIONS

After calculating the maximum daily precipitation for a return period of 100 years using max Gumbel, Log Pearson III and SQRT-Max methods for the station, the highest value of precipitation for each return period was chosen.

Therefore, the following precipitation value is obtained P_d:

Table 6. P_d precipitation values chosen for the return periods considered.

Return period (years)	Precipitation P_d (mm/day)
100	217.74

7. SCS UNIT HYDROGRAPH SUPERIMPOSITION METHOD

In the calculation of the maximum flood flows in the study basins, the method of superposing unit hydrographs of the SCS of the United States was applied. This methodology first calculates the effective precipitation or volume runoff in the basin and then estimates the maximum inflow from one or more unit hydrographs that are superposed in time.

The hydrological study developed through this method consists of the following stages:

A) Delimitation of catchment areas.
B) Determination of the physical characteristics of the basins: area, length and slope.
C) Evaluation of watershed morphology (type of terrain and land use).
D) Determination of maximum daily precipitation.
E) Obtaining IDF curves.
F) Curve number.
G) Net or effective precipitation estimation.
H) Obtaining the runoff coefficient.
I) Calculation of peak flows per unit hydrograph.
J) Calculation of peak flows by superposing unit hydrographs.

The steps concerning the morphological characteristics of the basins and the determination of the maximum daily precipitation have been calculated in previous sections (points 5 and 6 of this document).

7.1. OBTAINING IDF CURVES

The IDF curves method allows us to know the behavior of rainfall through a curve defined by the parameters of average intensity, duration and frequency of rainfall. For the calculation of the IDF curves corresponding to the study area, the results of maximum daily precipitation (P_d) obtained in point 6 of this document have been used.

The procedure followed to obtain the IDF curves is as follows.

7.1.1. OBTAINING MAXIMUM PRECIPITATION FOR DIFFERENT STORM DURATIONS

First, the maximum daily rainfall value (Pd) has to be converted into the maximum 24-hour rainfall value. For this purpose, Weiss (1964) established the average factors by which the precipitation measured at certain fixed intervals must be multiplied in order to transform them into the actual observed values of rainfall of such duration. In this case, in order for the daily rainfall to correspond to the 24-hour duration, it must be multiplied by 1.13:

Table 7. 24 h precipitation obtained by multiplying P_d by 1.13 according to Weiss (1964).

T (years)	P_d (mm)	P_{24h} (mm)
100	217.74	246.05

Secondly, the maximum precipitation per time duration of the precipitation event is obtained. Next, the coefficients proposed by Campos Aranda (1978) are shown, which establish a percentage relationship between 24-hour precipitation and other durations.

Table 8. Coefficients relating 24 h precipitation to lower precipitation times. Source: D.F. Campos A., 1978.

Durations, in hours									
1	2	3	4	5	6	8	12	18	24
0.30	0.39	0.46	0.52	0.57	0.61	0.68	0.80	0.91	1.00

Once the above coefficients have been applied, the maximum rainfall values for different return periods and rainfall duration times are as follows:

Table 9. Maximum rainfall for different precipitation durations and return periods.

Time of duration	Ratio	Maximum precipitation P (mm) according to return period
		100 years
24 hr	X24	246.046
18 hr	X18 = 91%	223.902
12 hr	X12 = 80%	196.837
8 hr	X8 = 68%	167.311
6 hr	X6 = 61%	150.088
5 hr	X5 = 57%	140.246
4 hr	X4 = 52%	127.944
3 hr	X3 = 46%	113.181
2 hr	X2 = 39%	95.958
1 hr	X1 = 30%	73.814

7.1.2. RAINFALL INTENSITIES OBTAINED FOR DIFFERENT STORM DURATIONS

The precipitation intensity is defined by the following expression:

$$I = \frac{P}{t}$$

Where:

- I = Rainfall intensity in mm/h.
- P = Maximum precipitation in mm.
- t = Duration of rain in hours.

Table 10. Rainfall intensities for different rainfall durations and return periods.

Time of duration		Rainfall intensity (mm/h) according to return period
h	**min**	**100 years**
24	1440	10.252
18	1080	12.439
12	720	16.403
8	480	20.914
6	360	25.015
5	300	28.049
4	240	31.986
3	180	37.727
2	120	47.979
1	60	73.814

7.1.3. IDF CURVE PARAMETERS

The mathematical expression of the Intensity - Duration - Frequency (IDF) curves is as follows:

$$I = \frac{KT^m}{t^n}$$

Where:

- I = Rainfall intensity in mm/h.
- t = Rain duration in min.
- T = Return period in years.
- K, m, n = Adjustment parameters.

Performing a variable change:

$$d = K \cdot T^{m}$$

Thus, from the above expression we obtain:

$$I = \frac{d}{t^{n}} \rightarrow I = d \cdot t^{-n}$$

After calculating the regression constants and undoing the change of variable applied, we obtain the fitting parameters for the IDF curve of the study area:

Table 11. IDF curve fitting parameters.

K	m	n
358.9128	0.205167	0.61639

7.1.4. IDF CURVE

Once the curve fitting parameters are defined, the expression that allows to calculate the rainfall intensity for any return period and rainfall duration is defined.

$$I = \frac{358.9128 \cdot T^{0.205167}}{t^{0.61639}}$$

Table 12. Intensity results of the IDF curve for rainfall duration between 5 and 60 minutes and different return periods.

Curve IDF												
T (years)	Duration in minutes											
	5	10	15	20	25	30	35	40	45	50	55	60
100	342.36	223.32	173.94	145.68	126.96	113.46	103.18	95.02	88.37	82.81	78.09	74.01

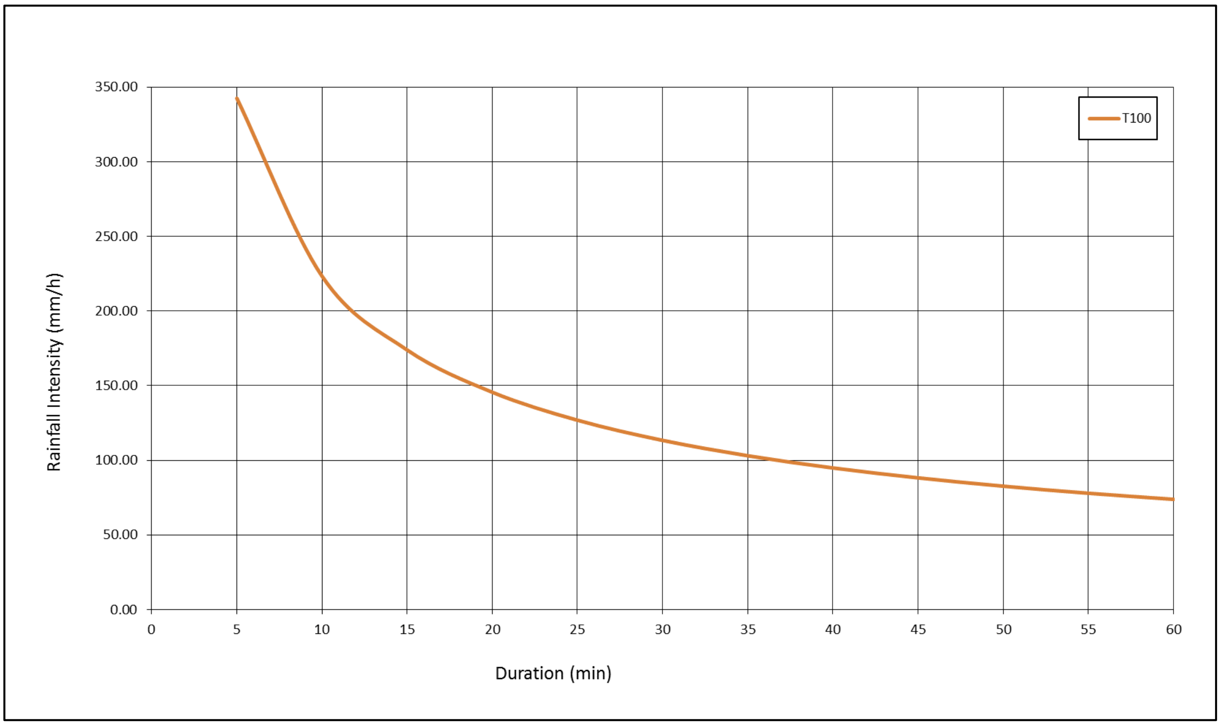


Figure 9. Graphical representation of IDF curves belonging to the study area for a rainfall duration between 5 and 60 minutes.

The rainfall intensity values obtained using the above expression for durations ranging from 1 to 24 hours are shown below:

Table 13. Intensity results of the IDF curve for rainfall duration between 1 and 24 hours and different return periods.

IDF curve		
Duration		T (years)
(h)	(min)	100
1	60	74.01
2	120	48.28
3	180	37.60
4	240	31.49
5	300	27.44
6	360	24.53
7	420	22.30
8	480	20.54
9	540	19.10
10	600	17.90
11	660	16.88
12	720	16.00
13	780	15.23

IDF curve		
Duration		T (years)
(h)	(min)	100
14	840	14.55
15	900	13.94
16	960	13.40
17	1020	12.91
18	1080	12.46
19	1140	12.05
20	1200	11.68
21	1260	11.33
22	1320	11.01
23	1380	10.71
24	1440	10.44

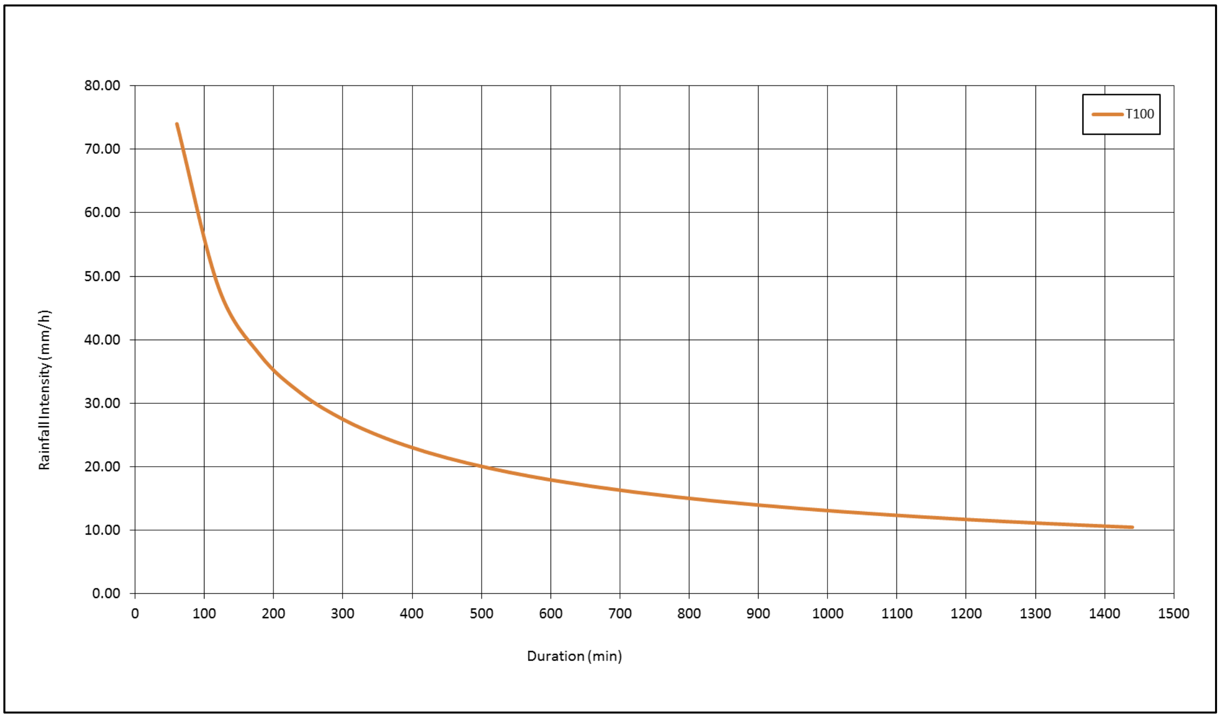


Figure 10. Graphical representation of IDF curves belonging to the study area for a rainfall duration between 1 and 24 hours.

7.2. ESTIMATION OF THE CURVE NUMBER (CN)

The curve number (CN) is a hydrologic parameter used by the US Soil Conservation Service (SCS) method to characterize the runoff potential of a watershed. This parameter is determined from the morphometric characteristics of the watershed such as land use, hydrologic group, etc. and takes integer values between 0 and 100. The following is a methodology for automatically obtaining the curve number based on the generation of three maps through the use of Geographic Information Systems (GIS).

7.2.1. GROUND COVER

For this section, the 2019 Land Cover Map prepared by The U.S. Geological Survey (USGS), in partnership with several federal agencies, was used: https://www.mrlc.gov/data/nlcd-2019-land-cover-conus. Figures 11 and 12 show how, for the study plot, the predominant land use is categorized as "Very light forest cover (forests, scrubland...)".

Figure 11. Legend of land uses in the study area.

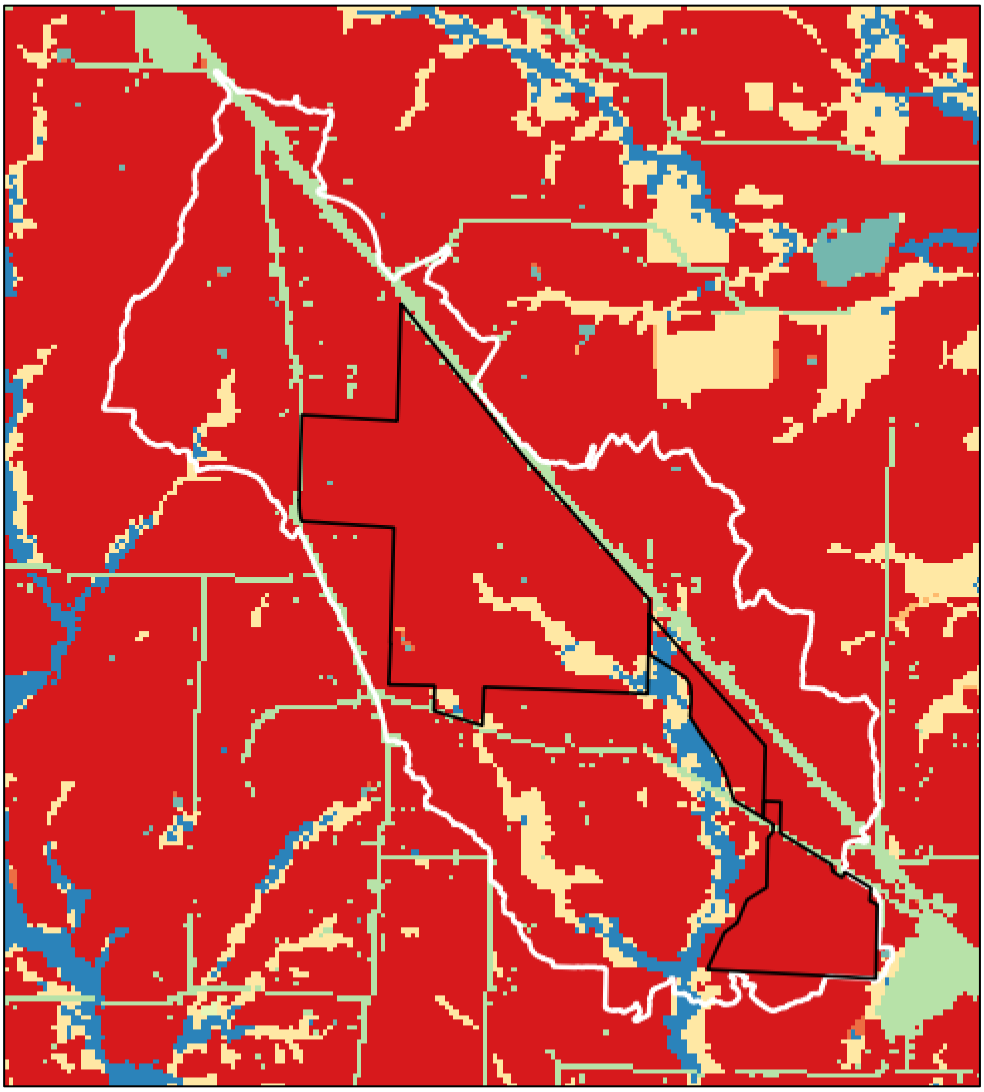


Figure 12. Land uses in the study area. Source: https://www.mrlc.gov/data/nlcd-2019-land-cover-conus.

7.2.2. SOIL TYPES

Another variable needed to determine the curve number is the soil type map. The SCS method classifies the soil into 4 types according to their infiltration capacity:

A. In them, water infiltrates quickly, even when they are very humid. Deep and coarse-textured (sandy or sandy-loam), they are excessively drained.
B. When very wet they have a moderate infiltration capacity. Soil depth is medium to deep, and their texture is sandy loam, loam, clay loam or silt loam. They are well or moderately drained.
C. When they are very humid, infiltration is slow. The soil depth is below average, and its texture is clay loam, silty clay loam or sandy clay loam. They are imperfectly drained soils.
D. When they are very wet, infiltration is very slow. They have clay horizons at or near the surface and are poorly or very poorly drained. Also included here are soils with a permanently high-water table and thin soils (lithosols).

To choose the type of soil existing in the study area, below is a map showing the soil types in the area according to the Soil Survey Geographic (SSURGO) Database, which can be downloaded at: https://www.nrcs.usda.gov/resources/data-and-reports/ssurgo/stats2go-metadata.

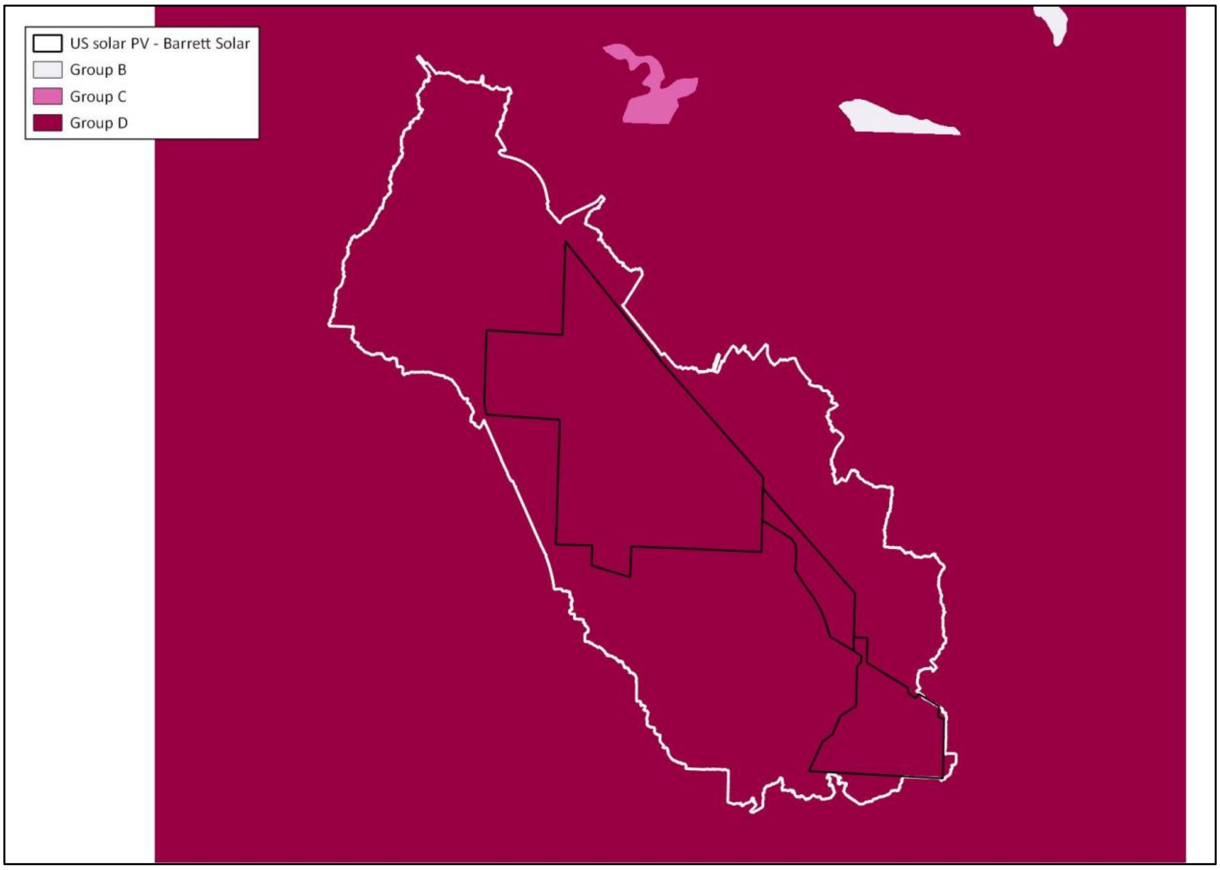


Figure 13. Existing soils in the study area. Source: SSURGO.

In view of the above image, there are three types of soil in the area of study according to this classification: group B, group C and group D.

7.2.3. SLOPE OF THE TERRAIN

Slope is a variable that has a great influence on the runoff and infiltration capacity of the land. The terrain is classified into two groups according to this criterion: less than 3%, or greater than or equal to 3%.

The slope raster of the study plots is shown below, where the cells with slope values equal to or greater than 3% are shown in blue, while those with values less than 3% are shown in pink.

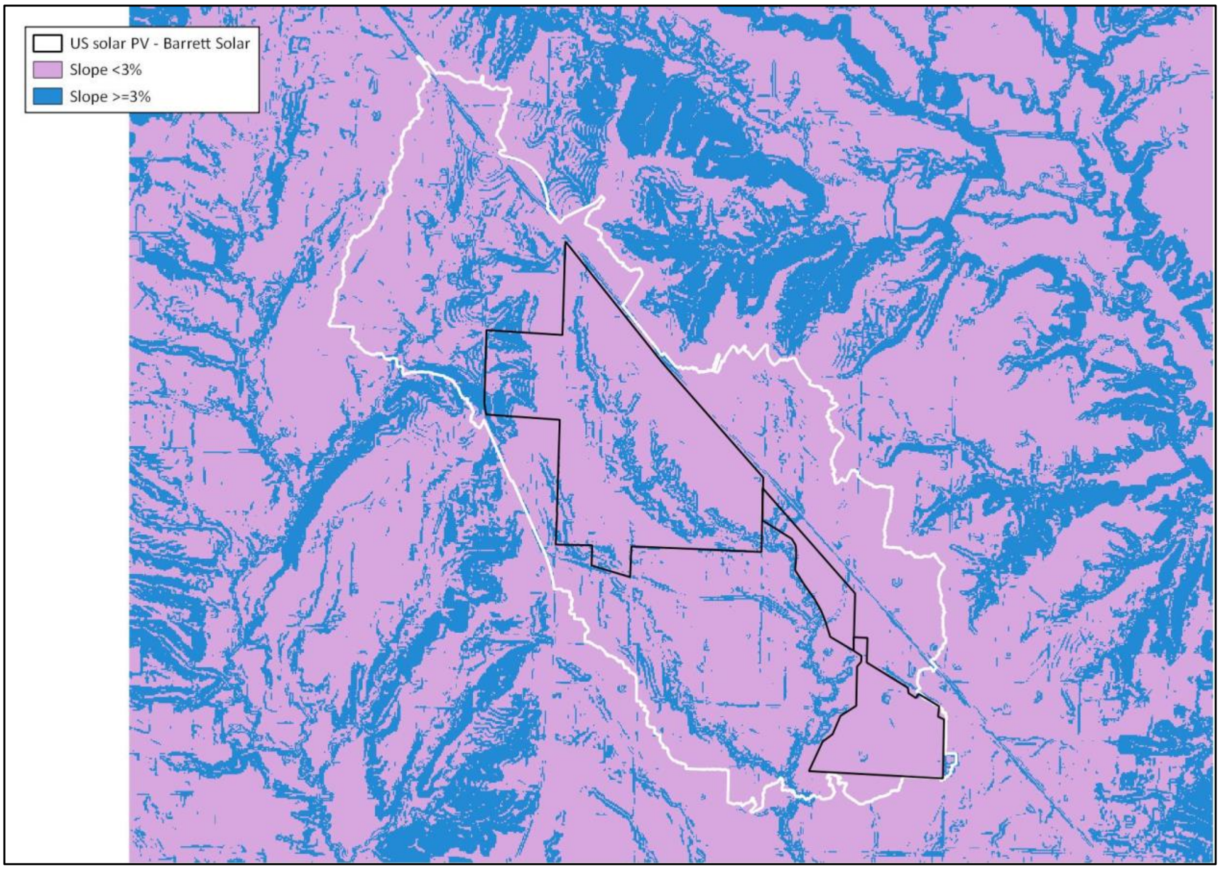


Figure 14. Slope classification raster.

7.2.4. ASSIGNMENT OF THE CN

There are three types of CN depending on the degree of initial humidity:

- CN(I): dry pre-humidity conditions.
- CN(II): normal pre-humidity conditions.
- CN(III): humid pre-humidity conditions.

The assignment of each curve number is established according to Table 14. This table associates a CN(II) according to the parameters obtained above (land use, permeability and slope) in each cell of the study area.

Table 14. Curve number for average CN(II) moisture conditions as a function of infiltration type.

ID	LAND USE	A	B	C	D
1	Rocky outcrop (Slope < 3%)	92	92	92	92
2	Rocky outcrop (Slope >= 3%)	92	92	92	92
3	Urbanized area (Slope < 3%)	98	98	98	98
4	Urbanized area (Slope >= 3%)	98	98	98	98

ID	LAND USE	A	B	C	D
5	Fallow (Slope < 3%)	71	78	82	86
6	Fallow (Slope >= 3%)	74	82	86	89
7	Winter cereals (Slope < 3%)	59	70	78	81
8	Winter cereals (Slope >= 3%)	61	73	81	83
9	Row crops (Slope < 3%)	64	73	78	82
10	Row crops (Slope >= 3%)	67	76	82	86
11	Wetlands and swampy areas (Slope < 3%)	96	96	96	96
12	Wetlands and swampy areas (Slope >= 3%)	96	96	96	96
13	Light forest cover (forests, scrubland...) (Slope < 3%)	46	68	78	83
14	Light forest cover (forests, scrubland...) (Slope >= 3%)	46	68	78	83
15	Thick forest cover (forests, scrubland...) (Slope < 3%)	36	52	62	69
16	Thick forest cover (forests, scrubland...) (Slope >= 3%)	36	52	62	69
17	Medium forest cover (forests, scrubland...) (Slope < 3%)	40	60	69	76
18	Medium forest cover (forests, scrubland...) (Slope >= 3%)	40	60	69	76
19	Very light forest cover (forests, scrubland...) (Slope < 3%)	56	75	86	91
20	Very light forest cover (forests, scrubland...) (Slope >= 3%)	56	75	86	91
21	Very thick forest cover (forests, scrubland...) (Slope < 3%)	29	44	54	60
22	Very thick forest cover (forests, scrubland...) (Slope >= 3%)	29	44	54	60
23	Regular plantations with good forest harvesting (Slope < 3%)	25	50	67	76
24	Regular plantations with good forest harvesting (Slope >= 3%)	33	54	69	77
25	Regular plantations with medium forest harvesting (Slope < 3%)	35	54	69	77
26	Regular plantations with medium forest harvesting (Slope >= 3%)	39	60	73	78
27	Regular plantations with poor forest harvesting (Slope < 3%)	40	60	73	78
28	Regular plantations with poor forest harvesting (Slope >= 3%)	45	66	77	83
29	Beaches and dunes (Slope < 3%)	25	25	25	25
30	Beaches and dunes (Slope >= 3%)	25	25	25	25
31	Good pasture (Slope < 3%)	29	48	69	78
32	Good pasture (Slope >= 3%)	42	60	74	79
33	Medium pasture (Slope < 3%)	39	59	75	83
34	Medium pasture (Slope >= 3%)	49	69	78	85
35	Very good pasture (Slope < 3%)	17	33	67	76
36	Very good pasture (Slope >= 3%)	39	55	69	77
37	Poor pasture (Slope < 3%)	46	67	81	88
38	Poor pasture (Slope >= 3%)	68	78	86	89
39	Rivers and watercourses (Slope < 3%)	100	100	100	100
40	Rivers and watercourses (Slope >= 3%)	100	100	100	100
41	Impermeable rocks (Slope < 3%)	93	93	93	93
42	Impermeable rocks (Slope >= 3%)	96	96	96	96
43	Permeable rocks (Slope < 3%)	91	91	91	91
44	Permeable rocks (Slope >= 3%)	94	94	94	94
45	Dense crop rotation (Slope < 3%)	52	67	76	79
46	Dense crop rotation (Slope >= 3%)	54	69	78	82
47	Rotation of poor crops (Slope < 3%)	63	73	79	83
48	Rotation of poor crops (Slope >= 3%)	64	75	82	86
49	Mining extraction areas (Slope < 3%)	76	85	89	91
50	Mining extraction areas (Slope >= 3%)	76	85	89	91

In the case of the study area, it has a relevant accumulated annual precipitation, so it is appropriate to consider the use of the curve number for humid pre-humidity conditions CN(III). This parameter is calculated from the curve number for normal pre-humidity conditions CN(II) with the following formula:

$$CN(III) = \frac{23 \cdot CN(II)}{10 + 0{,}13 \cdot CN(II)}$$

The result of the reclassification of the table according to the parameters obtained for the study plots is shown below, with the curve number under humid pre-humidity conditions averaging around 92.

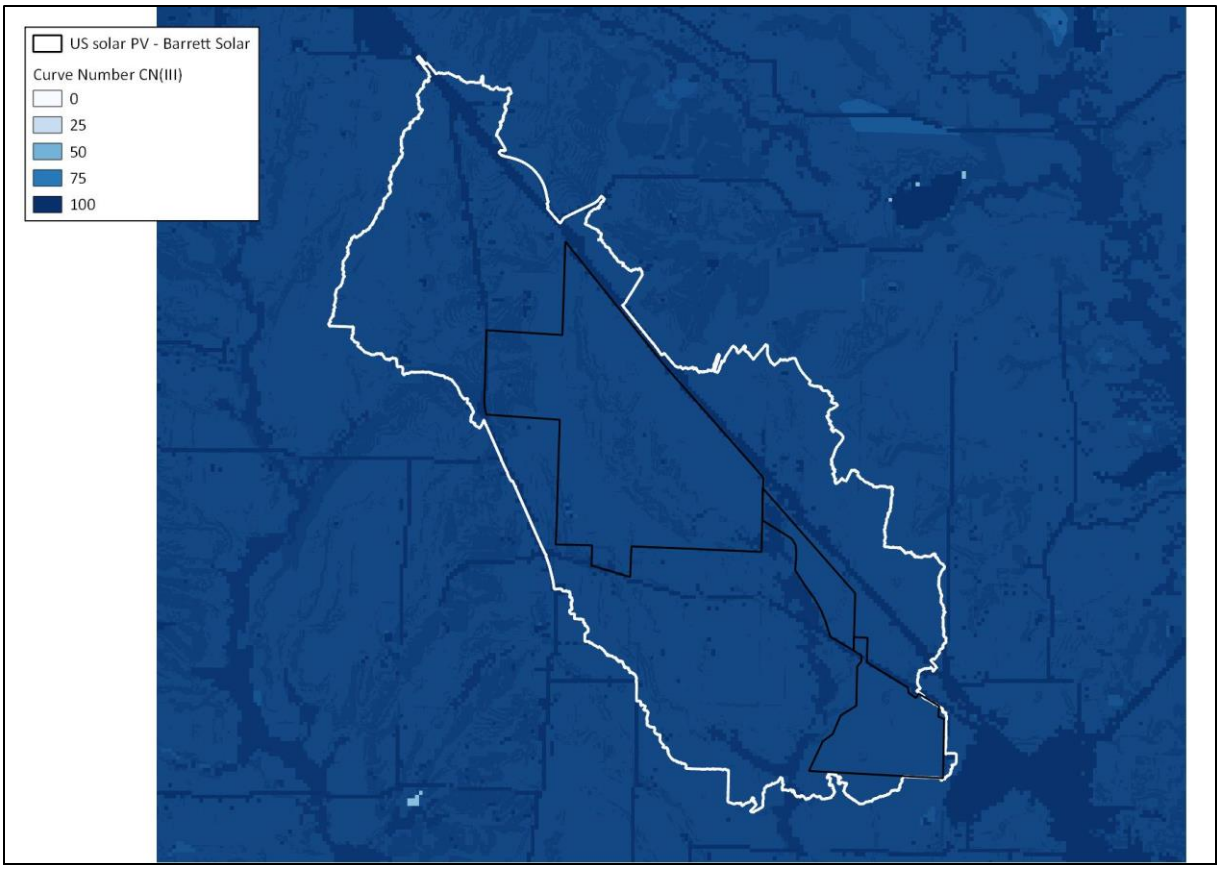


Figure 15. Map with the resulting CN(III) curve numbers.

Thus, the following curve number values are obtained for the study basins:

Table 15. Average curve number obtained in watershed W1.

Watershed 1				
ID	**Land Use**	**CN(II)**	**CN(III)**	**Area %**
47	Rotation of poor crops (Slope < 3%)	83	91.82	66.84%
48	Rotation of poor crops (Slope >= 3%)	89	94.90	21.22%
3	Urbanized area (Slope < 3%)	98	99.12	6.71%
4	Urbanized area (Slope >= 3%)	98	99.12	3.02%
13	Light forest cover (forests, scrubland...) (Slope < 3%)	83	91.82	1.02%
14	Light forest cover (forests, scrubland...) (Slope >= 3%)	83	91.82	0.62%
39	Rivers and watercourses (Slope < 3%)	100	100.00	0.21%
11	Wetlands and swampy areas (Slope < 3%)	96	98.22	0.18%
12	Wetlands and swampy areas (Slope >= 3%)	96	98.22	0.10%
40	Rivers and watercourses (Slope >= 3%)	100	100.00	0.06%
35	Very good pasture (Slope < 3%)	76	87.93	0.02%
36	Very good pasture (Slope >= 3%)	77	88.51	0.00%

CN(III) average	93.22

Table 16. Average curve number obtained in watershed W2.

Watershed 2				
ID	**Land Use**	**CN(II)**	**CN(III)**	**Area %**
47	Rotation of poor crops (Slope < 3%)	83	91.82	75.41%
48	Rotation of poor crops (Slope >= 3%)	89	94.90	7.80%
13	Light forest cover (forests, scrubland...) (Slope < 3%)	83	91.82	4.34%
3	Urbanized area (Slope < 3%)	98	99.12	4.18%
14	Light forest cover (forests, scrubland...) (Slope >= 3%)	83	91.82	2.75%
11	Wetlands and swampy areas (Slope < 3%)	96	98.22	1.99%
12	Wetlands and swampy areas (Slope >= 3%)	96	98.22	1.95%
4	Urbanized area (Slope >= 3%)	98	99.12	1.42%
39	Rivers and watercourses (Slope < 3%)	100	100.00	0.05%
19	Very light forest cover (forests, scrubland...) (Slope < 3%)	91	95.88	0.04%
35	Very good pasture (Slope < 3%)	76	87.93	0.03%
20	Very light forest cover (forests, scrubland...) (Slope >= 3%)	91	95.88	0.02%
36	Very good pasture (Slope >= 3%)	77	88.51	0.02%
40	Rivers and watercourses (Slope >= 3%)	100	100.00	0.01%

CN(III) average	92.73

Table 17. Average curve number obtained in watershed W3.

Watershed 3				
ID	**Land Use**	**CN(II)**	**CN(III)**	**Area %**
47	Rotation of poor crops (Slope < 3%)	83	91.82	98.49%
48	Rotation of poor crops (Slope >= 3%)	89	94.90	1.38%
3	Urbanized area (Slope < 3%)	98	99.12	0.12%
4	Urbanized area (Slope >= 3%)	98	99.12	0.01%

CN(III) average	91.87

Table 18. Average curve number obtained in watershed W4.

Watershed 4				
ID	**Land Use**	**CN(II)**	**CN(III)**	**Area %**
47	Rotation of poor crops (Slope < 3%)	83	91.82	92.22%
13	Light forest cover (forests, scrubland...) (Slope < 3%)	83	91.82	3.99%
48	Rotation of poor crops (Slope >= 3%)	89	94.90	3.24%
14	Light forest cover (forests, scrubland...) (Slope >= 3%)	83	91.82	0.55%

CN(III) average	91.92

Table 19. Average curve number obtained in watershed W5.

Watershed 5				
ID	**Land Use**	**CN(II)**	**CN(III)**	**Area %**
47	Rotation of poor crops (Slope < 3%)	83	91.82	82.70%
48	Rotation of poor crops (Slope >= 3%)	89	94.90	4.69%
14	Light forest cover (forests, scrubland...) (Slope >= 3%)	83	91.82	3.66%
39	Rivers and watercourses (Slope < 3%)	100	100.00	2.58%
13	Light forest cover (forests, scrubland...) (Slope < 3%)	83	91.82	2.48%
3	Urbanized area (Slope < 3%)	98	99.12	1.47%
40	Rivers and watercourses (Slope >= 3%)	100	100.00	1.27%
36	Very good pasture (Slope >= 3%)	77	88.51	0.66%
35	Very good pasture (Slope < 3%)	76	87.93	0.50%

CN(III) average	92.35

7.3. NET RAINFALL DURATION

Before the net precipitation and peak flow can be calculated by the SCS unit hydrograph superposition method, it is necessary to know the duration of the design storm D, for which the final peak flow will be the highest.

Since this duration is not obtained directly before calculating the peak flow, its value will be tested, performing the whole procedure of calculating the net precipitation and peak flow with multiple options of D, all of them increasing by 0.2 - Tc with respect to the previous case, whose value will also be the same that will be assumed for the duration of precipitation associated with each unit hydrograph ΔD.

7.4. CALCULATION OF DESIGN RAINFALL INTENSITY

The design rainfall intensity I_d is the total rainfall intensity that will fall over the entire design storm duration.

In this case, it is calculated from the IDF curves obtained previously in this document, where the intensity value is obtained by entering the formula of the curve with the return period T and the corresponding design storm duration D. The following tables show the I_d values obtained for each basin:

	I_d (mm/h) for T-100 years per Watershed				
D(h)	**W1**	**W2**	**W3**	**W4**	**W5**
0.1 · Tc	175.012	113.022	235.020	533.549	391.980
0.2 · Tc	114.161	73.725	153.304	348.035	255.690
0.4 · Tc	74.467	48.091	100.001	227.024	166.787
0.6 · Tc	58.000	37.456	77.887	176.820	129.904
0.8 · Tc	48.575	31.370	65.231	148.089	108.796
1 · Tc	42.333	27.339	56.848	129.059	94.815
1.2 · Tc	37.833	24.433	50.806	115.340	84.737
1.4 · Tc	34.404	22.218	46.201	104.886	77.056
1.6 · Tc	31.686	20.463	42.550	96.599	70.968
1.8 · Tc	29.467	19.030	39.571	89.834	65.998
2 · Tc	27.614	17.833	37.082	84.185	61.848
2.2 · Tc	26.039	16.816	34.967	79.382	58.319
2.4 · Tc	24.679	15.937	33.141	75.237	55.274
2.6 · Tc	23.491	15.170	31.545	71.615	52.613
2.8 · Tc	22.442	14.493	30.137	68.417	50.264

7.5. CALCULATION OF DESIGN RAINFALL

The design rainfall or $P_{d\text{-}D}$ is that which will fall in the duration of the design storm D. Obtaining this rainfall is used to subsequently extract its associated net rainfall $P_{n\text{-}D}$ and thus calculate the runoff coefficient for each duration of the design storm D. As mentioned above, multiple choices of D are assumed, all of them increasing by 0.2 - Tc with respect to the previous case.

Therefore, $P_{d\text{-}D}$ is obtained by multiplying I_d by the duration of the corresponding design storm D:

	$P_{d\text{-}D}$ (mm) for T-100 years per Watershed				
D(h)	W1	W2	W3	W4	W5
0.1 · Tc	43.319	56.868	36.057	21.646	26.226
0.2 · Tc	56.514	74.190	47.040	28.240	34.214
0.4 · Tc	73.728	96.788	61.369	36.842	44.636
0.6 · Tc	86.136	113.077	71.697	43.042	52.148
0.8 · Tc	96.186	126.271	80.062	48.065	58.232
1 · Tc	104.783	137.556	87.218	52.360	63.437
1.2 · Tc	112.373	147.521	93.536	56.153	68.032
1.4 · Tc	119.219	156.507	99.234	59.574	72.177
1.6 · Tc	125.485	164.733	104.450	62.705	75.970
1.8 · Tc	131.285	172.347	109.277	65.603	79.482
2 · Tc	136.700	179.456	113.785	68.309	82.760
2.2 · Tc	141.790	186.138	118.022	70.853	85.842
2.4 · Tc	146.603	192.456	122.028	73.258	88.755
2.6 · Tc	151.174	198.458	125.833	75.542	91.523
2.8 · Tc	155.534	204.180	129.461	77.721	94.162

7.6. CALCULATION OF NET OR EFFECTIVE PRECIPITATION

The net or effective precipitation $P_{n\text{-}D}$ is that part of the total precipitation fallen during the design storm D that generates direct runoff. Precipitation begins to produce runoff when the total precipitation fallen up to that time exceeds the initial abstraction I_a. The initial abstraction to be exceeded is considered to be 20% of the maximum possible abstraction (S). Thus, the maximum possible abstraction can be obtained by the following expression:

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

Where:

- S = Maximum potential retention in mm.
- CN = Curve number.

Table 20. Values of S (mm) and I_a (mm) by basin.

Watershed	CN (III) average	S (mm)	I_a (mm)
Watershed W1	93.22	18.46	3.69
Watershed W2	92.73	19.92	3.98
Watershed W3	91.87	22.46	4.49
Watershed W4	91.92	22.32	4.46
Watershed W5	92.35	21.05	4.21

Once the maximum abstraction has been obtained, the net precipitation is calculated using the following expression:

$$P_{n-D} = \frac{(P_{d-D} - 0.2 \cdot S)^2}{P_{d-D} + 0.8 \cdot S}$$

Where:

- P_{n-D} = Net or effective rainfall in mm during the design storm D.
- P_{d-D}= Design rainfall for the considered return period (mm) during the design storm D previously obtained.
- S = Maximum potential retention in mm.

The tables with the P_{n-D} values calculated according to the above formulation are shown below:

	P_{n-D} (mm) for T-100 years per Watershed				
D(h)	W1	W2	W3	W4	W5
0.1 · Tc	27.033	38.417	18.441	7.474	11.256
0.2 · Tc	39.142	54.692	27.846	12.264	17.635
0.4 · Tc	55.427	76.408	40.773	19.167	26.586
0.6 · Tc	67.361	92.252	50.369	24.440	33.313
0.8 · Tc	77.105	105.161	58.254	28.838	38.877
1 · Tc	85.481	116.240	65.059	32.672	43.699
1.2 · Tc	92.901	126.048	71.106	36.101	47.996
1.4 · Tc	99.610	134.908	76.584	39.225	51.897
1.6 · Tc	105.762	143.029	81.616	42.106	55.487
1.8 · Tc	111.466	150.555	86.287	44.789	58.824
2 · Tc	116.797	157.587	90.659	47.308	61.951
2.2 · Tc	121.815	164.202	94.777	49.686	64.900
2.4 · Tc	126.562	170.461	98.676	51.942	67.694
2.6 · Tc	131.076	176.408	102.386	54.093	70.355
2.8 · Tc	135.382	182.083	105.928	56.150	72.897

7.7. RUNOFF COEFFICIENT

The runoff coefficient determines the portion of the total precipitation falling in the study area that will be converted to runoff. To calculate the runoff coefficient C, P_{n-D} is divided by P_{d-D} for each design storm duration D and return period. The results are:

	C for T-100 years per Watershed				
D(h)	W1	W2	W3	W4	W5
0.1 · Tc	0.624	0.676	0.511	0.345	0.429
0.2 · Tc	0.693	0.737	0.592	0.434	0.515
0.4 · Tc	0.752	0.789	0.664	0.520	0.596
0.6 · Tc	0.782	0.816	0.703	0.568	0.639
0.8 · Tc	0.802	0.833	0.728	0.600	0.668
1 · Tc	0.816	0.845	0.746	0.624	0.689
1.2 · Tc	0.827	0.854	0.760	0.643	0.705
1.4 · Tc	0.836	0.862	0.772	0.658	0.719
1.6 · Tc	0.843	0.868	0.781	0.671	0.730
1.8 · Tc	0.849	0.874	0.790	0.683	0.740
2 · Tc	0.854	0.878	0.797	0.693	0.749
2.2 · Tc	0.859	0.882	0.803	0.701	0.756
2.4 · Tc	0.863	0.886	0.809	0.709	0.763
2.6 · Tc	0.867	0.889	0.814	0.716	0.769
2.8 · Tc	0.870	0.892	0.818	0.722	0.774

7.8. CALCULATION OF THE DESIGN PRECIPITATION FOR A UNIT HYDROGRAPH

The design precipitation or $P_{d-\Delta D}$ is that which will fall over the duration of precipitation associated with each unit hydrograph posed ΔD. As stated above, it is assumed for all cases that the precipitation duration in each of these hyetograms will be 0.2 - Tc, with the exception of D=0.1 - Tc, where the associated hyetogram will have that same duration.

Therefore, $P_{d-\Delta D}$ is obtained by multiplying I_d by the duration of the corresponding unit hydrograph ΔD:

		$P_{d-\Delta D}$ (mm) for T-100 years per Watershed				
D(h)	ΔD (h)	W1	W2	W3	W4	W5
0.1 · Tc	0.1 · Tc	43.319	56.868	36.057	21.646	26.226
0.2 · Tc	0.2 · Tc	56.514	74.190	47.040	28.240	34.214
0.4 · Tc	0.2 · Tc	36.864	48.394	30.684	18.421	22.318
0.6 · Tc	0.2 · Tc	28.712	37.692	23.899	14.347	17.383
0.8 · Tc	0.2 · Tc	24.047	31.568	20.016	12.016	14.558
1 · Tc	0.2 · Tc	20.957	27.511	17.444	10.472	12.687
1.2 · Tc	0.2 · Tc	18.729	24.587	15.589	9.359	11.339

		$P_{d\text{-}\Delta D}$ (mm) for T-100 years per Watershed				
D(h)	**ΔD (h)**	**W1**	**W2**	**W3**	**W4**	**W5**
1.4 · Tc	0.2 · Tc	17.031	22.358	14.176	8.511	10.311
1.6 · Tc	0.2 · Tc	15.686	20.592	13.056	7.838	9.496
1.8 · Tc	0.2 · Tc	14.587	19.150	12.142	7.289	8.831
2 · Tc	0.2 · Tc	13.670	17.946	11.378	6.831	8.276
2.2 · Tc	0.2 · Tc	12.890	16.922	10.729	6.441	7.804
2.4 · Tc	0.2 · Tc	12.217	16.038	10.169	6.105	7.396
2.6 · Tc	0.2 · Tc	11.629	15.266	9.679	5.811	7.040
2.8 · Tc	0.2 · Tc	11.110	14.584	9.247	5.551	6.726

7.9. CALCULATION OF THE NET OR EFFECTIVE PRECIPITATION FOR A UNIT HYDROGRAPH

The net or effective precipitation $P_{n\text{-}\Delta D}$ is that part of the total precipitation that produces runoff and will fall over the duration of precipitation associated with each unit hydrograph posed ΔD. As stated above, it is assumed for all cases that the precipitation duration in each of these hyetograms will be 0.2 - Tc, with the exception of D = 0.1 - Tc, where the associated hyetogram will have that same duration.

$P_{n\text{-}\Delta D}$ is obtained by multiplying $P_{d\text{-}\Delta D}$ by the corresponding runoff coefficient C previously calculated:

		$P_{n\text{-}\Delta D}$ (mm) for T-100 years per Watershed				
D(h)	**ΔD (h)**	**W1**	**W2**	**W3**	**W4**	**W5**
0.1 · Tc	0.1 · Tc	27.033	38.417	18.441	7.474	11.256
0.2 · Tc	0.2 · Tc	39.142	54.692	27.846	12.264	17.635
0.4 · Tc	0.2 · Tc	27.713	38.204	20.387	9.583	13.293
0.6 · Tc	0.2 · Tc	22.454	30.751	16.790	8.147	11.104
0.8 · Tc	0.2 · Tc	19.276	26.290	14.563	7.210	9.719
1 · Tc	0.2 · Tc	17.096	23.248	13.012	6.534	8.740
1.2 · Tc	0.2 · Tc	15.484	21.008	11.851	6.017	7.999
1.4 · Tc	0.2 · Tc	14.230	19.273	10.941	5.604	7.414
1.6 · Tc	0.2 · Tc	13.220	17.879	10.202	5.263	6.936
1.8 · Tc	0.2 · Tc	12.385	16.728	9.587	4.977	6.536
2 · Tc	0.2 · Tc	11.680	15.759	9.066	4.731	6.195
2.2 · Tc	0.2 · Tc	11.074	14.927	8.616	4.517	5.900
2.4 · Tc	0.2 · Tc	10.547	14.205	8.223	4.329	5.641
2.6 · Tc	0.2 · Tc	10.083	13.570	7.876	4.161	5.412
2.8 · Tc	0.2 · Tc	9.670	13.006	7.566	4.011	5.207

7.10. CALCULATION OF PEAK FLOWS PER UNIT HYDROGRAPH

The method to be used to obtain the maximum flow in each Basin is based on the S.C.S. and consists of the superposition of several triangular unit hydrographs. This simplification in the form of a triangle allows us to obtain the fundamental parameters defining each of these hydrographs in a simpler way.

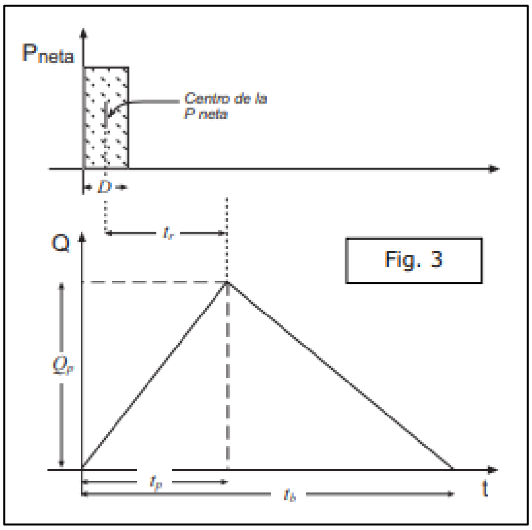


Figure 16. Fundamental parameters of a triangular hydrograph.

These parameters are repeated in the unit hydrographs:

- Peak time (t_p): Time it takes to produce the peak flow (in hours).

$$t_P = 0.5 \cdot \Delta D + 0.6 \cdot t_c$$

Where:

  - t_C: Concentration time (h).
  - ΔD: Duration of net precipitation for each unit hydrograph (h). A duration of 0.2 - Tc is estimated, except for the case of D = 0.1 - Tc, where this will be the duration value to be considered.

- Base time (t_b): time elapsed from the onset of precipitation to the end of direct runoff. Wanielista (1997) proposes the following empirical values that relate the peak time to the base time as a function of terrain type.

Urban area, steep slopes ……………………………………………; $t_b = 2.25 \cdot t_P$

Average SCS ………………………………………………………………; $t_b = 2.67 \cdot t_P$

Rural/urban mix ……………………………………………………………; $t_b = 3.25 \cdot t_P$

Rural, hills …………………………………………………………………; $t_b = 4.33 \cdot t_P$

Rural, gentle slopes ……………………………………………………; $t_b = 6.5 \cdot t_P$

Rural, very flat ……………………………………………………………; $t_b = 13 \cdot t_P$

Given that the study area is located in a rural area with slopes of less than 3%, the following expression is used:

$$t_b = 6.5 \cdot t_P$$

Therefore, the following t_p and t_b values of the unit hydrograph are obtained for each Basin:

		W1		W2		W3	
D(h)	ΔD (h)	t_p (h)	t_b (h)	t_p (h)	t_b (h)	t_p (h)	t_b (h)
0.1 · Tc	0.1 · Tc	1.609	10.458	3.271	21.258	0.997	6.482
0.2 · Tc	0.2 · Tc	1.733	11.262	3.522	22.894	1.074	6.981
0.4 · Tc	0.2 · Tc	1.733	11.262	3.522	22.894	1.074	6.981
0.6 · Tc	0.2 · Tc	1.733	11.262	3.522	22.894	1.074	6.981
0.8 · Tc	0.2 · Tc	1.733	11.262	3.522	22.894	1.074	6.981
1 · Tc	0.2 · Tc	1.733	11.262	3.522	22.894	1.074	6.981
1.2 · Tc	0.2 · Tc	1.733	11.262	3.522	22.894	1.074	6.981
1.4 · Tc	0.2 · Tc	1.733	11.262	3.522	22.894	1.074	6.981
1.6 · Tc	0.2 · Tc	1.733	11.262	3.522	22.894	1.074	6.981
1.8 · Tc	0.2 · Tc	1.733	11.262	3.522	22.894	1.074	6.981
2 · Tc	0.2 · Tc	1.733	11.262	3.522	22.894	1.074	6.981
2.2 · Tc	0.2 · Tc	1.733	11.262	3.522	22.894	1.074	6.981
2.4 · Tc	0.2 · Tc	1.733	11.262	3.522	22.894	1.074	6.981
2.6 · Tc	0.2 · Tc	1.733	11.262	3.522	22.894	1.074	6.981
2.8 · Tc	0.2 · Tc	1.733	11.262	3.522	22.894	1.074	6.981

		W4		W5	
D(h)	ΔD (h)	t_p (h)	t_b (h)	t_p (h)	t_b (h)
0.1 · Tc	0.1 · Tc	0.264	1.714	0.435	2.827
0.2 · Tc	0.2 · Tc	0.284	1.846	0.468	3.044
0.4 · Tc	0.2 · Tc	0.284	1.846	0.468	3.044
0.6 · Tc	0.2 · Tc	0.284	1.846	0.468	3.044
0.8 · Tc	0.2 · Tc	0.284	1.846	0.468	3.044
1 · Tc	0.2 · Tc	0.284	1.846	0.468	3.044
1.2 · Tc	0.2 · Tc	0.284	1.846	0.468	3.044
1.4 · Tc	0.2 · Tc	0.284	1.846	0.468	3.044
1.6 · Tc	0.2 · Tc	0.284	1.846	0.468	3.044

		W4		W5	
D(h)	ΔD (h)	t_p (h)	t_b (h)	t_p (h)	t_b (h)
1.8 · Tc	0.2 · Tc	0.284	1.846	0.468	3.044
2 · Tc	0.2 · Tc	0.284	1.846	0.468	3.044
2.2 · Tc	0.2 · Tc	0.284	1.846	0.468	3.044
2.4 · Tc	0.2 · Tc	0.284	1.846	0.468	3.044
2.6 · Tc	0.2 · Tc	0.284	1.846	0.468	3.044
2.8 · Tc	0.2 · Tc	0.284	1.846	0.468	3.044

- Peak flow of a hydrograph, (Q_O): Maximum direct runoff flow (in m^3/s) in a hydrograph.

$$Q_o = \frac{P_{n-\Delta D} \cdot A}{1.8 \cdot t_b}$$

Where:

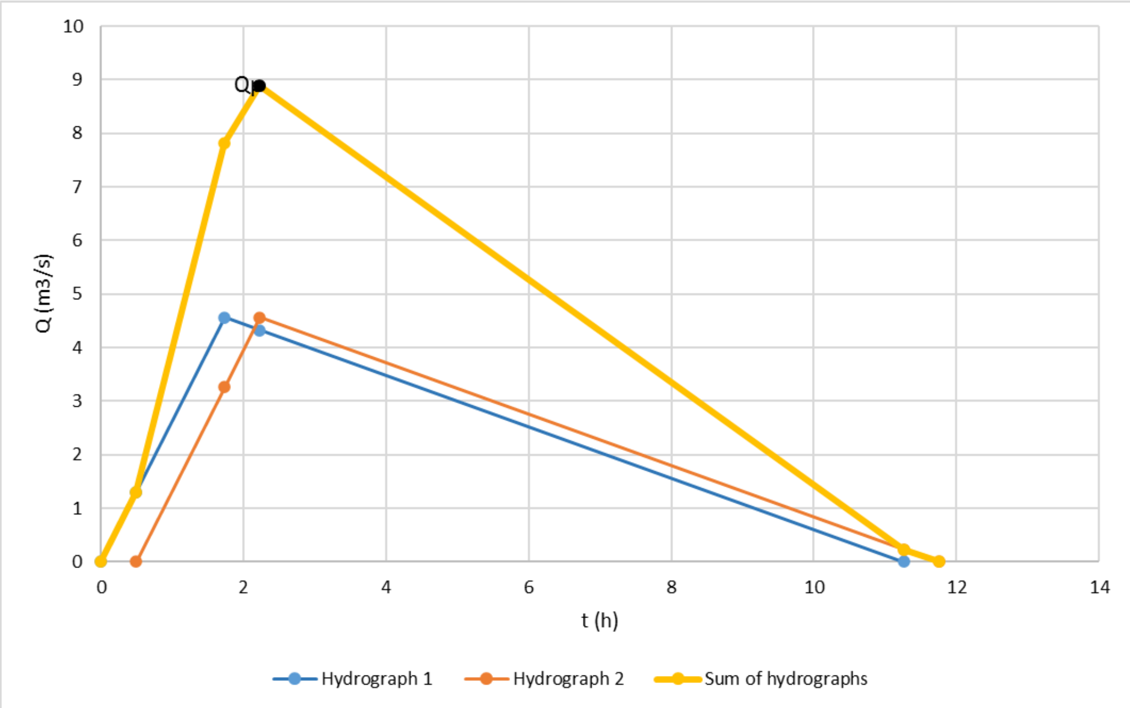
- $P_{n-\Delta D}$ = Net precipitation for a unit hydrograph (mm).
- A = Area of the Basin or drained surface, in km^2.
- t_b = Base time (hours).

Once the fundamental parameters defining the unit triangular hydrograph are known, the peak flows associated with them for the return period 100 years are calculated. The following are the values calculated for Q_O by Basin:

		Q_o (m^3/s) for T-100 years per Watershed				
D(h)	ΔD (h)	W1	W2	W3	W4	W5
0.1 · Tc	0.1 · Tc	4.794	8.468	0.779	0.062	0.178
0.2 · Tc	0.2 · Tc	6.445	11.195	1.093	0.095	0.260
0.4 · Tc	0.2 · Tc	4.563	7.820	0.800	0.074	0.196
0.6 · Tc	0.2 · Tc	3.697	6.294	0.659	0.063	0.164
0.8 · Tc	0.2 · Tc	3.174	5.381	0.571	0.056	0.143
1 · Tc	0.2 · Tc	2.815	4.759	0.511	0.051	0.129
1.2 · Tc	0.2 · Tc	2.550	4.300	0.465	0.047	0.118
1.4 · Tc	0.2 · Tc	2.343	3.945	0.429	0.043	0.109
1.6 · Tc	0.2 · Tc	2.177	3.659	0.400	0.041	0.102
1.8 · Tc	0.2 · Tc	2.039	3.424	0.376	0.039	0.096
2 · Tc	0.2 · Tc	1.923	3.226	0.356	0.037	0.091
2.2 · Tc	0.2 · Tc	1.823	3.055	0.338	0.035	0.087
2.4 · Tc	0.2 · Tc	1.737	2.908	0.323	0.034	0.083
2.6 · Tc	0.2 · Tc	1.660	2.778	0.309	0.032	0.080
2.8 · Tc	0.2 · Tc	1.592	2.662	0.297	0.031	0.077

7.11. CALCULATION OF PEAK FLOW

Finally, to calculate the peak flow in each Basin, it is necessary to add the flow values of the unit hydrographs proposed for each case of D, because they overlap in time. As an example, the following graph shows the case of D = 0.4 - Tc, in which the values of the 2 unit hydrographs must be added to obtain the value of the peak flow, which is the maximum value of Q obtained on the graph:



A similar procedure is followed in the other cases, changing the number of superimposed hydrographs as a function of the D considered. The peak flow values obtained for each Basin as a function of D are shown below, marking in green the maximum values for each return period studied:

			Q_p (m^3/s) for T-100 years per Watershed				
D(h)	**ΔD (h)**	**Nº superimposed hydrographs**	**W1**	**W2**	**W3**	**W4**	**W5**
0.1 · Tc	0.1 · Tc	0	4.794	8.468	0.779	0.062	0.178
0.2 · Tc	0.2 · Tc	1	6.445	11.195	1.093	0.095	0.260
0.4 · Tc	0.2 · Tc	2	8.890	15.233	1.558	0.145	0.381
0.6 · Tc	0.2 · Tc	3	10.515	17.902	1.874	0.180	0.465
0.8 · Tc	0.2 · Tc	4	11.707	19.847	2.108	0.206	0.528
1 · Tc	0.2 · Tc	5	12.613	21.321	2.288	0.227	0.577

			Q_p (m^3/s) for T-100 years per Watershed				
D(h)	**ΔD (h)**	**Nº superimposed hydrographs**	**W1**	**W2**	**W3**	**W4**	**W5**
1.2 · Tc	0.2 · Tc	6	13.311	22.449	2.428	0.243	0.615
1.4 · Tc	0.2 · Tc	7	13.846	23.310	2.537	0.257	0.645
1.6 · Tc	0.2 · Tc	8	14.249	23.953	2.620	0.267	0.668
1.8 · Tc	0.2 · Tc	9	14.540	24.413	2.682	0.275	0.686
2 · Tc	0.2 · Tc	10	14.736	24.715	2.726	0.281	0.699
2.2 · Tc	0.2 · Tc	11	14.848	24.880	2.753	0.285	0.707
2.4 · Tc	0.2 · Tc	12	**14.886**	**24.922**	**2.766**	0.287	0.712
2.6 · Tc	0.2 · Tc	13	14.856	24.854	2.765	**0.288**	**0.713**
2.8 · Tc	0.2 · Tc	14	14.765	24.685	2.753	0.288	0.711

Finally, the reference peak flow for each Basin is obtained by choosing the maximum from among the values of D considered, thus obtaining the following table of final peak flows for each Basin:

Watershed	**D(h)**	**ΔD (h)**	**Q_p (m^3/s) for T-100 years**
Watershed W1	2.4 · Tc	0.2 · Tc	14.886
Watershed W2	2.4 · Tc	0.2 · Tc	24.922
Watershed W3	2.4 · Tc	0.2 · Tc	2.766
Watershed W4	2.6 · Tc	0.2 · Tc	0.288
Watershed W5	2.6 · Tc	0.2 · Tc	0.713

As a result, the value of D varies according to the Basins and return periods considered, but it remains in any case between the values of 2.4 - Tc and 2.6 - Tc.

ANNEX 2. FLOOD STUDY

INDEX

1. INTRODUCTION

This "*Annex 2. Flood Study*" is carried out at the request of COBRA to determine the variation of flood risks in the area of the “PV BELLTOWN - BARRETT SOLAR” plant, located 84.9 km west of the city of Dallas (USA).

The following is the procedure followed:

- Generation of a 2D flood analysis model, carried out with the IBER program, version 3.2.2. This model will analyze the scope of the study plots in a sufficient area, both upstream, so that there is no influence caused by the concentration of inflow at a single point, and downstream, so that the modeled sinkholes do not affect the flood zone near the plots or can generate a backwater. The results obtained from the model are depths, velocities, elevations and specific discharge.
- The scope of the study plots is modeled in the program, and their flood risk is obtained by introducing a hyetograph of alternating blocks of precipitation with a total duration of 24 hours subdivided into blocks of 1 hour. This hyetograph is obtained by multiplying the design intensity extracted from the IDF curves calculated in the "*Annex 1 Climatological and Hydrological Study*". In the IBER model itself, the curve numbers corresponding to the study area are input so that the program will obtain the corresponding runoff.

2. DETERMINATION OF THE DESIGN RAINFALL

In the case of the present study, a design rainfall will be introduced in the study area. The design rainfall will be entered into the IBER model as a hyetograph reflecting the distribution of rainfall over the rainiest 24 hours that can occur in the study area for the 100 years return period.

There is a wide variety of procedures for the construction of design hyetographs, several of which are based on Intensity-Duration-Frequency curves. In this case, the alternating block methodology (alternating block method, Chow et al. 1994) will be used. Next, we will explain the procedure followed to make a 24-hour-long hyetograph of a downpour, with time increments of 1 hour, for a return period of 100 years.

First, the IDF curve will be selected, the expression and graphic representation appears in "*Annex 1 Climatological and Hydrological Study*". In this case, the expression of the IDF curve is as follows:

$$I = \frac{358.9128 \cdot T^{0.205167}}{t^{0.61639}}$$

Where:

- T= Return period (years)
- t= Precipitation duration time (min)

Figure 1 shows the graphical representation of the IDF curve for the return period considered in the "*Annex 1 Climatological and Hydrological Study"*.

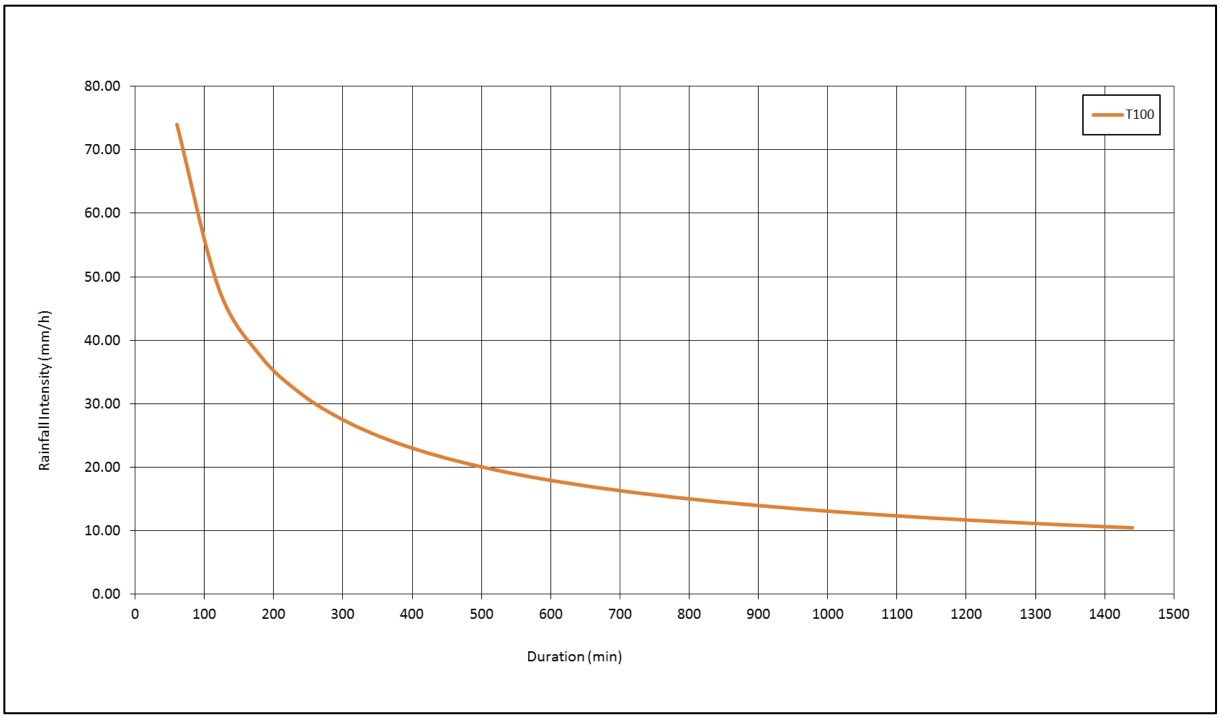


Figure 1. IDF curve according to the return period considered.

Applying the above formula for T-100, precipitation intensities (in mm/h) will be obtained for various time increments from 1 to 24 hours. Then, the design rainfall (in mm) at each time interval will be calculated by multiplying the rainfall intensity by the duration of the interval. With the obtained values of design rainfall, the precipitation fallen in one-hour increments will be calculated. The calculation process followed for the preparation of the hyetograph is exemplified in Table 2.

Table 1. Calculation of the design hyetograph for T100.

t (h)	I (mm/h)	P (mm)		ΔP (mm)	
1	74.01	74.01 · 1 =	74.01	74.01 =	74.01
2	48.28	48.28 · 2 =	96.56	96.56 - 74.01 =	22.54
3	37.60	37.6 · 3 =	112.80	112.8 - 96.56 =	16.25
4	31.49	31.49 · 4 =	125.97	125.97 - 112.8 =	13.16
5	27.44	27.44 · 5 =	137.22	137.22 - 125.97 =	11.26
6	24.53	24.53 · 6 =	147.17	147.17 - 137.22 =	9.94
7	22.30	22.3 · 7 =	156.13	156.13 - 147.17 =	8.96
8	20.54	20.54 · 8 =	164.34	164.34 - 156.13 =	8.21
9	19.10	19.1 · 9 =	171.93	171.93 - 164.34 =	7.60
10	17.90	17.9 · 10 =	179.02	179.02 - 171.93 =	7.09
11	16.88	16.88 · 11 =	185.69	185.69 - 179.02 =	6.67
12	16.00	16 · 12 =	191.99	191.99 - 185.69 =	6.30
13	15.23	15.23 · 13 =	197.98	197.98 - 191.99 =	5.99
14	14.55	14.55 · 14 =	203.69	203.69 - 197.98 =	5.71
15	13.94	13.94 · 15 =	209.15	209.15 - 203.69 =	5.46
16	13.40	13.4 · 16 =	214.39	214.39 - 209.15 =	5.24
17	12.91	12.91 · 17 =	219.44	219.44 - 214.39 =	5.04
18	12.46	12.46 · 18 =	224.30	224.3 - 219.44 =	4.86
19	12.05	12.05 · 19 =	229.00	229 - 224.3 =	4.70
20	11.68	11.68 · 20 =	233.55	233.55 - 229 =	4.55
21	11.33	11.33 · 21 =	237.97	237.97 - 233.55 =	4.41
22	11.01	11.01 · 22 =	242.25	242.25 - 237.97 =	4.28
23	10.71	10.71 · 23 =	246.42	246.42 - 242.25 =	4.17
24	10.44	10.44 · 24 =	250.47	250.47 - 246.42 =	4.06

For the construction of the hyetograph, the values in the last column of the previous table will be used. These values will be ordered taking into account that the maximum intensity will be the central value of the hyetograph, while the remaining values will be placed in decreasing order alternately to the right and left of the central block. Thus, the resulting hyetograph for a return period of 100 years is as follows:

Table 3. Design hyetograph for T100.

t (h)	I (mm/h)
1	4.17
2	4.41
3	4.70

t (h)	I (mm/h)
4	5.04
5	5.46
6	5.99
7	6.67
8	7.60
9	8.96
10	11.26
11	16.25
12	74.01
13	22.54
14	13.16
15	9.94
16	8.21
17	7.09
18	6.30
19	5.71
20	5.24
21	4.86
22	4.55
23	4.28
24	4.06

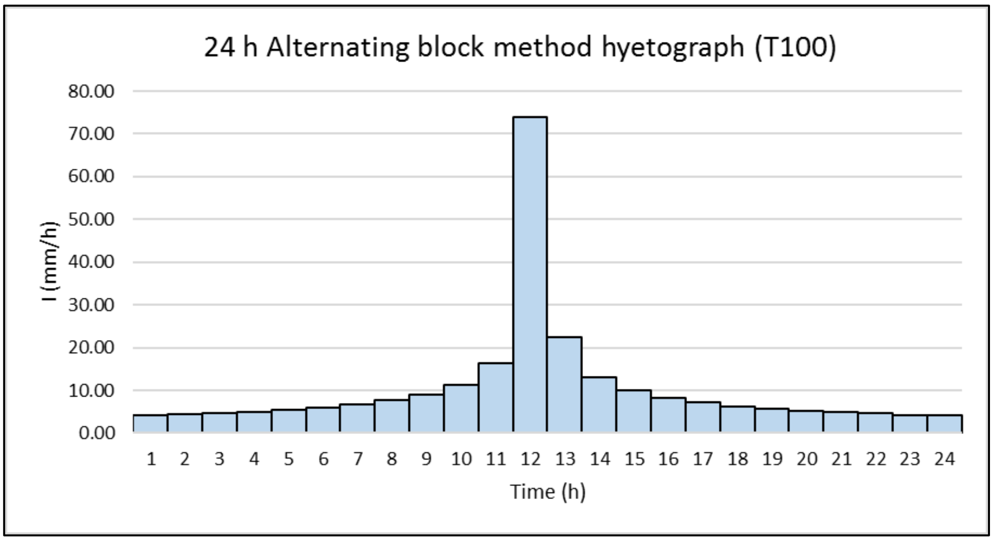


Figure 2. 24-hour alternating block method of precipitation for T100.

As previously mentioned, in the construction of the IBER model a design rainfall will be used for each return period by distributing the rainfall in 24 hours by the alternating block method. In addition to the above, a raster file including the curve number (CN) values obtained in "*Annex 1 Climatological and Hydrological Study*" will be introduced in the IBER model. Thus, the program itself will interpret at each point the degree of existing permeability and obtain the actual runoff.

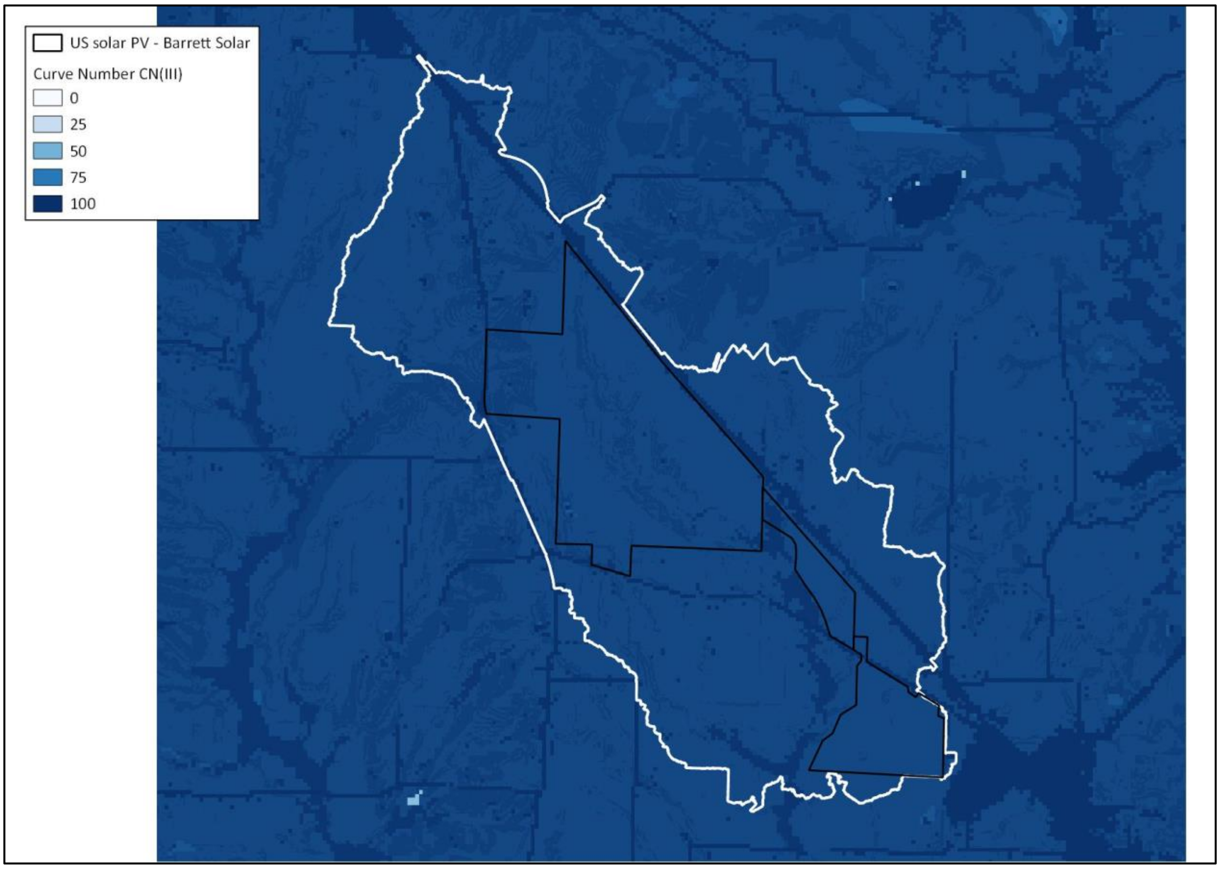


Figure 3. Map with the CN curve numbers input into the IBER model.

3. DETERMINATION OF FLOOD ZONES, DEPTHS AND VELOCITIES

This section will explain the procedure followed for the calculation of the flood zones, depths, and velocities. The results will be presented in the form of plans in the final part of this document.

3.1. METHODOLOGY

For the determination of flood risk zones, depths, and velocities for the intensity of precipitation associated with the return period of 100, their respective analysis models are carried out in the IBER software.

The steps followed to obtain the results are as follows:

- Model generation.
- Assigning boundary and initial conditions to the model.
- Obtaining results.

3.2. MODEL GENERATION

The modeled area corresponds to the watersheds that affect the study plot. The model inputs are:

- Precipitation spread over the entire model grid.
- Raster file with the CN curve numbers under consideration.
- The model's outlet drains are located outside the plots to be studied, as there are no obstacles in the vicinity that could obstruct the flow of water.

The model width must be sufficient to avoid edge effect at the model boundaries. This is verified at the end of the calculation, when it is observed that the flooding zone is not limited by the model boundaries.

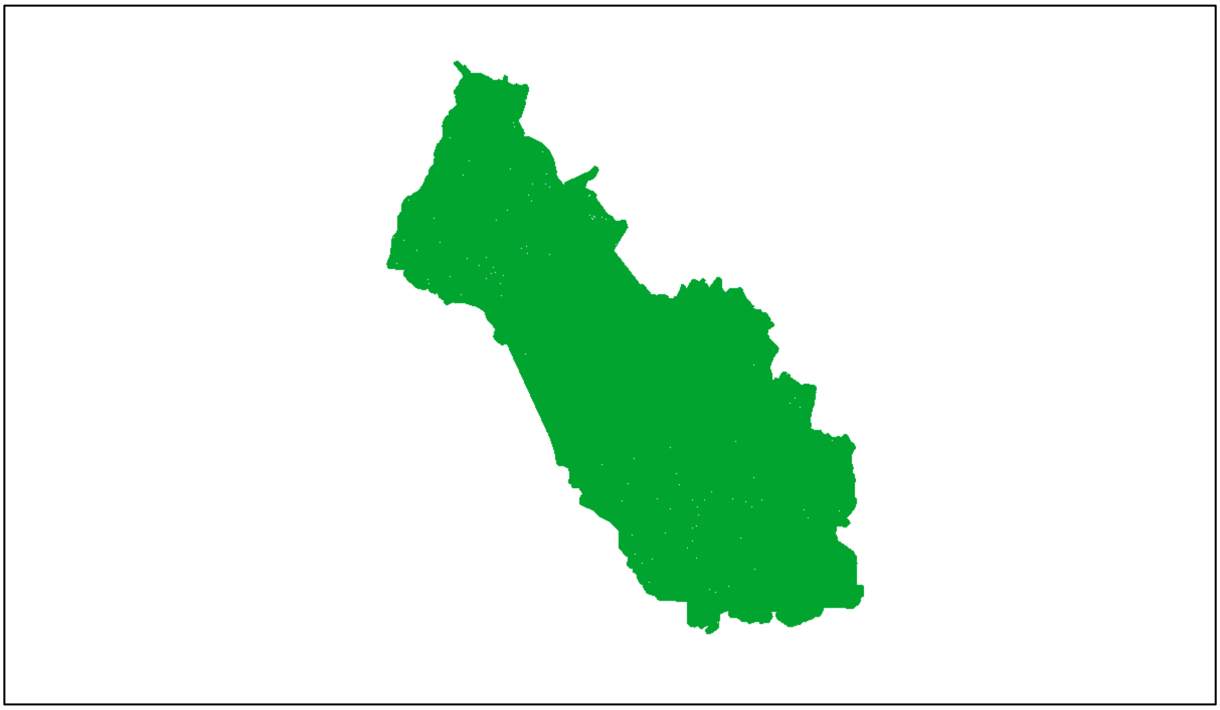


Figure 4. Model limits.

3.3. ASSIGNMENT OF INITIAL MODEL CONDITIONS

To perform the analysis, the following initial values must be specified:

- Terrain elevations: the heights of the mesh nodes are assigned from the elevation values of a Digital Elevation Model (DEM):
  - Digital Elevation Model with 1x1 m pixel resolution extracted from the East Texas Lidar project. This lidar collection covers portions of east Texas. This project was managed by the Texas Strategic Mapping (StratMap) Program and utilized the StratMap contracts held by the Department of Information Resources (DIR). Collection took place December 29, 2016, through April 14, 2017. Data was acquired and processed by Fugro EarthData, Inc. with third party quality assurance/quality control by AECOM.

Figure 8. Model and elevations.

- Initial depth: a value of 0 is assigned, corresponding to totally dry soil without any type of surface runoff.
- Terrain roughness: a Manning's coefficient value of 0.05 is assigned, which corresponds to an average value for a pasture land use.
- Precipitation: the precipitation intensities previously calculated for each case are assigned to the entire surface of the model.
- The model output points have been defined on the perimeter of the model under study.

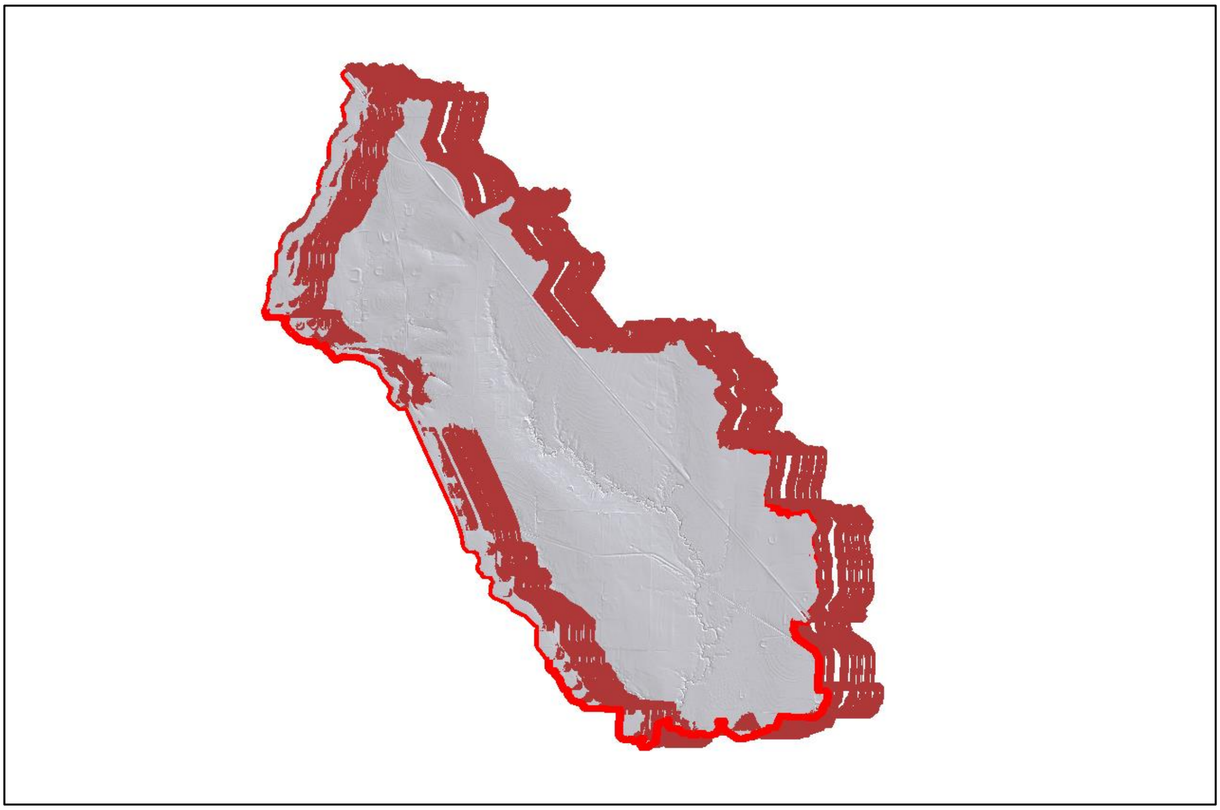


Figure 9. Model output contour.

3.4. ANALYSIS AND RESULTS

The analysis is carried out for a period of 90000 seconds (25 hours), sufficient to have stabilized the results, and to have allowed some movement in the precipitation and stream water. Next, the flood zones produced for T100 will be analyzed, the results obtained for the rest of the return periods are attached in the plans included at the end of this document.

From the analysis carried out, we obtain the graphic of depths for the 100-year return period. Flood depths of less than 5 cm have been discarded as they are considered irrelevant for this project. The following image shows the associated depths on the study plot:

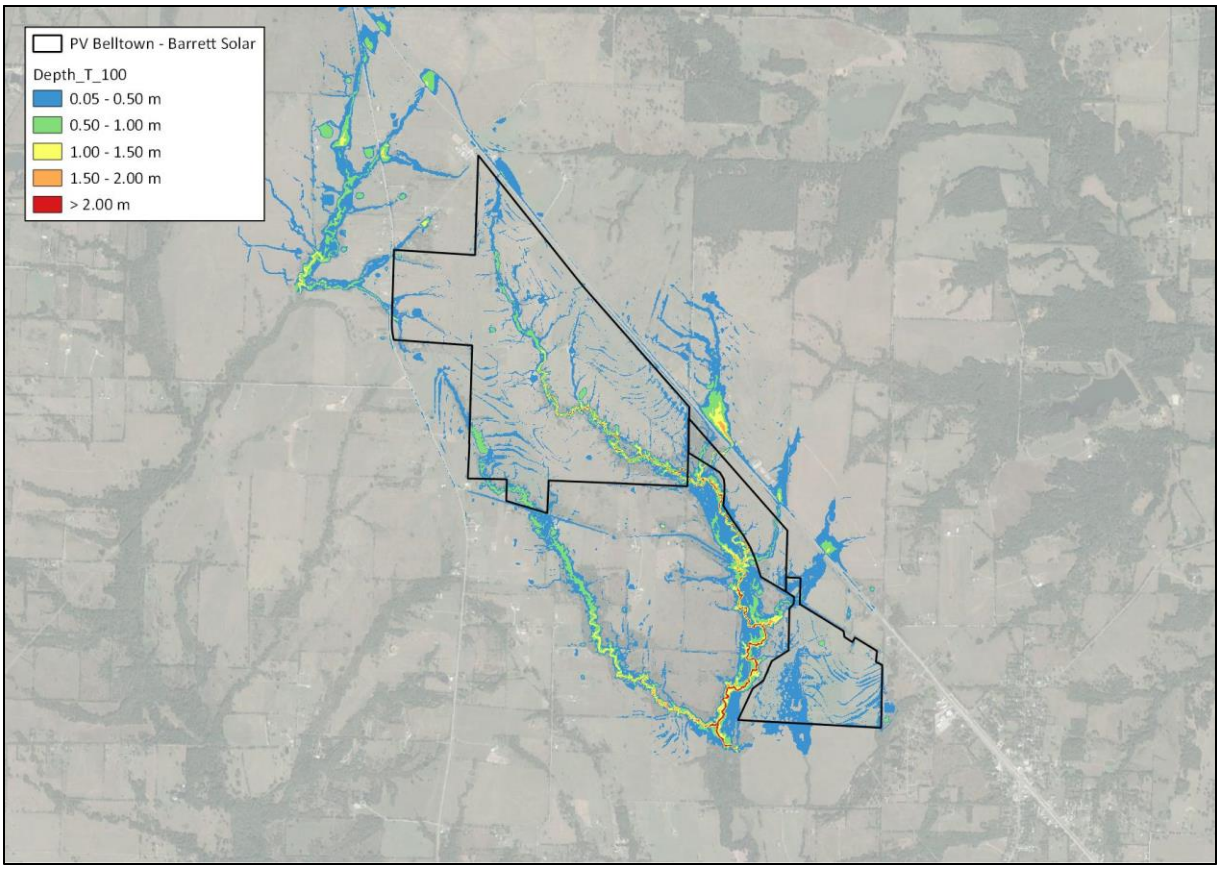


Figure 10. Maximum flooding depths obtained in each cell for a return period of 100 years.

The greatest impact is on the watercourse that crosses the central part of the northern plot, where depths up to 2.78 m occur. In the rest of the area there are depths that generally remain below 0.50 m and are distributed along its surface.

In addition to the depths, the velocity graph for the return periods considered is also extracted from the analysis. Below is a snapshot of the results obtained for a return period of 100 years. Velocities lower than 10 cm/s have been discarded as they are considered irrelevant for this project. The following image shows the associated velocities in the study plot:

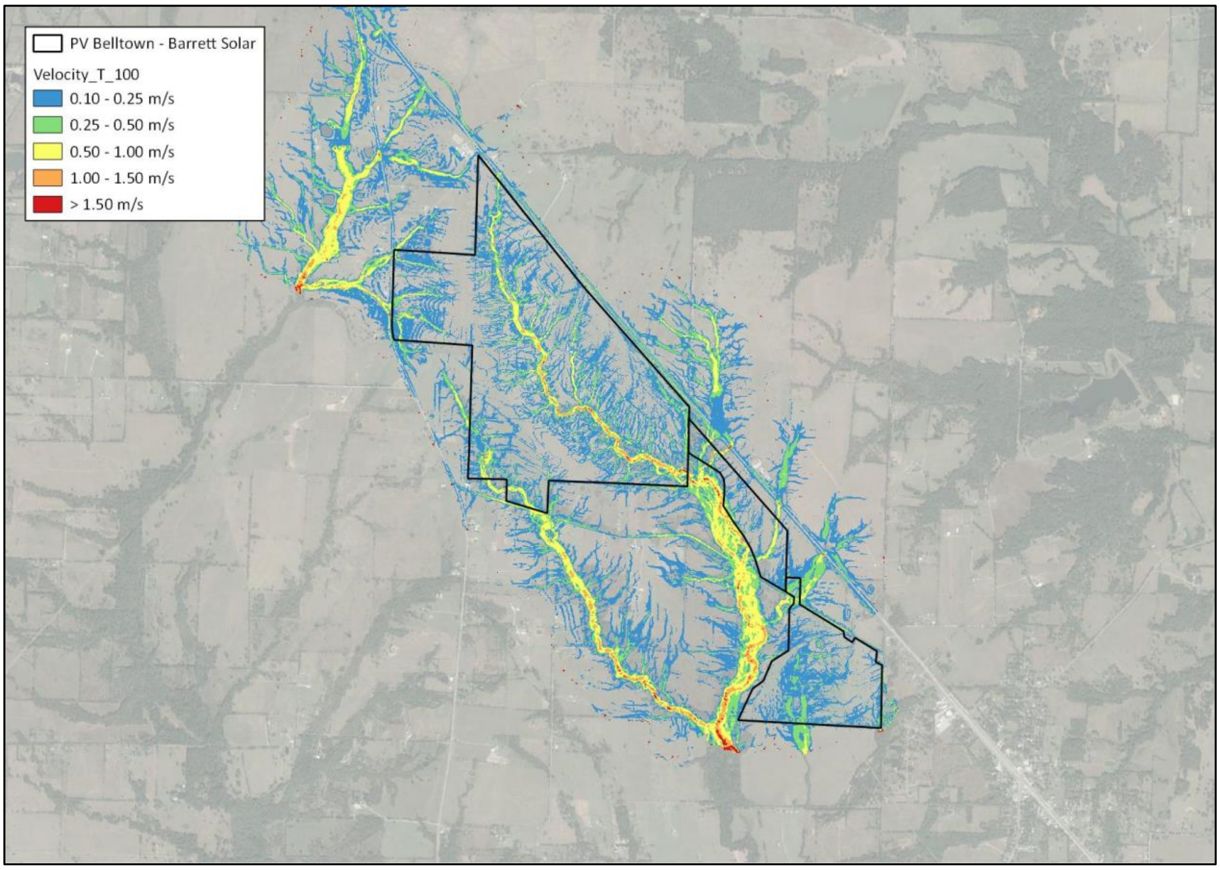


Figure 13. Maximum flood velocities obtained in each cell for a return period of 100 years.

The highest velocities occur in the watercourse that crosses the central part of the northern plot, where maximum velocities of 3.95 m/s are obtained. Despite this maximum value, the most common velocity values are low, generally below 0.50 m/s in the case of streams and 0.25 m/s outside them.

4. CONCLUSIONS

From the model made with the IBER software, it has been possible to determine the flood level of the study plots for a return period of 100 years.

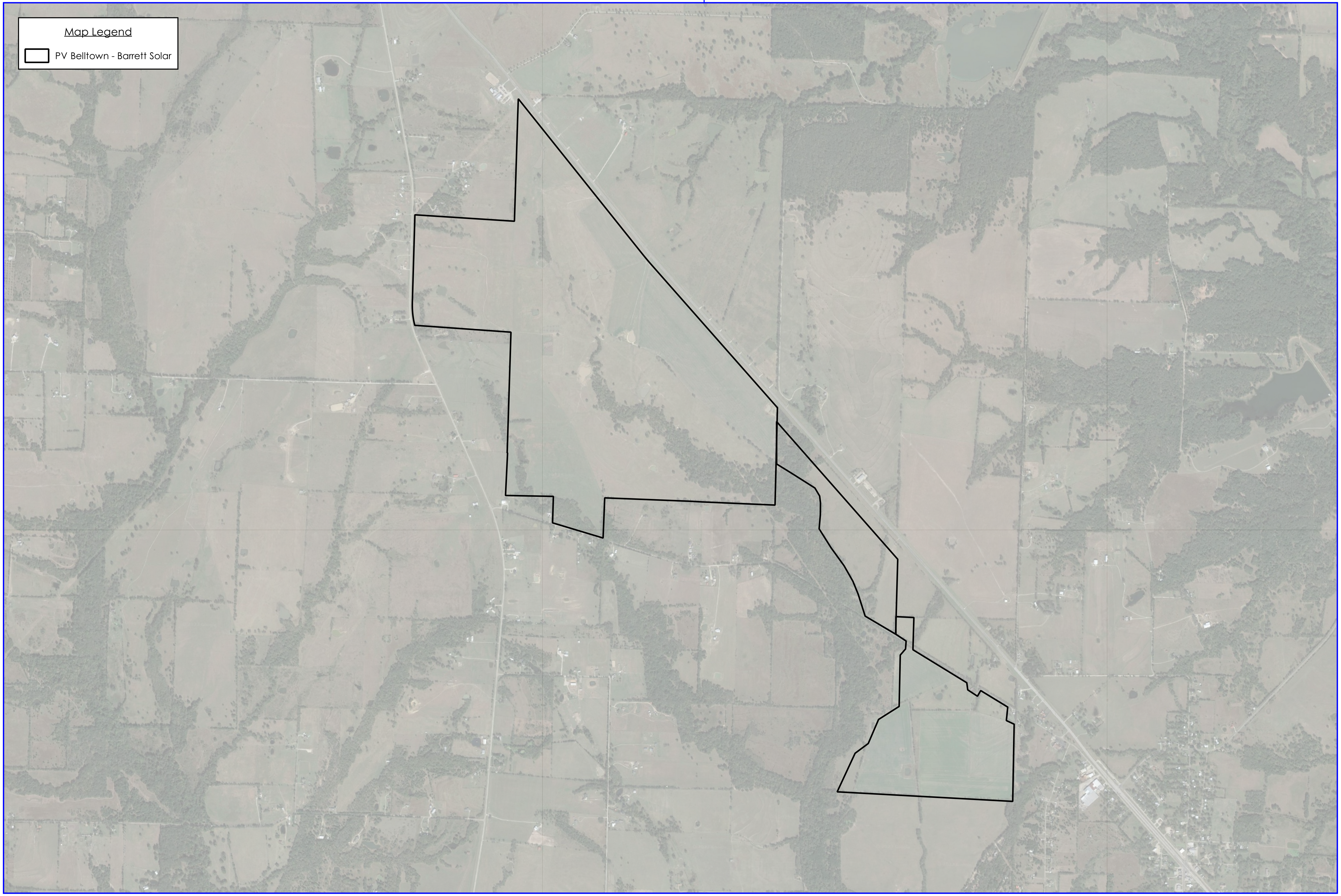
In terms of depths, the greatest impact is on the watercourse that crosses the central part of the northern plot, where depths up to 2.78 m occur. In the rest of the area there are depths that generally remain below 0.50 m and are distributed along its surface.

Regarding flood velocities, the most affected areas are around the watercourse that crosses the central part of the northern plot, where maximum velocities of 3.95 m/s are obtained. Despite this maximum value, the most common velocity values are low, generally below 0.50 m/s in the case of streams and 0.25 m/s outside them.

DRAWINGS BARRETT SOLAR

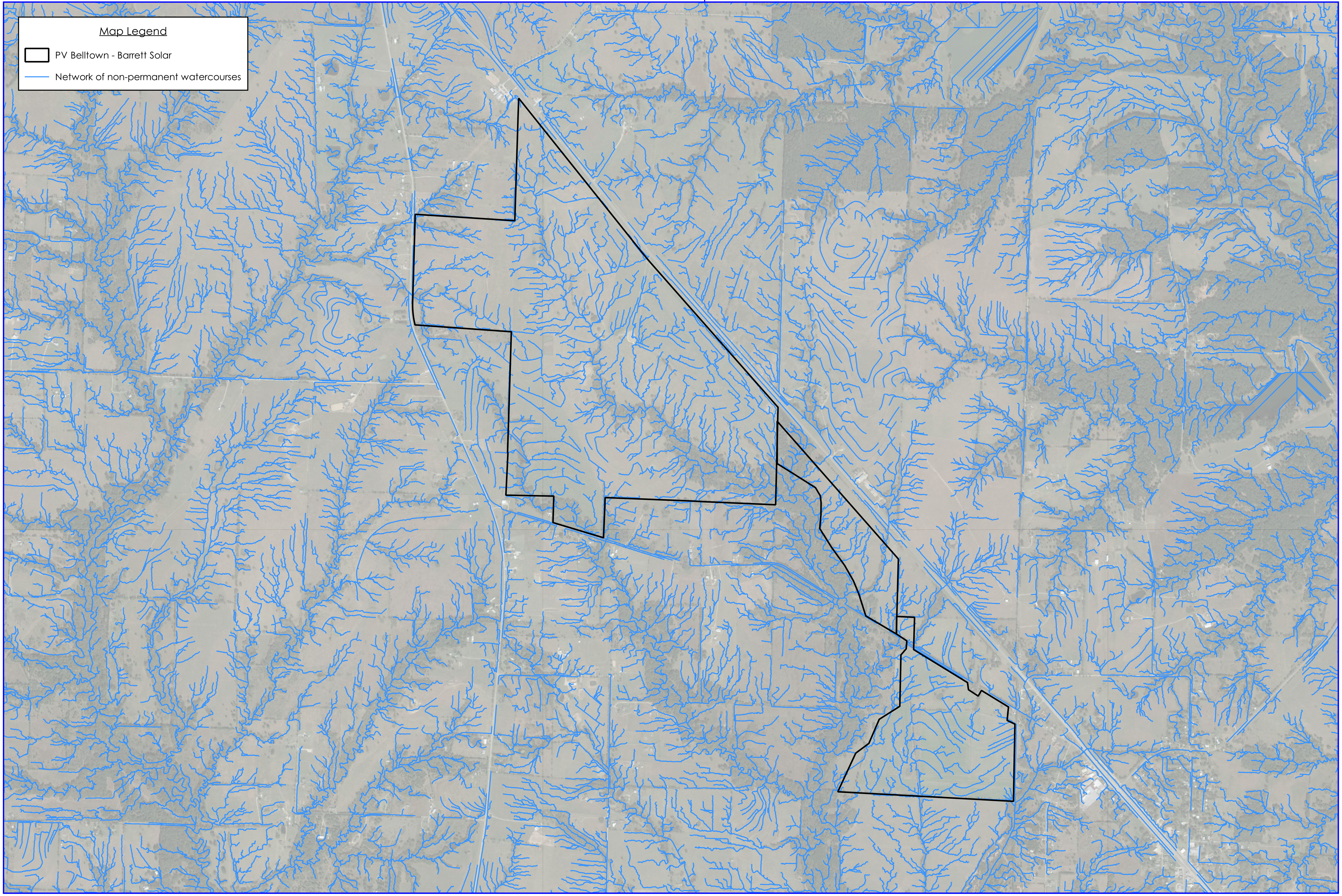
INDEX OF DRAWINGS

DRAWING Nº1: ORTHOIMAGERY



www.zubiaingenieros.com	ENGINEER AUTHOR OF THE STUDY: D. JAVIER ZUBÍA Civil Engineer Registration number: 12773	SCALE: 1: 20000 (0 — 150 — 300 m) FORMAT A3	PROJECT: PV BELLTOWN - BARRETT SOLAR	DATE: July 2023	TITLE: ORTHOIMAGERY	PROJECT KEY: 038_23_B SHEET 01 OF 08

DRAWING Nº2: NETWORK OF NON-PERMANENT WATERCOURSES



ENGINEER AUTHOR OF THE STUDY:	SCALE:	PROJECT:	DATE:	TITLE:	PROJECT KEY:
D. JAVIER ZUBÍA Civil Engineer Registration number: 12773	1: 20000 0 150 300 m FORMAT A3	PV BELLTOWN - BARRETT SOLAR	July 2023	NETWORK OF NON-PERMANENT WATERCOURSES	038_23_B SHEET 02 OF 08

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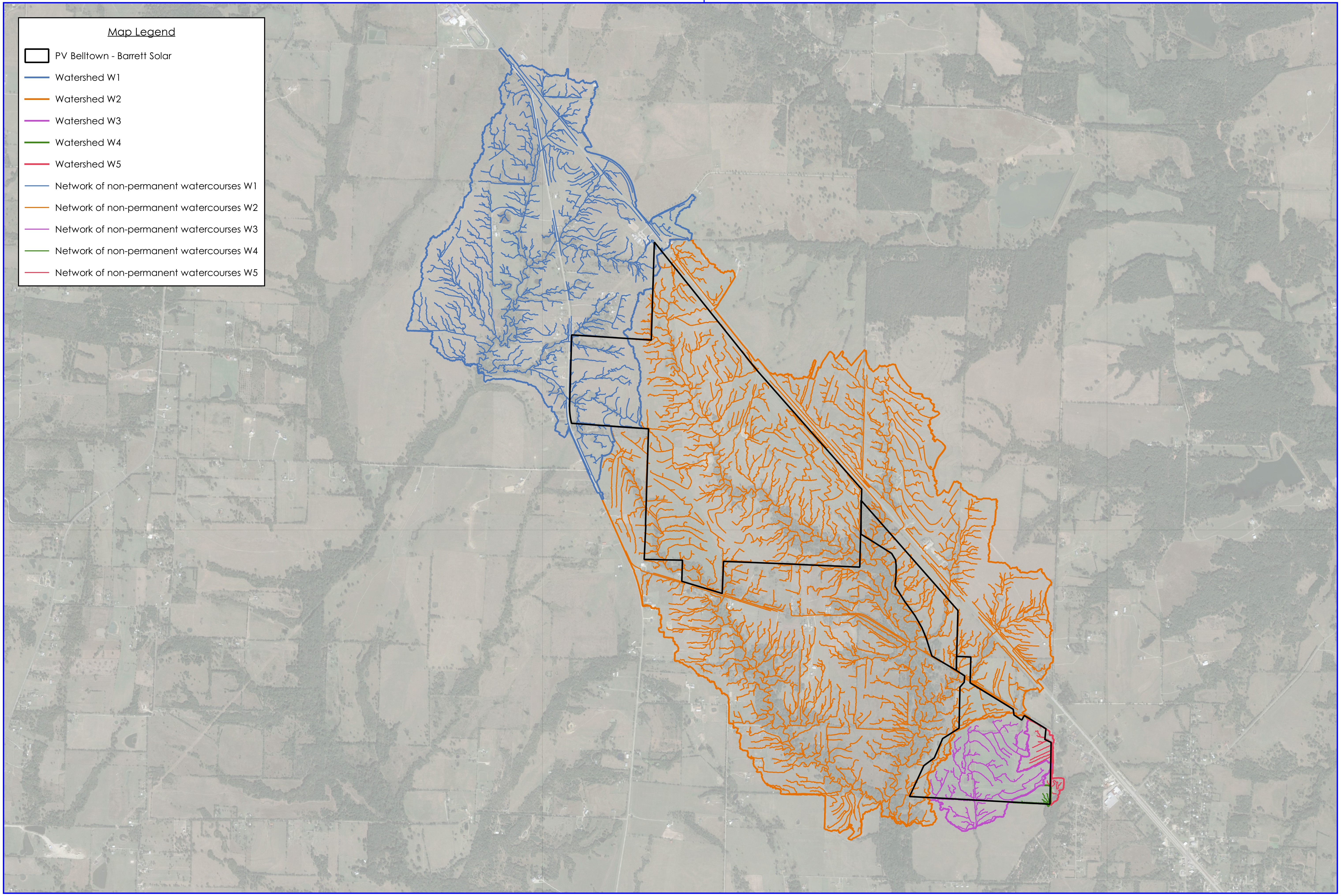
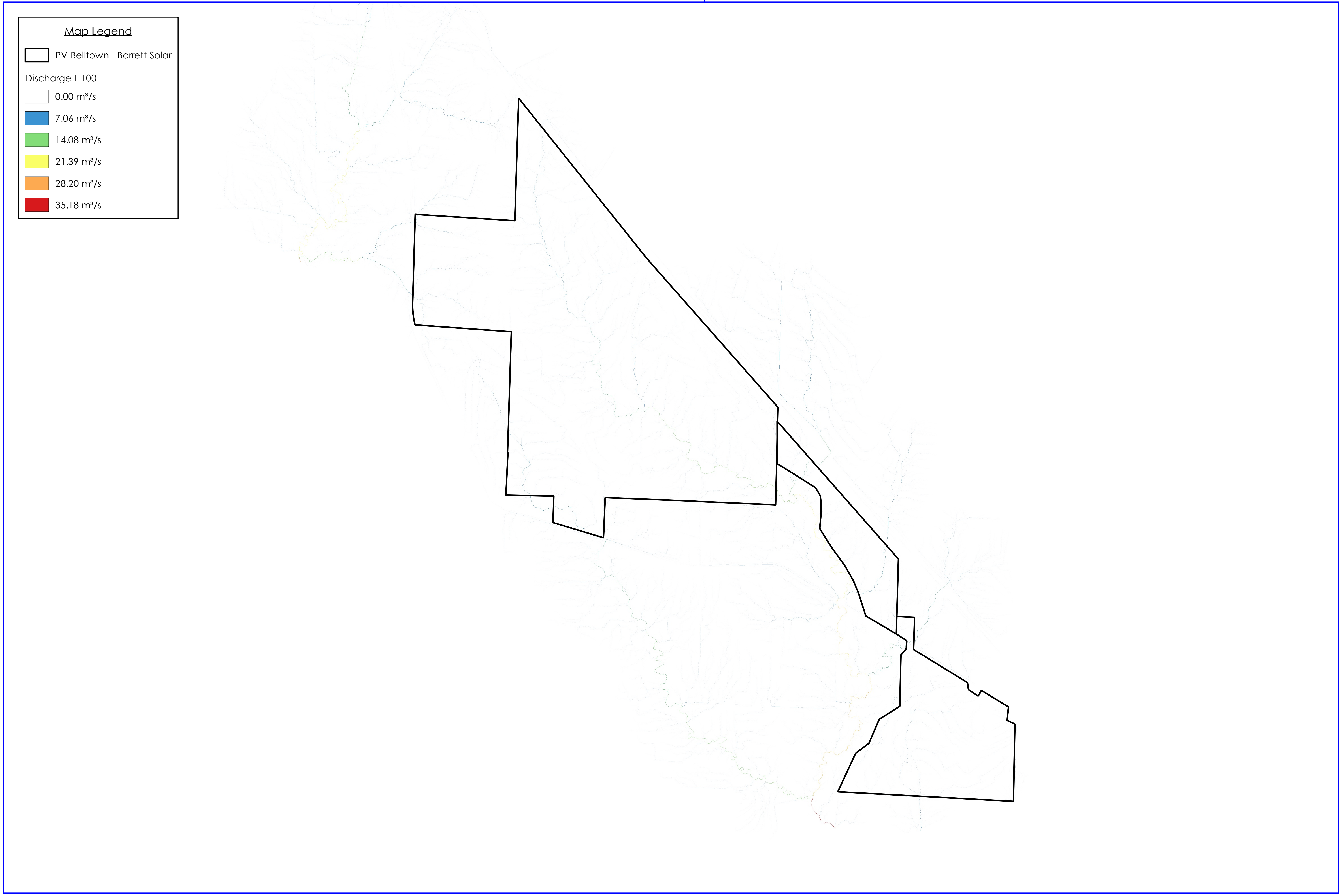
DRAWING Nº3: WATERSHEDS



www.zubiaingenieros.com	ENGINEER AUTHOR OF THE STUDY: D. JAVIER ZUBIA Civil Engineer Registration number: 12773	SCALE: 1: 25000 (0 300 600 m) FORMAT A3	PROJECT: PV BELLTOWN - BARRETT SOLAR	DATE: July 2023	TITLE: WATERSHEDS	PROJECT KEY: 038_23_B SHEET 03 OF 08

DRAWING Nº4: DISCHARGE T-100 YEARS

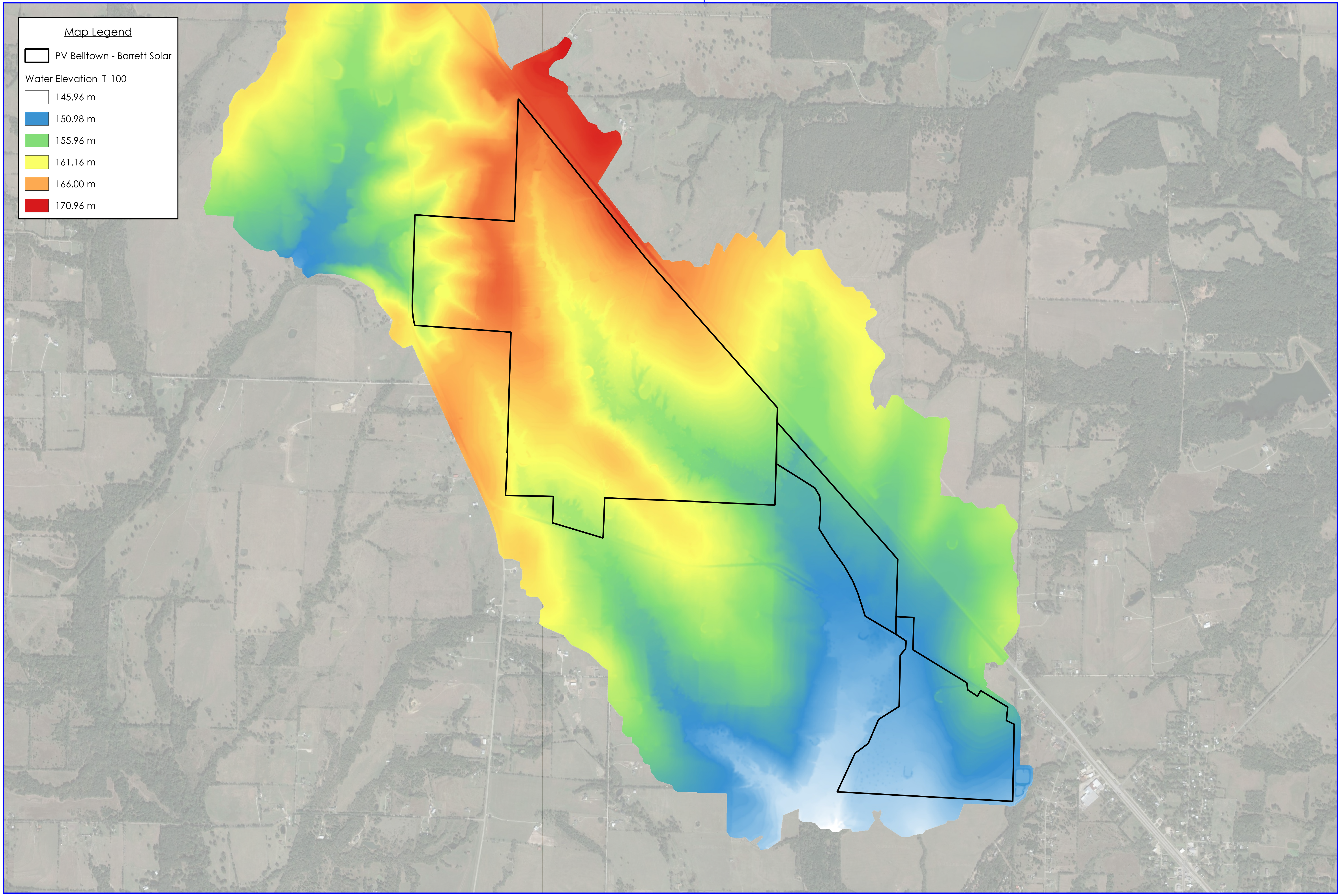


www.zubiaingenieros.com	ENGINEER AUTHOR OF THE STUDY: D. JAVIER ZUBÍA, Civil Engineer, Registration number: 12773	SCALE: 1: 20000 (0 – 150 – 300 m), FORMAT A3	PROJECT: PV BELLTOWN - BARRETT SOLAR	DATE: July 2023	TITLE: DISCHARGE T-100 YEARS

PROJECT KEY: 038_23_B

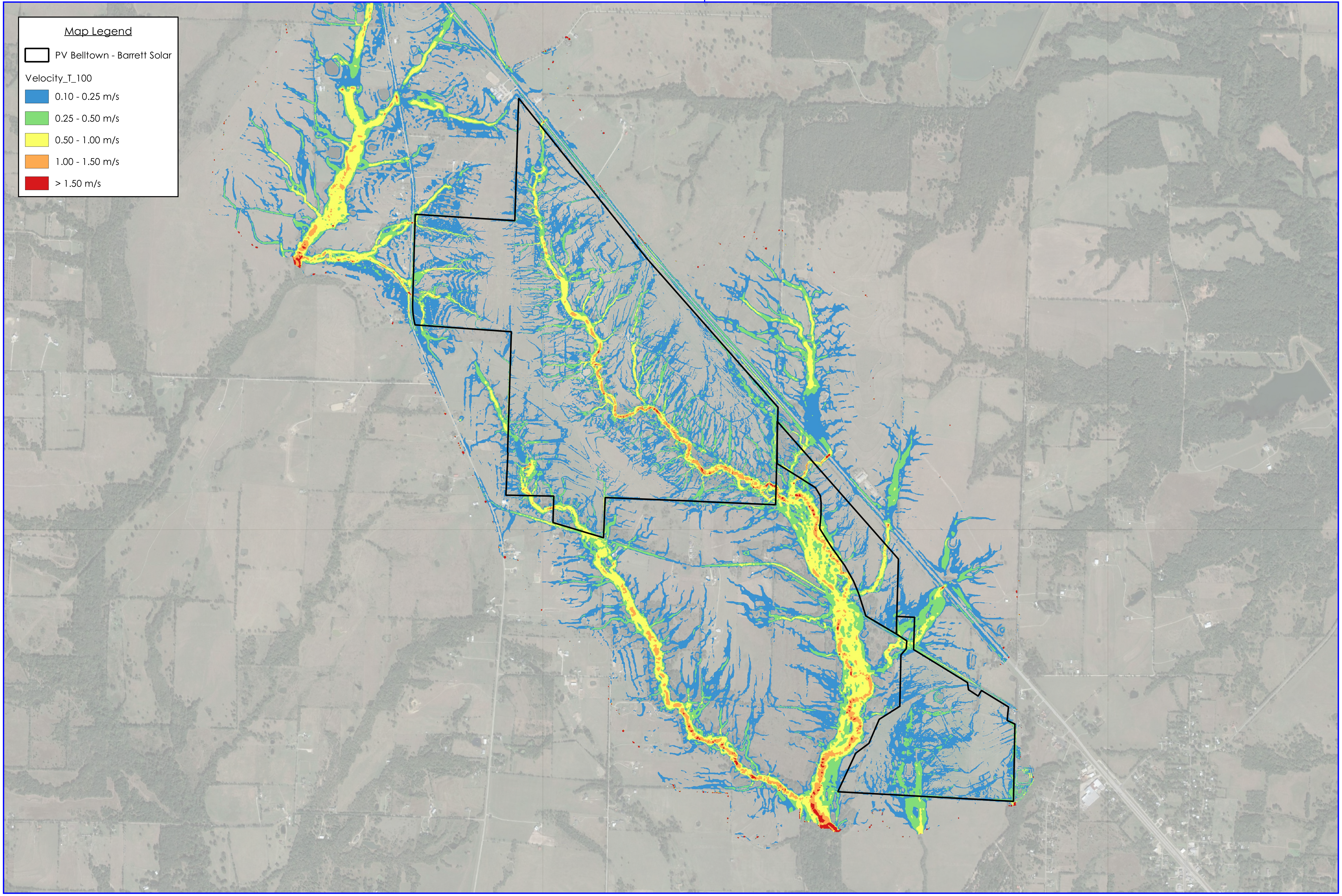
SHEET 04 OF 08

DRAWING Nº5: WATER ELEVATION T-100 YEARS



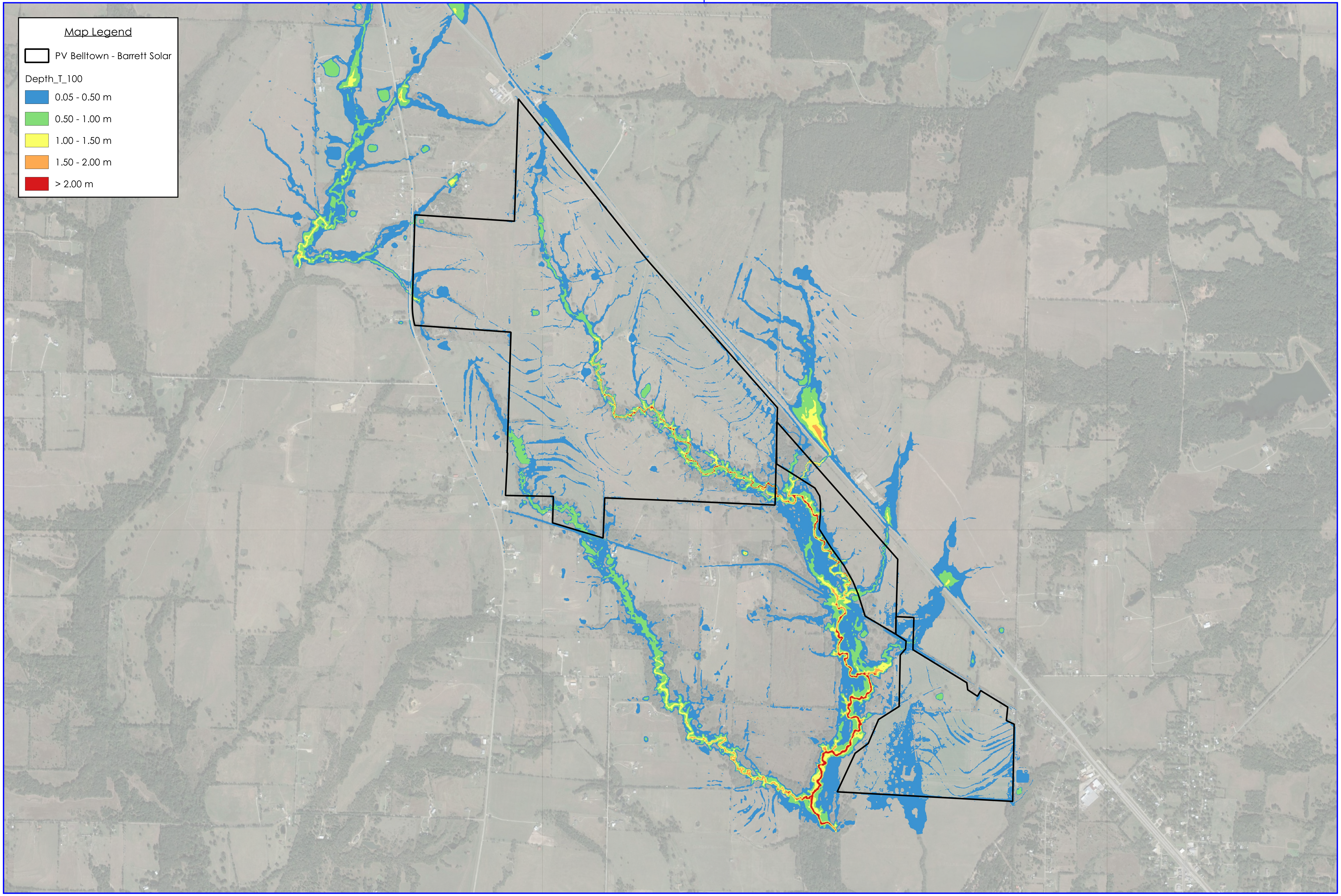
www.zubiaingenieros.com	ENGINEER AUTHOR OF THE STUDY: D. JAVIER ZUBIA, Civil Engineer, Registration number: 12773	SCALE: 1: 20000 (0 – 150 – 300 m), FORMAT A3	PROJECT: PV BELLTOWN - BARRETT SOLAR	DATE: July 2023	TITLE: WATER ELEVATION T-100 YEARS	PROJECT KEY: 038_23_B, SHEET 05 OF 08

DRAWING Nº6: VELOCITY T-100 YEARS



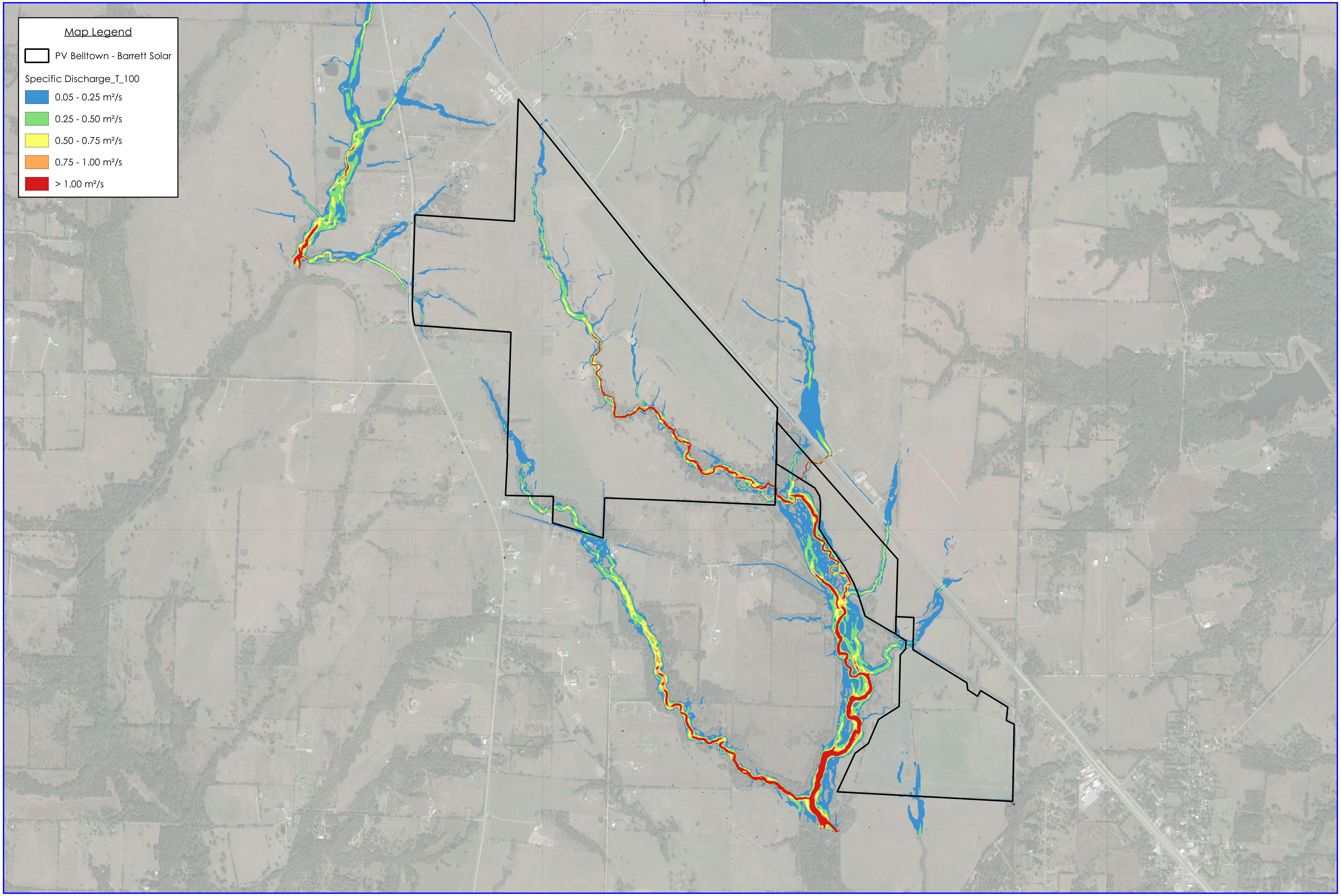
www.zubiaingenieros.com	ENGINEER AUTHOR OF THE STUDY: D. JAVIER ZUBÍA Civil Engineer Registration number: 12773	SCALE: 1: 20000 (0 150 300 m) FORMAT A3	PROJECT: PV BELLTOWN - BARRETT SOLAR	DATE: July 2023	TITLE: VELOCITY T-100 YEARS	PROJECT KEY: 038_23_B SHEET: 06 OF 08

DRAWING Nº7: DEPTH T-100 YEARS



www.zubiaingenieros.com	ENGINEER AUTHOR OF THE STUDY: D. JAVIER ZUBIA, Civil Engineer, Registration number: 12773	SCALE: 1: 20000 (0 – 150 – 300 m), FORMAT A3	PROJECT: PV BELLTOWN - BARRETT SOLAR	DATE: July 2023	TITLE: DEPTH T-100 YEARS	PROJECT KEY: 038_23_B, SHEET 07 OF 08

DRAWING Nº8: SPECIFIC DISCHARGE T-100 YEARS



www.zubiaingenieros.com	ENGINEER AUTHOR OF THE STUDY: D. JAVIER ZUBIA, Civil Engineer, Registration number: 12773	SCALE: 1: 20000 (0 – 150 – 300 m) FORMAT A3	PROJECT: PV BELLTOWN - BARRETT SOLAR	DATE: July 2023	TITLE: SPECIFIC DISCHARGE T-100 YEARS	PROJECT KEY: 038_23_B SHEET 08 OF 08