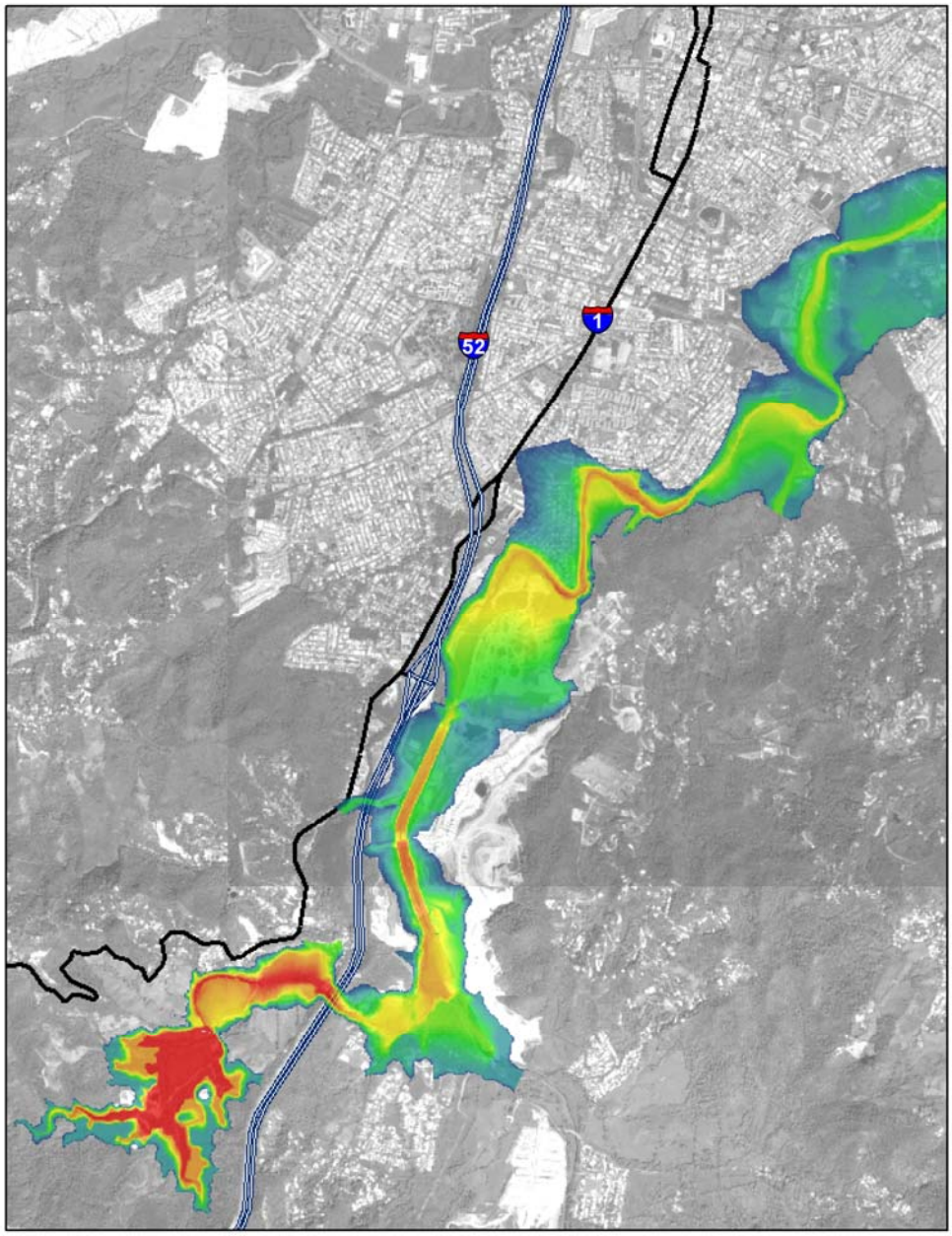
Dam Break Flood Hazard Analysis for Quebrada Beatriz Reservoir



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Dam Break Flood Hazard Analysis for Quebrada Beatriz Offstream Reservoir - Caguas, P.R.

1 INTRODUCTION

1.1 Project Description & Location

The Puerto Rico Infrastructure Financing Authority (AFI), through the "**Planning and Feasibility Studies for Potential Dam Sites in Puerto Rico**", has identified a site in Quebrada Beatriz for the construction of an offstream reservoir which could provide a safe water yield of 14 million gallons per day (MGD) with a storage capacity of 7.0 million cubic meters (Mm^3). This site was previously studied by the United States Corps of Engineers (CoE) in 1997 and the United States Geological Survey (USGS) in 2001.

The proposed dam is located in the Beatriz ward approximately six kilometers southeast of the town of Caguas. The proposed strategy to develop this site is to impound flows from Quebrada Beatriz with an earth embankment dam located approximately 2.2 kilometers upstream of its confluence with Río Turabo. The dam structure is designed to have a maximum height of 36 meters and a length of nearly 450 meters between abutments. Embankments slopes were set to 3:1 and 2.5:1 in the upstream and downstream sides of the reservoir respectively.

The entire spillway structure will be constructed over native abutment material and not on the fill embankment itself. A super-critical spillway chute followed by a downstream stilling basin was selected as the optimal configuration. The spillway crest was designed as a labyrinth weir to enable the conveyance of a high specific discharge and allow better matching of spillway crest width to the downstream chute width.

Access to the reservoir site will be through roads PR-1 or PR-765. The dam will be located nearly 800 meters southwest of the intersection of theses two roads. **Figure 1** illustrates the configuration of the reservoir in the USGS topographic quadrangle for Caguas.

1.2 Scope & Purpose of Report

The purpose of this study is to determine the water surface levels and the extent of flooding caused by a dam failure. Three conditions were analyzed: (1) a sunny day piping dam failure event, (2) a dam failure coincident with the 100 year flood, and (3) a dam failure coincident with the PMP flood. These dam failure scenarios will be used as the basis for the development of emergency action plans and are intended to serve as support for the dam security and emergency action plan.

1.3 Authorization

Preparation of this report was authorized by the Puerto Rico Infrastructure Financing Authority (AFI) by contract with Gregory L. Morris Engineering.

1.4 Parties Involved in Project

Project Developer:	Puerto Rico Infrastructure Financing Authority (AFI)
Engineers:	Gregory Morris P.E., Ph.D.
	Carlos Calderón P.E.
	Juan Portalatín, E.I.T.

1.5 Study Limitations

The hydraulic analysis was performed within by a one-dimensional unsteady flow hydraulic model. Model parameters, such as hydraulic roughness, were estimated based on observed field conditions and interpretation of aerial photography. Model cross sections were derived from contracted survey data, USGS bathymetric survey data, USGS topographic mapping, field measurements taken by GPS, and by geometry from FEMA modeling as available. The dam rupture event was simulated using standard breach models.

Results presented in this report should be considered consistent with the level of the best standards of practice in the field, but they should not be considered exact or infallible. For this reason, in any planning or emergency action related to this event, some uncertainty should be accounted for by providing an adequate margin of safety.

2 STUDY AREA DESCRIPTION

2.1 Watershed Area and Location Map

The watershed area tributary to the reservoir is presented in **Figure 1**. This site has a fairly large watershed area considering that it is configured as an offstream reservoir supplied by Río Turabo. The most important characteristics of this reservoir and its watershed are listed in **Table 1**.

Table 1: Characteristics of Quebrada Beatriz Reservoir.

Parameter	Quebrada Beatriz Reservoir
Watershed Area (km^2)	11.41
Type of Dam	Earthfill
Height of Dam (m)	33
Spillway	Labirynth & Supercritical chute
Max Surface Area of Reservoir (km^2) a/	0.67
Proposed reservoir volume (Mm^3)	7.0

a/ At spillway crest elevation

2.2 Study Reach

A study reach of approximately 30 kilometers extending from the proposed location of the Quebrada Beatriz Reservoir to the Lago Loíza dam was modeled. Portions of Quebrada Beatriz, Río Turabo, and Río Grande de Loíza were analyzed. The major tributaries to the studied reach include: Quebrada de las Quebradillas, Río Turabo, Río Grande de Loíza, Río Cagüitas, Río Bairoa, Río Gurabo, and Río Cañas. Much of the modeled reach runs through urban areas within the municipality of Caguas.

The area drained by the studied reach and its tributaries is illustrated in **Figure 2** showing the watershed limits for the modeled basins and the principal structures along its channel up to the Lago Loíza Reservoir. The Lago Loíza dam is the only mayor structure impounding flows along this river.

A 2.4 kilometer segment of Río Turabo has been recently channelized. This modification to the river's channel was made as a flood protection measure for the extensive urban development along its banks. **Figure 2** illustrates the segment of river that has been

channelized. The analysis performed in this study accounts for the channel geometry associated with this channelization.

2.3 Land Use

There is considerable urban development within the modeled watersheds, especially at lower elevations. Areas at higher elevations are mostly covered by secondary forest and pasture with considerable rural development. The average population density within the entire watershed area considered is near 650 people per square kilometer, based on 2000 census and determined using US Census Bureau Topologically Integrated Geographic Encoding and Referencing System (TIGER) georeferenced files within the GIS framework. Major towns within the watershed above Lago Loíza include: Aguas Buenas, Caguas, Gurabo, Juncos, Las Piedras, and San Lorenzo.

Areas adjacent to the segment of Río Turabo considered in this analysis are highly urbanized, especially overt it's left bank. This condition continues into Río Loíza for more than 3 kilometers downstream of its confluence with Río Turabo. Also, there are areas of concentrated development adjacent to the Lago Loíza Reservoir. Highways PR-1, PR-52, PR-30, PR-189, PR-183, PR-176, PR-175, and PR-172 are the major roads that run through the watershed. Bridge crossings over Río Turabo and Río Loíza by highways PR-52, PR-183, PR-189, and PR-30 will be impacted by the flooding events associated to the Quebrada Beatriz Reservoir dam breach.

2.4 Review of Previous Studies

FEMA has studied Río Turabo to a point 7 kilometers upstream of its confluence with Río Loíza. In addition, FEMA has studied a 6 kilometer portion of Río Loíza upstream of its confluence with Río Gurabo. FEMA mapping shows a Zone-A fringe extending from The Loiza Reservoir to the proposed site for the Quebrada Beatriz Reservoir. **Figure 3** shows the digitized limits of the inundations presented in FEMA's Flood Insurance Rate Maps in panels 7200000114b, 7200000177d, and 200000179b.

The USGS study "Floods at Caguas, Gurabo, Juncos, and San Lorenzo, Puerto Rico" by Fred K. Fields (1972) documents major floods in the area. The mayor floods documented in this report occurred in August 4, 1945 and September 6, 1960. The 1945

flood produced the highest stages from Río Cagüitas to the confluence of Río Loíza and Río Turabo. The peak discharge for this event was estimated to be near 2,406 cms at the Caguas gaging station on Río Loiza. The 1960 flood event produced the highest stages in Lago Loíza and for Río Gurabo and its tributaries. A peak discharge of approximately 2,025 cms was estimated for this event at the Caguas gage (50055000). Both events were assigned a recurrence interval of 70 and 30 years. **Figure 4** shows a digitized map with the area affected by the flooding event of 1,960 as reported by the USGS.

3 STUDY APROACH AND METHODOLOGY

All analyses were conducted using the guidelines issued by the International Commission on Large Dams (ICOLD 1998). Reference material from the U.S. Bureau of Reclamation, the Federal Energy Regulatory Commission, and the National Weather Service was used for the verification of input parameter values and model results.

Hydrologic simulations to develop storm hydrographs for the 100 year and PMP events were performed using the HEC-HMS model developed by the Hydrologic Engineering Center of the US Army Corps of Engineers. The unit hydrograph methodology developed by the Natural Resources Conservation Service was selected for runoff computations.

Hydraulic Modeling was performed using the U.S. Army Corps of Engineers River Analysis System (HEC-RAS) developed by Hydrologic Engineering Center. This model is capable of simulating a dam breach event and routing it as one-dimensional unsteady flow through a network of open channels in a mixed flow regime. HEC-RAS also allows for the incorporation of Geographic Information Systems (GIS) through the use of GEO-RAS interface. The analysis performed in this study incorporates the use of several methodologies provided by the GIS framework that aid and greatly improve the processes of geometric representation, parameter estimation, and result interpretation.

The developed hydraulic model simulates a piping breach using an initial condition of full reservoir level (FRL) at the reservoir for the three different scenarios presented below:

1- Sunny day dam failure,

2- Dam failure coincident with the 100 yr flood and

3- Dam failure coincident with the PMP flood.

4 HYDROLOGIC ANALYSIS

4.1 Watershed Delimitation

The Río Loíza basin was subdivided into 10 watersheds to provide different flow generation points along the river for hydraulic computations. **Figure 5** presents a scaled map of the watersheds modeled. Watershed delineation and parameter estimation were performed within the framework of a GIS model prepared for this study. This GIS was built from a Digital Elevation Model (DEM) obtained from the National Elevation Dataset maintained by the United States Geological Survey. **Figure 5** presents the DEM used.

4.2 Rainfall Depths

Rainfall depths used in the simulation are summarized in **Error! Reference source not found.**, as obtained from Weather Bureau Technical Paper #42 (U.S. Dept. of Commerce, 1961) for the PMP event and from the US Department of Commerce (2006) in NOAA Atlas 14 for the 100-yr and 2-yr events. The PMP rainfall is reported in TP-42 for only three durations, and was not updated in the NOAA Atlas 14 publication.

Table 2: 24-hour Rainfall Depths.

Rainfall Event	Duration (hrs)	Depth (mm)	Depth (in)
PMP a/	1	267	10.5
PMP a/	6	762	30.0
PMP a/	24	1,080	42.5
100-year b/	24	400-510	15.7-50.1

a/ TP-42

b/ NOAA Atlas 14

Data on maximum rainfalls recorded in Puerto Rico were also obtained as part of the procedure for development of the PMP storm. These data are presented in Table 3.

Table 3: Historical Maximum Point Rainfalls in Puerto Rico Through 2005.

Station	Date	Duration (hours)	Precipitation (inches)	(mm)
Yabucoa	10/11/1973	0.25	2	51
Cayey	1/5/1992	0.5	3.3	84
El Yunque	1/5/1992	1	5.9	150
Cayey	1/5/1992	2	8.4	213
Cayey	1/5/1992	3	11.2	284
Cayey	1/5/1992	6	18.4	467
Cerro Maravilla	10/7/1985	24	24.6	625

Source: U.S. Weather Bureau, San Juan.

4.3 Rainfall Distribution

The following methodology was used to determine the 24-hour hyetograph (rainfall depth over time).

100-yr event. Local timewise distributions were constructed from NOAA Atlas-14 depth-duration values for all watersheds.

PMP event. Table 3-5 of TP-42 (U.S. Dept of Commerce, 1961) provides guidelines for the ordering of hourly rainfall increments to develop a hyetograph for a hurricane PMP rainfall event. To use this table it is necessary to determine the anticipated rainfall intensities for different durations, from which the hourly increments can be computed for subsequent ordering by Table 3-5 (TP-42).

Because PMP rainfall depths are given for only three durations (1, 6 and 24 hours) it is necessary to determine the depths for all other durations. The PMP rainfalls (**Error! Reference source not found.**) and maximum observed rainfalls in Puerto Rico (Table 3) are plotted in Figure 6. From this plot it is apparent that a two-stage curve should be used with a break point at 6 hours. An Intensity-Duration (I-D) equation was fitted to the PMP rainfall having the following form:

for duration $\leq$ 6 hrs

$$Depth(mm) = 255D^{0.61}$$

for duration > 6 hrs

$$Depth(mm) = 255D^{0.2925}$$

where depth is total rainfall depth in mm, and D is rainfall duration in hours. There correspond to the two segments of the line plotted in Figure 6.

This equation was used to determine the depth of rainfall for durations of 15-, 30- and 45-minutes and for 1 through 24 hours at hourly increments. These hourly rainfall increments were ordered per TP-42 (Table 3-5) recommendations for a 45" PMP event. The peak hour was ordered in 15-minute intervals by intensity with the following sequence: 3, 1, 2, 4. These computations are summarized in Table 4.

Table 4: Timewise Rainfall Distribution for PMP Modeling

Time in hours	Order Sequence byTP-42, Table 3-5, 45" PMP	By Depth-D Equation		Cumulative Precip. a/
		Precip. Increment a/	Intensity per hour	
Col (1)	Col (2)	Col (3)	Col (4)	Col (5)
1.00	23	0.013	0.013	0.013
2.00	18	0.015	0.015	0.028
3.00	14	0.018	0.018	0.046
4.00	10	0.023	0.023	0.070
5.00	7	0.031	0.031	0.101
6.00	5	0.076	0.076	0.176
7.00	2	0.117	0.117	0.294
7.25	0.75	0.041	0.164	0.335
7.50	0.25	0.096	0.383	0.431
7.75	0.50	0.050	0.202	0.481
8.00	1.00	0.036	0.144	0.517
9.00	3	0.096	0.096	0.613
10.00	4	0.084	0.084	0.696
11.00	6	0.070	0.070	0.766
12.00	8	0.028	0.028	0.794
13.00	9	0.025	0.025	0.820
14.00	11	0.022	0.022	0.841
15.00	12	0.021	0.021	0.862
16.00	13	0.019	0.019	0.881
17.00	15	0.017	0.017	0.899
18.00	16	0.017	0.017	0.915
19.00	17	0.016	0.016	0.931
20.00	19	0.015	0.015	0.946
21.00	20	0.014	0.014	0.960
22.00	21	0.014	0.014	0.974
23.00	22	0.013	0.013	0.987
24.00	24	0.012	0.012	1.000

a/ Decimal of total 24 hour precipitation

The columns in Table 4 are described below:

Col (1) Time in hours

Col (2) Sequence given in Table 3-5 of TP-42 for 45" PMP rainfall. In this case, the rainfall from the 23rd most intense hour is used at hour one of the day, the 18th for the second hour, etc.

Col (3) Precipitation increment corresponding to the time period in column 2, expressed as decimal part of the 24-hour rainfall. These are computed by the Intensity-duration equation. For example, the rainfall for the 4th most intense hour of rainfall is computed as the rain with 5 hours duration less the rain with 4 hours duration, and the order provided in column 2 indicates that it is to be placed chronologically during hour 10 in column 1.

Col (4) Intensity of rainfall (decimal/hour) corresponding the rainfall depths in column 3.

Col (5) Cumulative precipitation over the 24 hour period, expressed in decimal format.

A plot of the derived unit distribution can be found in Appendix A.

4.4 Curve Number

The Curve Number parameter was estimated by combining digitized Natural Resources Conservation Service (NRCS) soils maps from the Soil Survey Geographic (SSURGO) database with land use data from the USGS which was updated using recent aerial photography. The computed curve numbers for each watershed are listed in **Table 5** for AMC–II condition.

Table 5: NRCS Curve Numbers by Watershed.

Watershed	Area (km²)	Curve Number
Queb. Beatriz	11.41	70
Queb. Beatriz 2	3.16	81
Río Turabo 1	29.13	79
Queb. Las Quebradillas	19.73	79
Río Loíza	146.20	79
Río Turabo 2	20.06	81
Río Bairoa	18.47	81
Río Caguitas	57.40	82
Río Gurabo	179.44	75
Río Cañas	52.19	80

4.5 Time of Concentration

Time of concentration was estimated by computing travel times in the three expected flow regimes along the longest flow path for each watershed. Within this framework, flow was conceptualized to develop as sheet flow, shallow concentrated flow and, ultimately, open channel flow. Computed values were adjusted for model calibration. The times of concentration used for each watershed are listed in **Table 6**.

Table 6: Time of Concentration by Watershed.

Watershed	Time of Concentration (min)
Queb. Beatriz	32.5
Queb. Beatriz 2	13.3
Río Turabo 1	66.7
Queb. Las Quebradillas	56.7
Río Loíza	206.5
Río Turabo 2	41.7
Río Bairoa	70.0
Río Caguitas	160.0
Río Gurabo	282.0
Río Cañas	142.5

4.6 Hydrologic Analysis Results Verification

Hydrologic modeling output files are contained in **Appendix A**. Hydrographs for each event are also included in this appendix. Peak discharges for the 100-yr event are presented in **Table 7.** Complete input and output files, including tabular output data, are included in the attached CD-ROM.

Table 7: Peak Discharge for the 100-yr Event.

Watershed	Discharge (m^3/s)
Queb. Beatriz	388
Queb. Beatriz 2	164
Río Turabo 1	755
Queb. Las Quebradillas	549
Río Loíza	2,449
Río Turabo 2	691
Río Bairoa	458
Río Caguitas	889
Río Gurabo	2,348
Río Cañas	861

4.7 Hydrologic Analysis Verification

The 100-yr discharge was verified against FEMA values were available. Log Pearson Discharge was used to verify the discharge at USGS Gage 5005500 at Río Loiza downstream confluence with Río Turabo. Results of this verification are presented in Table 8 for locations were FEMA coincides with this study.

Table 8: 100-yr Event Discharge Verification.

Location	HMS Discharge (m3/s)	FEMA Discharge (m3/s)	Log Pearson Discharge (m3/s)
Río Bairoa Mouth	458	440	--
Río Caguitas Mouth	889	780	--
Río Gurabo Mouth	2,348	2,270	--
Río Cañas	861	--	--
USGS Gage 5005500	3,490	3,280	3,342

4.8 Comparison to Historical Floods in Puerto Rico

Historical peak flood discharges registered at USGS gages in Puerto Rico have been plotted as a function of watershed area in **Figure 7**. Most floods plotted in the graph do not represent flood peaks having a 100-year return interval, but comparison of predicted flood peaks against historical peak floods on the island helps evaluate whether the obtained values are reasonable. The predicted values all fall within the upper range of observed peaks, which is reasonable.

5 DAM BREAK ANALYSIS

5.1 Conceptualization of the Hydraulic System

The hydraulic system analyzed consists of an earthfill dam, initially full, which experiences a piping failure. Breach parameters were set to initiate the dam failure at the time of maximum flood surcharge in the reservoir. All flow from the breach is delivered into the Quebrada Beatriz downstream of the dam at hydraulic model cross-section 27,859. The resulting flood is routed along Qurbrada Beatriz, Río Turabo, and Río Loíza down to Lago Loíza.

5.2 Dam Breach Methodology

The dam break analysis was performed using the unsteady module of the HEC-RAS model. This component of HEC-RAS is based on the UNET model and uses a similar framework to solve the unsteady flow equations. It is based on an implicit finite-difference solution of the complete one-dimensional continuity and momentum equations for unsteady flow coupled with an assortment of internal boundary conditions for simulating unsteady flows controlled by a wide variety of hydraulic structures.

A piping failure occurs when the initial breach formation takes place at some point below of the dam crest due to erosion of an internal channel through the earthen dam by escaping water. The failure starts at a user-defined time and at a user-defined centerline elevation. Modeling of this type of breach is performed using the orifice equation. When the orifice enlarges sufficiently, the breach transitions to broad crested weir flow and subsequent flow is computed using the broad crested weir equations. Broad crested weir flow computations take into consideration the progression of the breach geometry coupled with water surface elevations upstream and downstream of the breached impoundment. Time for breach formation was determined to be near one hour.

5.3 Dam Breach Parameter Calculation

Breach shape is assigned or pre-determined depending on the model used. Usually, this shape is assumed to be triangular or trapezoidal. The HEC-RAS model uses a trapezoidal section to simulate the final stages of the dam breach. The following empirical formulas

are suggested by the Bureau of Reclamation (Reclamation, 1998) to determine the time of development and the final width of a breach initiated by a piping failure.

$$B_{avg} = 3h_w$$

$$t_f = 0.011(B_{avg})$$

where:

B_{avg} = average breach width (m)

h_w = water depth over bottom of breach (m)

t_f = failure time duration (hr)

Breach parameters used in this analysis are given in **Table 9**.

Table 9: Dam Breach Parameters.

Breach Parameter		**Value**
Bottom Width (m)		70
Left Side Slope		.9H:1V
Right Side Slope		.9H:1V
Bottom Elevation of Breach (m)		120
Top Elevation of Breach (m)		156
Breach Starting WSEL (m)		
	Sunny Day	152.31
	100 year	153.34
	PMP	154.87
Breach Duration (hr)		1.1
Orifice Coefficient		0.7
Weir Coefficient		2.9
Weir Crest Elevation (m)		152
Dam Crest Elevation (m, msl)		156
Centerline Elevation of Piping Failure (m)		120

5.4 Dam Break Peak Discharge

Peak discharges produced by the piping dam break were computed under three different scenarios: sunny day failure, failure coincident with the 100 year event, and failure coincident with the PMP event. Breach parameters were set to initiate dam failure at the time of maximum flood level in the reservoir. Results are presented in **Table 10** for comparison purposes.

Table 10: Piping Dam Break below Dam for Different Scenarios.

Event	Peak Discharge (m^3/s)	Peak Discharge (ft^3/s)
Sunny Day Failure	11,800	416,714
Failure with 100 yr event	12,700	448,497
Failure with PMP event	14,010	494,759

Time series flow hydrographs for each failure event were generated. Additional flow hydrographs generated in the hydrologic simulations were introduced at selected locations along Río Turabo and Rio Loíza to account for the runoff produced by the 100 year and PMP events from the remaining drainage area below the reservoir. The piping dam breach discharge hydrographs are shown in **Figure 8**. All additional flow data are presented as **Appendix A**.

5.5 Dam Break Peak Discharge Verification

Empirical relations for peak breach outflow estimation have been developed through simple regression analysis of compiled case study data from real dam failure events. Wahl (2004) presents a compilation and a thorough evaluation of several of the derived equations. Three of the equations presented by Wahl were used to estimate the maximum breach outflow for the three scenarios modeled in this study to verify the peak breach discharges generated by the simulations. The three relationships are based on studies performed by the U.S. Bureau of Reclamation (1988), MacDonald and Langridge-Monopolis (1984), and the Soil Conservation Service (1981).

The Bureau of Reclamation (1988) presents an empirical equation for peak discharge estimation using water depth above the breach invert through the following equation.

$$Q_p = 19.1(h_w)^{1.85}$$

Where:

Q_p = peak breach outflow (m^3/s)

h_w = water depth over bottom of breach (m)

The Soil Conservation Service (1981) proposes a similar relationship using the same parameters with only a small variation in the coefficient. The suggested equation is presented below.

$$Q_p = 16.6(h_w)^{1.85}$$

MacDonald and Langridge-Monopolis (1984) incorporate the volume of water stored above the breach invert (V_w) into the equation. The resulting relationship is similar to the previous two and has the form:

$$Q_p = 3.85(V_w h_w)^{0.411}$$

Table 11 summarizes the dam failure peak discharge estimations made using the previous equations. The peak discharges computed by the HEC-RAS model are also presented for comparison.

Table 11: Dam Failure Peak Discharge Verification.

Event	**Discharge (m^3/s)**			
	Reclamation	**SCS**	**MacDonald**	**HEC-RAS**
Sunny day	11,833	10,289	10,105	11,800
100 yr	12,546	10,900	10,236	12,700
PMP	13,632	11,848	10,427	14,010

The peak discharges computed by HEC-RAS for the events modeled are slightly higher than the maximum values computed by the three empirical equations. This indicates that the breach modeling in this study produces a conservatively large flood for these events.

5.6 Layout of Hydraulic System

The dam failure flood wave was routed through a total of 361 cross sections that extend from the Quebrada Beatriz reservoir site to cross section 113.42 below the Lago Loíza dam. Model cross sections were derived from a surface in the form of a Triangulated Irregular Network (TIN) created using contracted survey data, USGS bathymetric survey data, USGS topographic mapping, field measurements taken by GPS, and by geometry from FEMA modeling as available. An additional 1,710 cross sections were interpolated within HEC-RAS to enhance computational stability.

A total of 5 bridges were included in the model: PR-52, PR-183, PR-189, and PR-30. These bridges were modeled as hydraulic structures whenever the necessary data were available. In the case of insufficient data the bridges were modeled as regions of increased surface roughness. **Figure 9** shows a selected set of the 361 cross-sections used for the dam break model. These correspond to cross-sections spaced at 1 kilometer intervals throughout the studied reaches and the bridge locations within the model. Additional cross-section information can be seen in the program output data contained in **Appendix B** of this report

Table 12: Selected Cross Sections.

River Station	Description	River Reach
27973.93	U/S Beatriz Dam	Quebrada Beatriz
27859.23	D/S Beatriz Dam	Quebrada Beatriz
27011.51	Model Cross Section	Quebrada Beatriz
26744.88	U/S PR-52 Bridge	Quebrada Beatriz
26024.08	Model Cross Section	Quebrada Beatriz
24966.11	U/S Caguas Real Bridge	Río Turabo
24039.47	Model Cross Section	Río Turabo
22991.18	Model Cross Section	Río Turabo
22011.10	Model Cross Section	Río Turabo
21003.61	Model Cross Section	Río Turabo
20218.61	1.7 km U/S PR-183 Bridge	Río Turabo
19011.99	Model Cross Section	Río Turabo
18525.49	U/S PR-183 Bridge	Río Turabo
18045.93	Model Cross Section	Río Turabo
17004.48	0.4 km U/S Turabo-Loiza River Confluence	Río Turabo
16060.68	Model Cross Section	Río Loíza
15028.34	0.2 km U/S PR-189 Bridge	Río Loíza
14813.56	U/S PR-189 Bridge	Río Loíza
13946.75	Model Cross Section	Río Loíza
13700.38	U/S PR-30 Bridge	Río Loíza
13069.44	Model Cross Section	Río Loíza
12062.40	Model Cross Section	Río Loíza
11017.97	Model Cross Section	Río Loíza
10053.61	0.1 km D/S Loiza-Gurabo River Confluence	Río Loíza
9105.84	Model Cross Section	Río Loíza
8118.26	Model Cross Section	Río Loíza
7002.47	Lago Loiza (7 km U/S Dam)	Río Loíza
6026.40	Lago Loiza (6 km U/S Dam)	Río Loíza
5099.70	Lago Loiza (5 km U/S Dam)	Río Loíza
4085.10	Lago Loiza (4 km U/S Dam)	Río Loíza
3085.76	Lago Loiza (31 km U/S Dam)	Río Loíza
2072.56	Lago Loiza (2 km U/S Dam)	Río Loíza
1081.62	Lago Loiza (1 km U/S Dam)	Río Loíza
375.01	U/S Carraizo Dam	Río Loíza
325.45	D/S Carraizo Dam	Río Loíza

5.7 Hydraulic Roughness

Cross-sections hydraulic roughness was determined using the FEMA model of Rio Turabo, from field inspection, and from Chow (1959). Manning's roughness coefficients were used to calibrate the 100 year flood profile to stages provided by FEMA. Roughness values for Río Turabo vary from 0.07 to 0.09 within the river's channel and from 0.11 to 0.13 in the channel overbanks. Similar values ranging from 0.07 to 0.09 in the channel and 0.1 to 0.13 in the channel overbanks were used for the rest of the modeled cross sections. Roughness values as high as 0.09 and 0.13 were necessary to achieve model stability. Use of these higher n-values will result in higher water surface elevations, which is a conservative assumption.

6 DAM BREAK ANALYSIS RESULTS

Peak water surface elevations, discharges and channel velocities along Quebrada Beatriz, Río Turabo, and Río Loíza for the three events modeled are summarized in **Table 13.**

Because the small tributary area to the Beatriz reservoir, peak discharge values for Sunny Day and 100-year event downstream the Beatriz Dam are similar with a slightly difference that increase with river station as noted in **Table 13**. This indicates that the peak discharge at the dam is primarily influenced by the reservoir volume and the increase in difference with river station is the result of the increase in tributary area.

Table 13: Dam Break Model Analysis Results for Selected Cross Sections.

River Station	Description	Sunny Day				100-yr				PMP			
		W.S. Elev. (m)	Peak Flow (m3/s)	Flood Wave Arrival Time (min)	Time to Peak (min)	W.S. Elev. (m)	Peak Flow (m3/s)	Flood Wave Arrival Time (min)	Time to Peak (min)	W.S. Elev. (m)	Peak Flow (m3/s)	Flood Wave Arrival Time (min)	Time to Peak (min)
27859.23	D/S Beatriz Dam	129.5	11,797	1	37	129.8	12,706	1	38	130.7	14,010	1	39
27011.51		127.2	9,714	5	39	127.7	10,166	7	41	128.7	11,300	5	40
26744.88	U/S PR-52 Bridge	123.2	9,362	8	40	123.5	9,873	8	41	124.1	11,007	6	42
26024.08		112.6	8,692	12	42	113.0	9,086	12	43	114.3	10,081	7	43
24966.11	U/S Caguas Real Bridge	107.0	7,445	17	49	107.2	8,059	19	47	108.0	10,133	11	48
24039.47		92.1	7,081	21	52	92.4	7,808	23	52	93.2	10,356	13	51
22991.18		89.0	4,702	32	59	90.0	6,183	29	59	91.7	9,322	15	55
22011.10		83.3	4,276	38	67	84.7	5,621	36	64	87.5	8,439	18	61
21003.61		76.8	3,748	43	76	77.5	4,929	40	73	79.7	7,911	21	69
20218.61		71.0	3,473	47	82	72.0	4,683	43	78	73.8	7,712	24	72
19011.99		64.6	3,213	55	92	65.4	4,449	48	86	67.2	7,457	31	78
18525.49	U/S PR-183 Bridge	63.2	3,105	57	97	64.0	4,348	51	89	66.0	7,299	36	81
18045.93		61.0	2,965	59	102	62.3	4,143	54	93	65.1	7,083	40	84
17004.48		57.8	2,649	68	114	61.3	3,552	59	102	64.4	6,670	43	89
16060.68		56.6	2,150	74	129	60.4	5,195	64	115	65.1	7,083	48	97
15028.34		53.4	2,090	79	138	57.1	5,062	68	126	60.7	11,447	58	106

Table 13: Dam Break Model Analysis Results for Selected Cross Sections. (continued)

River Station	Description	Sunny Day				100-yr				PMP			
		W.S. Elev. (m)	Peak Flow (m3/s)	Flood Wave Arrival Time (min)	Time to Peak (min)	W.S. Elev. (m)	Peak Flow (m3/s)	Flood Wave Arrival Time (min)	Time to Peak (min)	W.S. Elev. (m)	Peak Flow (m3/s)	Flood Wave Arrival Time (min)	Time to Peak (min)
14813.56	U/S PR-189 Bridge	51.5	2,085	80	140	55.4	5,024	69	126	60.5	11,301	60	107
13946.75		49.4	1,988	85	148	55.4	5,024	70	134	60.3	9,761	63	111
13700.38	U/S PR-30 Bridge	49.2	1,944	86	154	54.2	4,773	74	134	60.3	9,265	64	113
13069.44		47.8	1,910	89	160	53.4	4,460	77	135	60.2	8,361	66	117
12062.40		46.0	1,768	95	172	53.3	4,362	80	152	60.1	9,012	67	122
11017.97		45.4	1,315	104	204	53.2	4,060	84	178	60.1	8,609	70	159
10053.61		44.4	1,249	109	215	52.1	6,110	89	178	59.1	14,284	73	167
9105.84		43.7	1,213	114	227	51.7	6,031	92	188	58.8	14,135	75	191
8118.26		42.7	1,192	119	239	50.6	5,978	96	199	57.9	14,029	79	197
7002.47		40.4	1,188	125	246	47.5	5,965	99	204	55.7	13,983	81	198
6026.40		36.8	1,172	131	253	45.2	6,221	101	206	53.5	15,113	84	199
5099.70		36.0	1,150	136	261	44.0	6,187	104	211	52.8	14,999	86	210
4085.10		35.4	1,128	140	270	43.0	6,153	107	217	51.9	14,922	88	218
3085.76		34.9	1,111	144	279	41.9	6,122	110	224	51.3	14,865	91	225
2072.56		34.3	1,098	147	289	40.9	6,090	113	233	50.7	14,820	94	233
1081.62		33.8	1,090	151	298	40.0	6,075	117	241	50.3	14,800	96	240
375.01	U/S Carraizo Dam	33.4	1,089	155	302	37.9	6,073	119	244	48.9	14,798	98	243

Comparative flood levels for the 100-year and PMP events with and without the dam breach are presented in the following table.

Table 14: Comparative Flood Levels at Selected Cross-Sections.

River Station	100-yr W.S. Elev. (m, msl)		Difference (m)	PMP W.S. Elev. (m, msl)		Difference (m)
	Without Dam Break	With Dam Break		Without Dam Break	With Dam Break	
27859.23	119.19	129.81	10.62	120.84	130.69	9.85
25008.49	97.08	107.13	10.05	100.05	107.62	7.57
20118.61	68.99	71.96	2.97	71.88	73.81	1.93
15028.34	55.55	57.11	1.56	60.23	60.69	0.46
10053.61	51.42	52.3	0.88	58.79	59.27	0.48
5099.703	43.17	43.93	0.76	52.25	52.71	0.46
375.009	37.34	37.91	0.57	48.52	48.89	0.37

The attenuation of the breach wave is presented by the difference in water levels. As presented in **Table 14** the influence vary from 10 to 0.6 meters. Cross section 375 presents the impact on Lago Carraizo water levels. Results indicate that Lago Loiza Dam is not overtopped for the 100-year event for both conditions. Otherwise during PMP event for both conditions the Dam is overtopped.

Flood limits, flood arrival times, and max water surface elevation times associated with the three dam failure events along the studied reaches are shown in **Figure 10-15**. The following arrival times are shown for each failure event:

1- Start of dambreak flood, and

2- Maximum dambreak flood peak

Because of the length of the studied reach, the 100-year and PMP events can cause flooding to be initiated in areas below the dam prior to arrival of the flood wave from a dambreak. This condition affects the temporal development of floods in the lower reaches of the studied area, especially in large events with multiple hydrographs entering from lateral basins. Travel times for the 100-year and PMP events were determined by superimposing model output hydrographs at selected cross sections for each event with and without the dam breach. The point were hydrograph ordinates including the breach departed from hydrograph ordinates without the breach was

considered as the initial point of influence from the breach flood wave. Water surface profiles are shown in **Figure 16**.

For practical purposes, only a portion of the output data for dam break simulation is contained in **Appendix B** of this report. The complete input an output datasets are included in the accompanying CD.

7 CERTIFICATION

I hereby certify that the document "Dam Break Flood Hazard Analysis for Quebrada Beatriz Offstream Reservoir" has been prepared in accordance with the best hydrologic and hydraulic practices as described in this document and that, based on the studies and field measurements provided by other parties, results are true and correct.

Certified today May, 2007

Gregory L. Morris, P.E., Ph.D.

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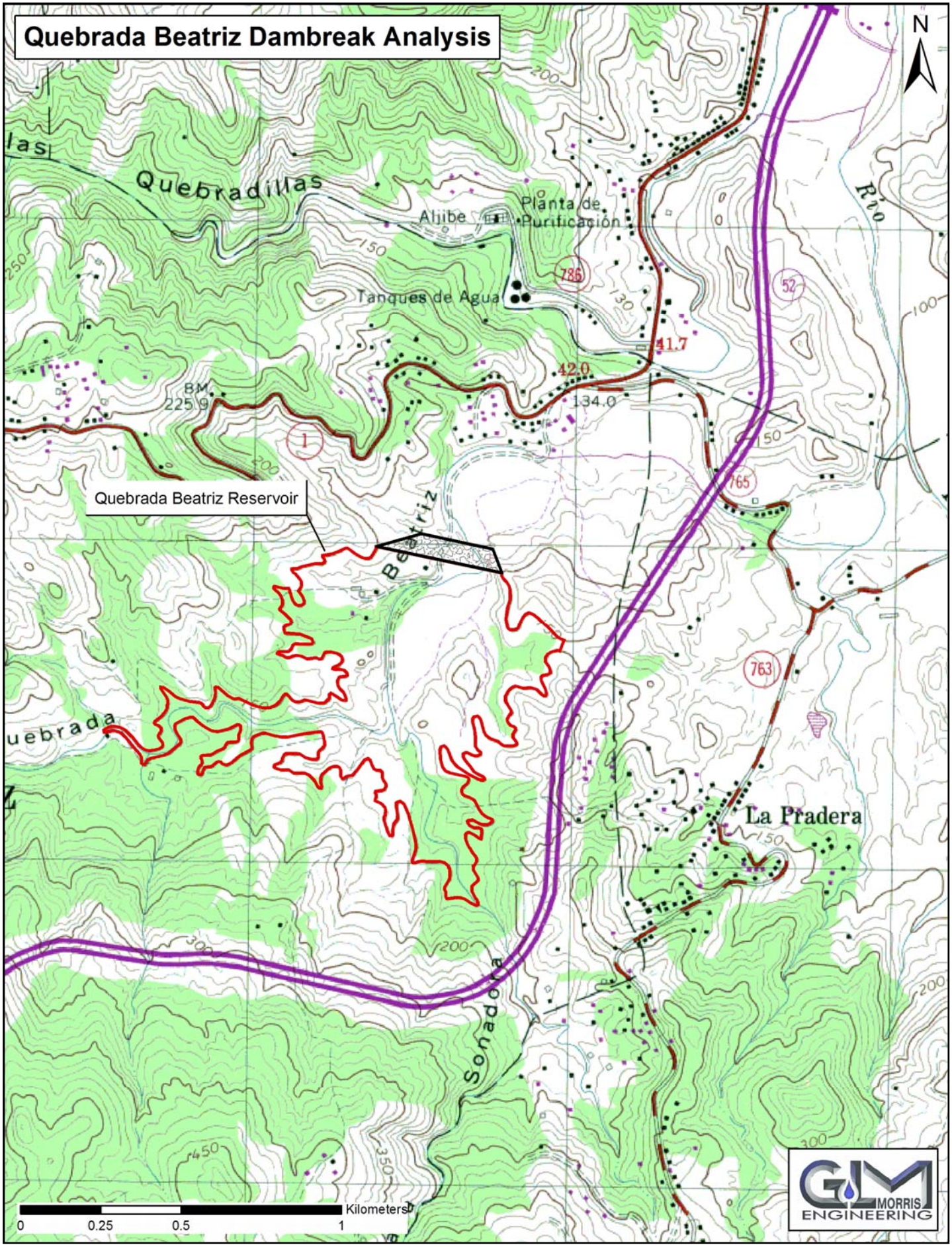

Figure 1. Reservoir configuration.

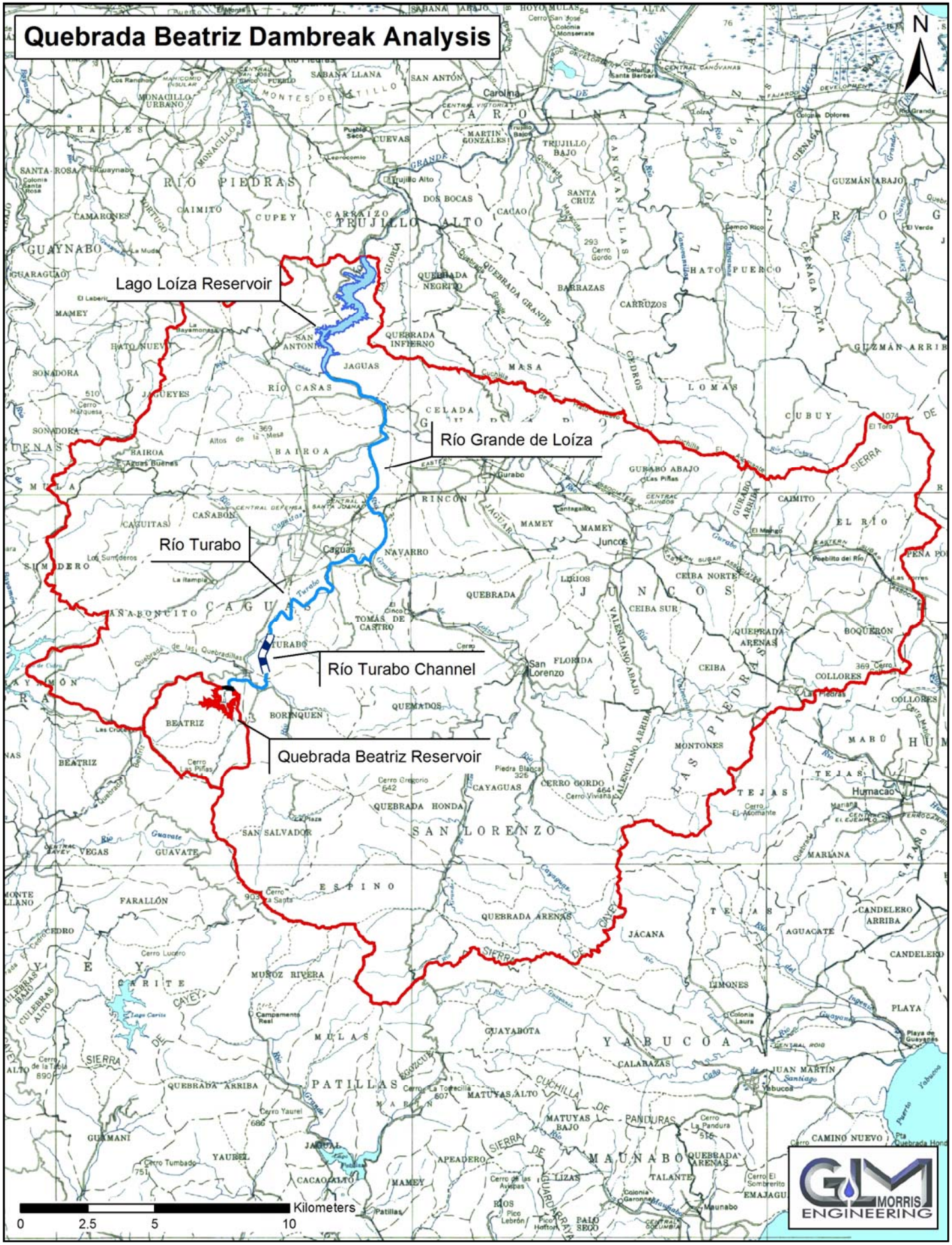

Figure 2. Río Loíza drainage area.

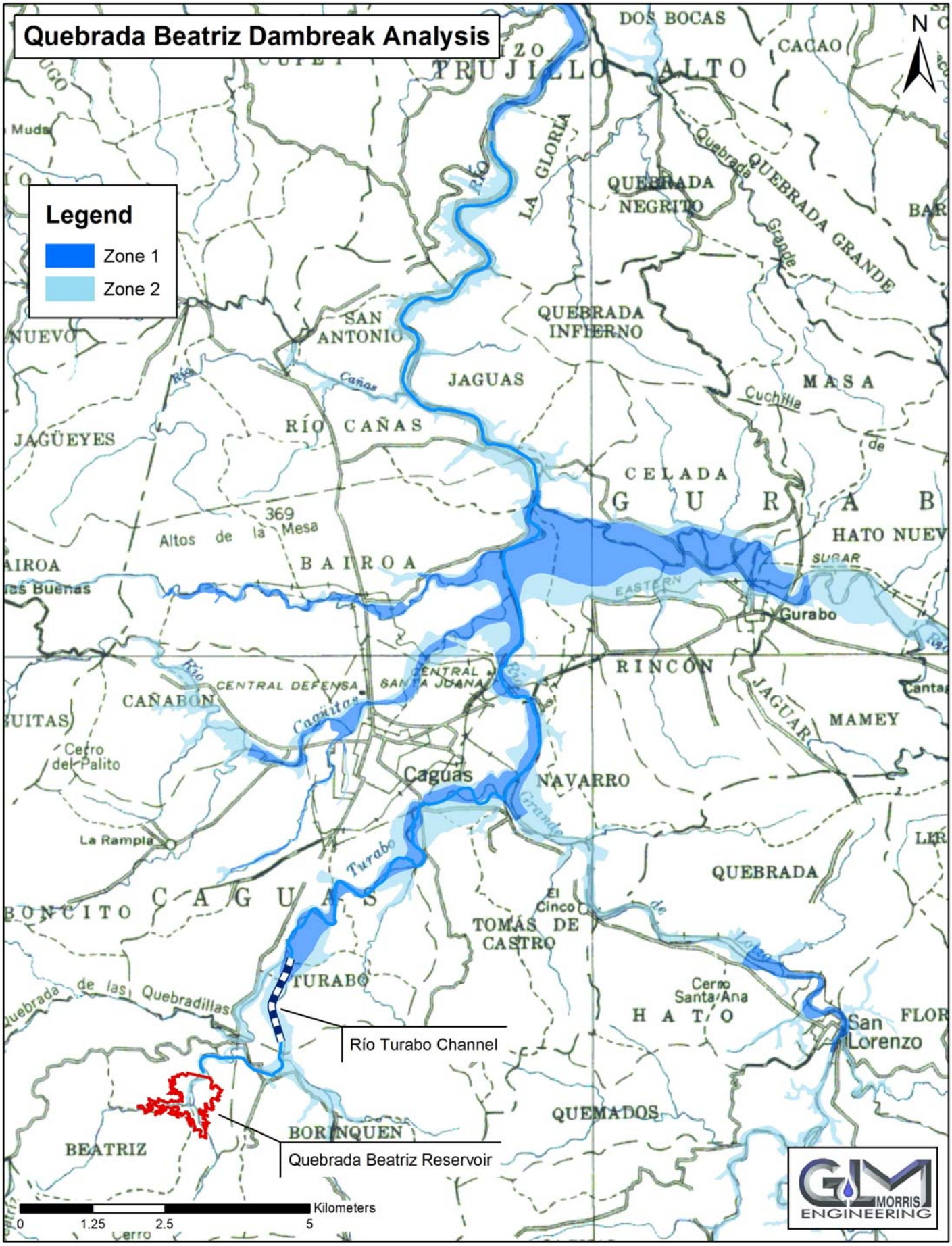

Figure 3. FEMA flood limits.

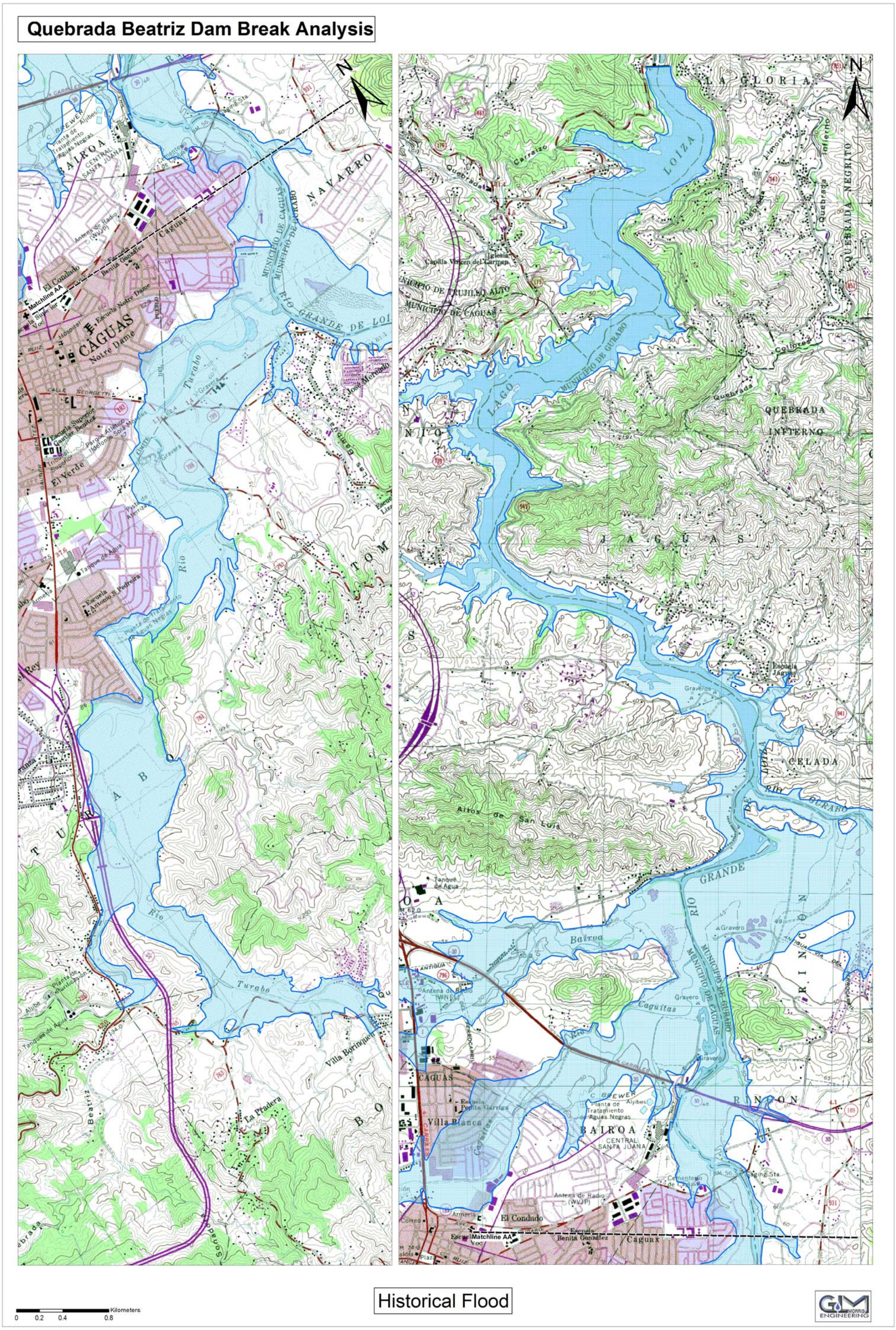

Figure 4: Historical flood 1960 (USGS)

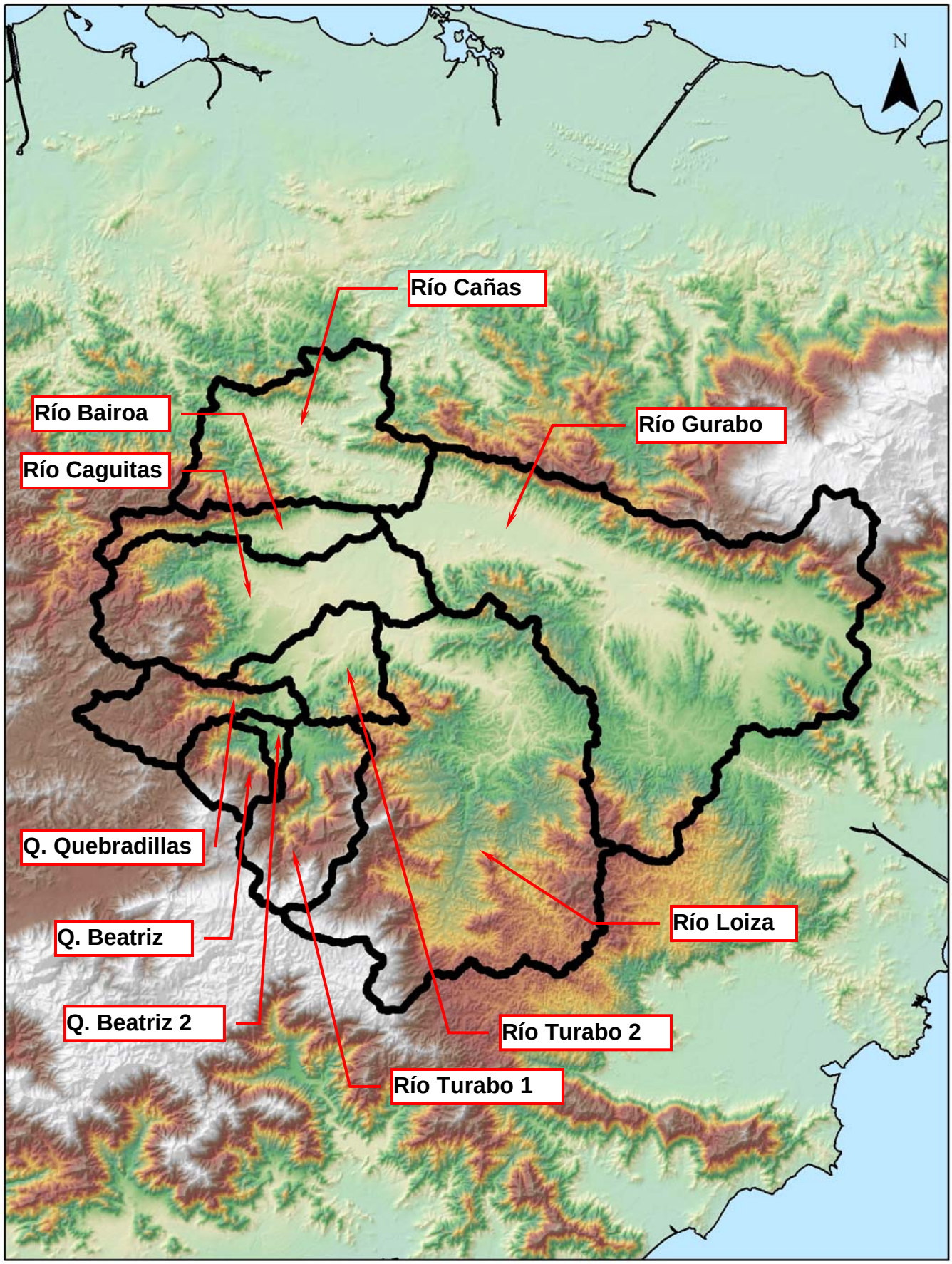

Figure 5. Watershed limits.

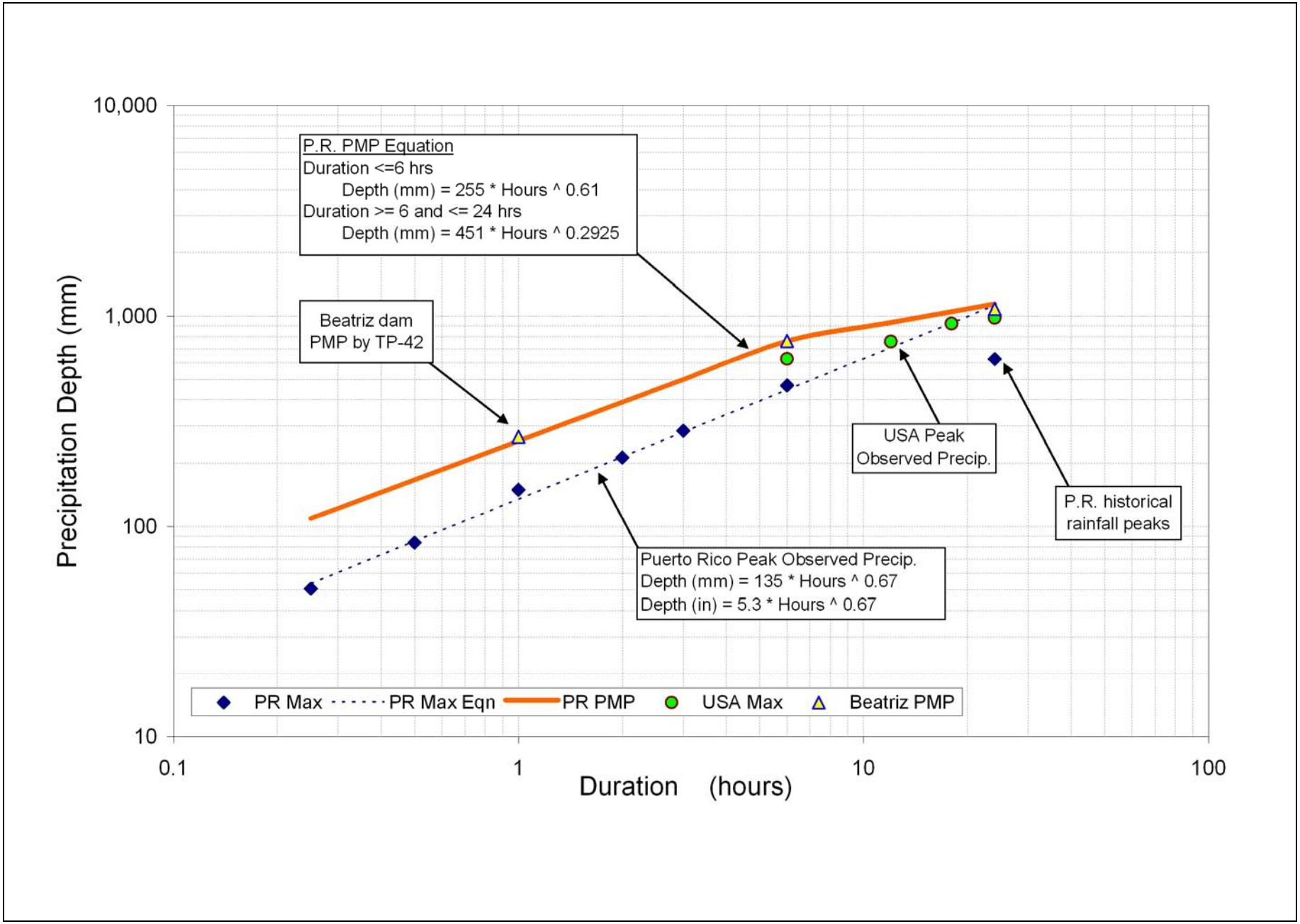

Figure 6: Depth-Duration data and computed relationships for Puerto Rico.

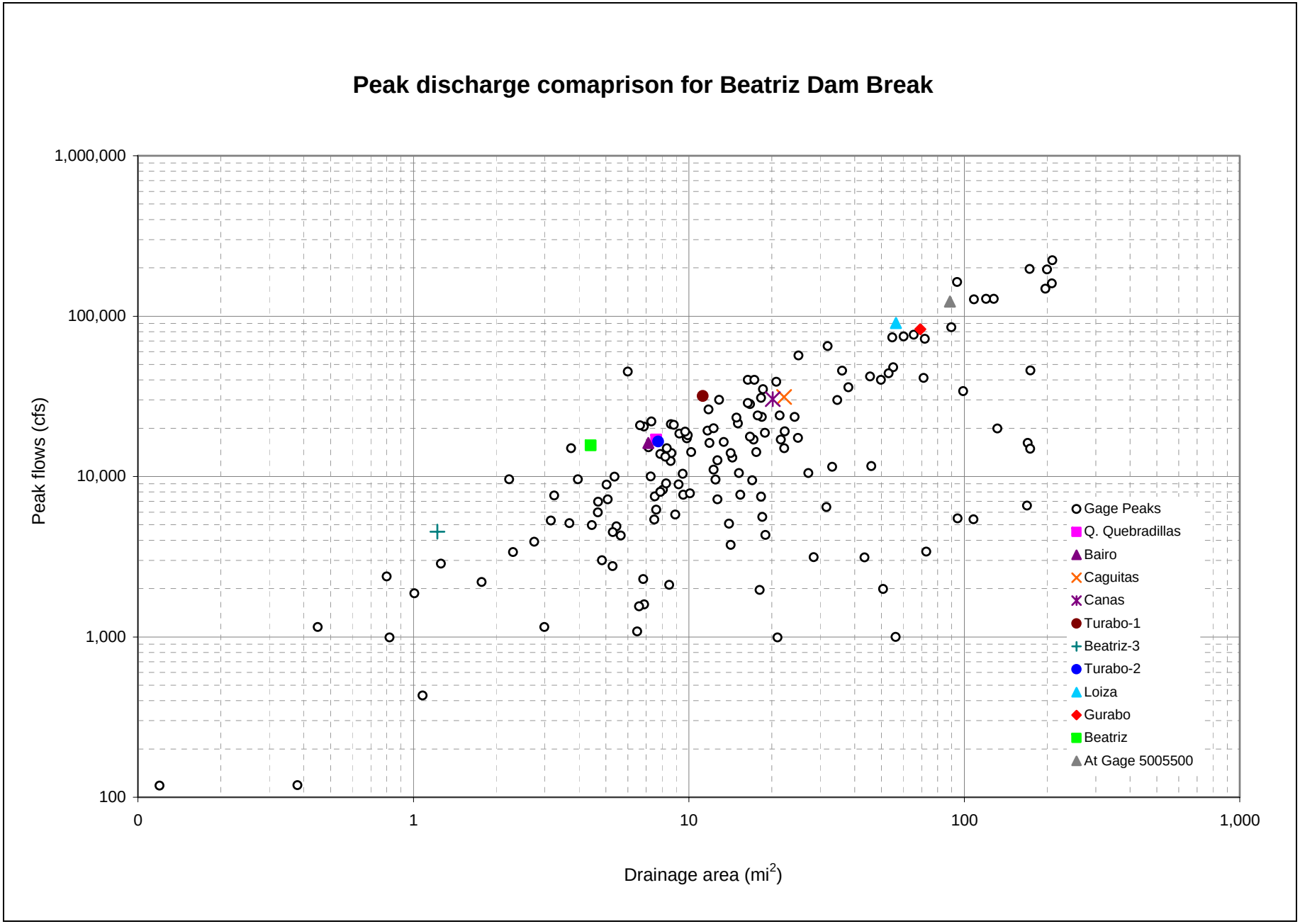

Figure 7: Historical Peak Runoffs at USGS Station vs. Drainage Area.

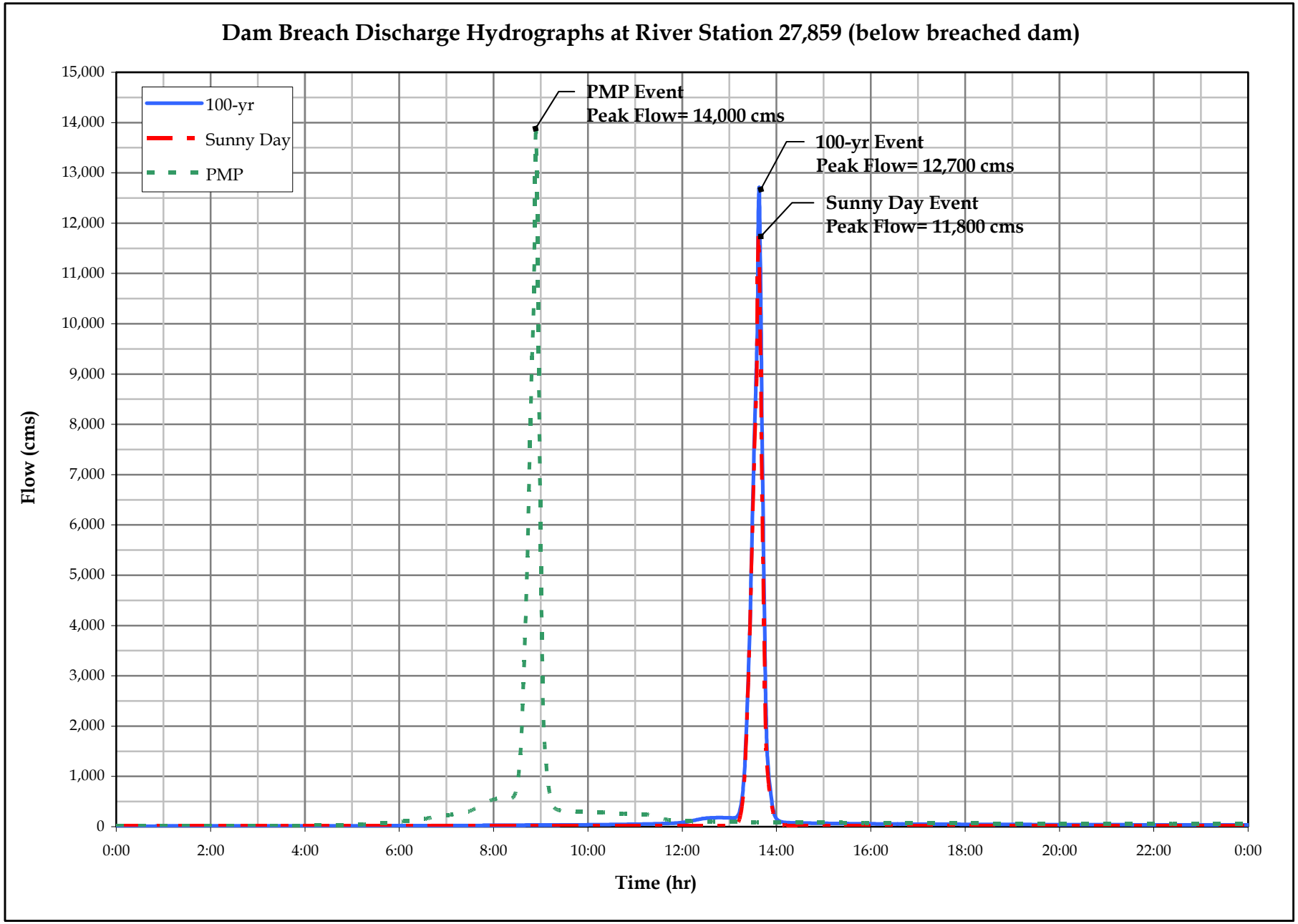

Figure 8: Piping dam breach discharge hydrographs, including both the dambreak flow plus the flood flow

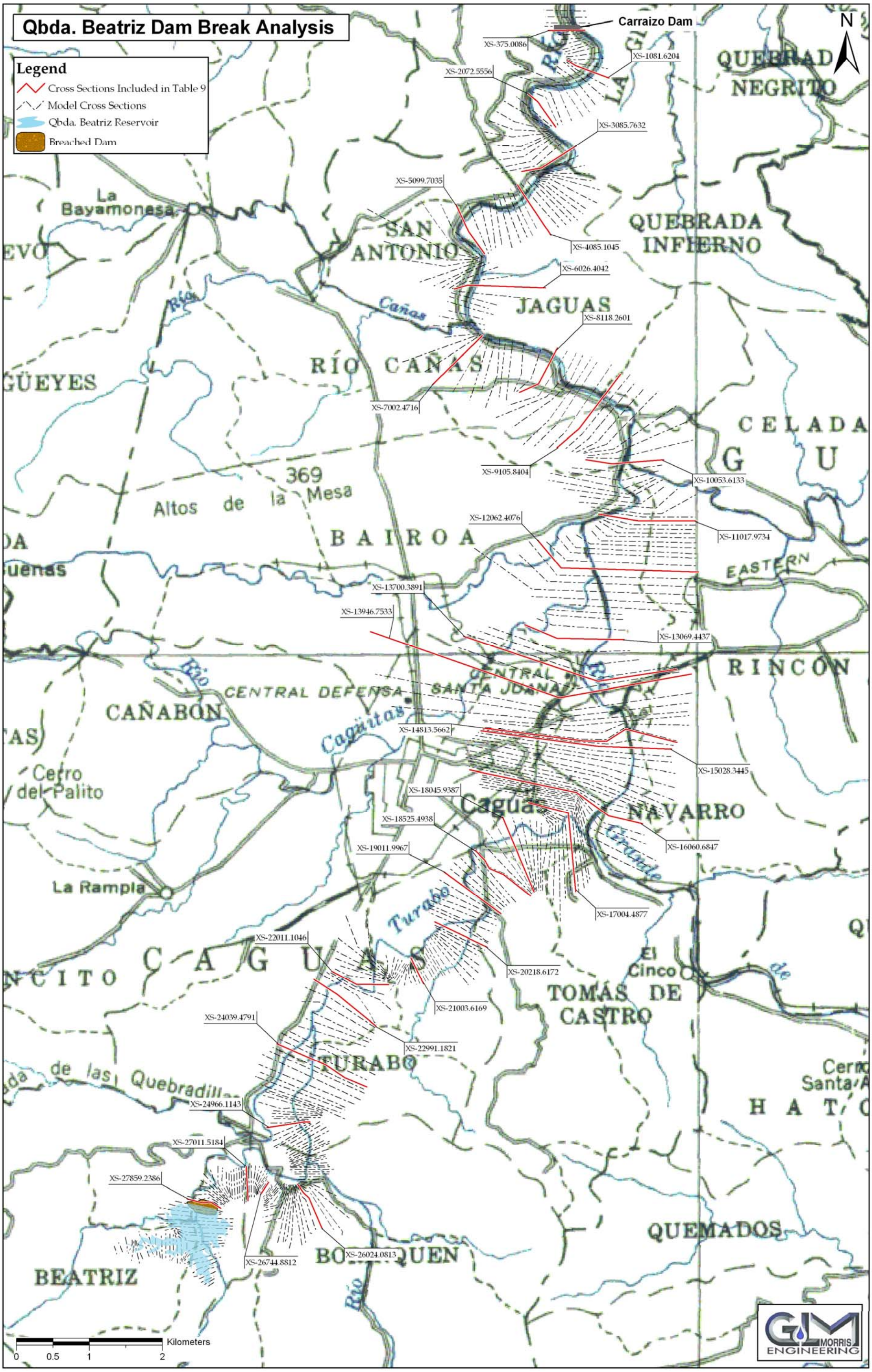

Figure 9: Cross section location

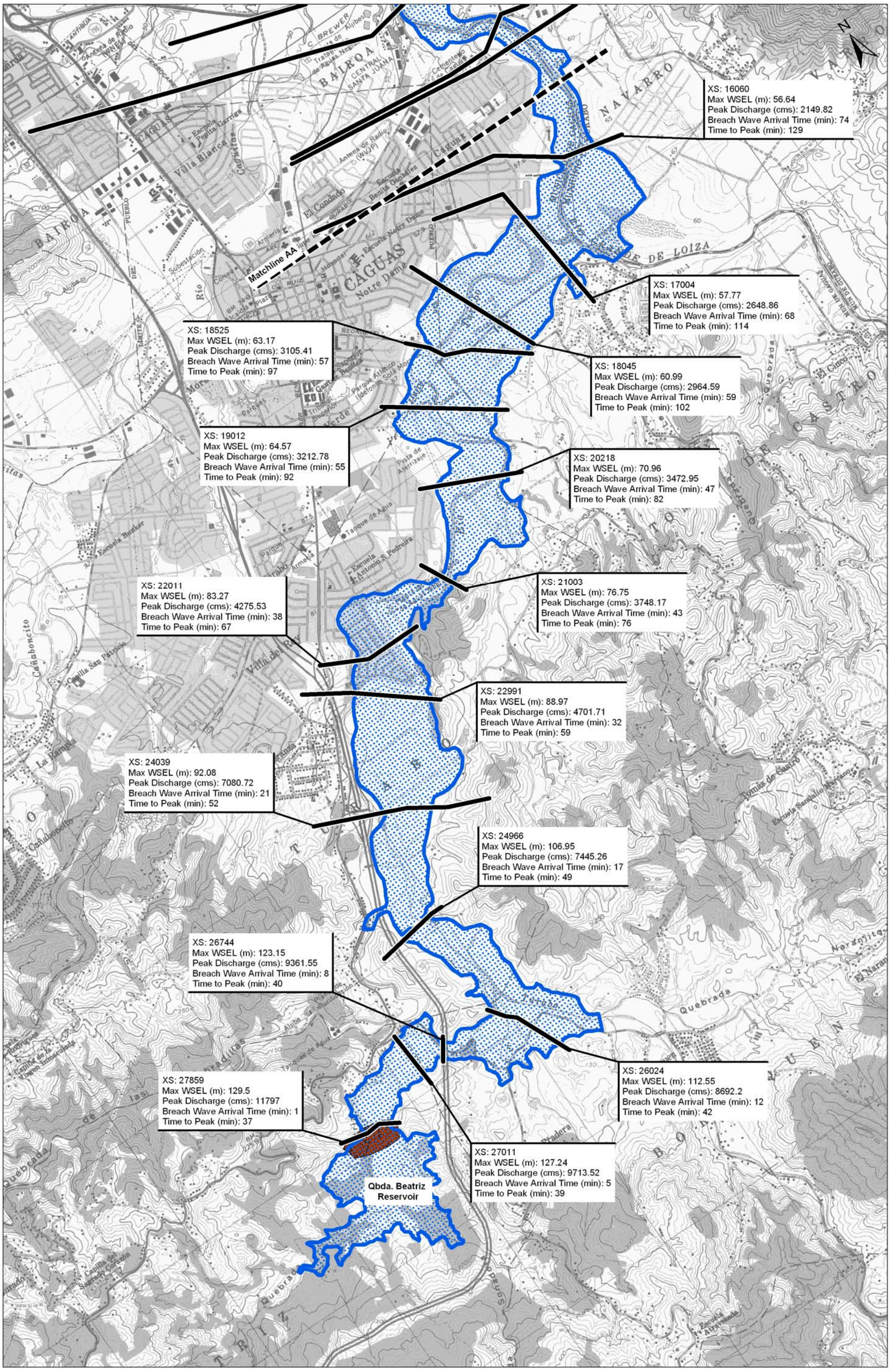

Figure 10: Flood limits: Sunny Day event

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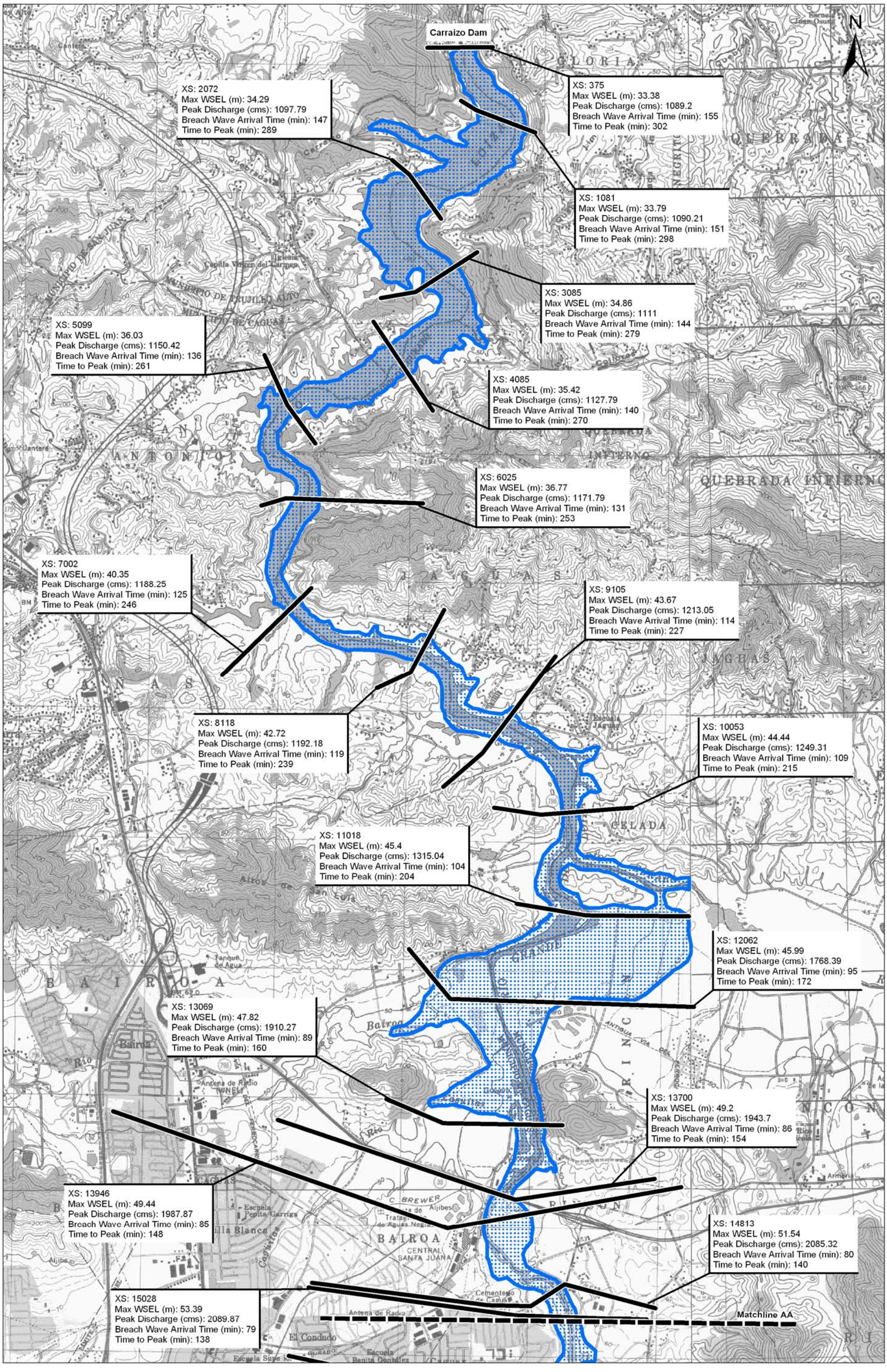

Legend

Flood Limits

Kilometers

0 0.25 0.5 1

Figure 11: Flood limits: Sunny Day event

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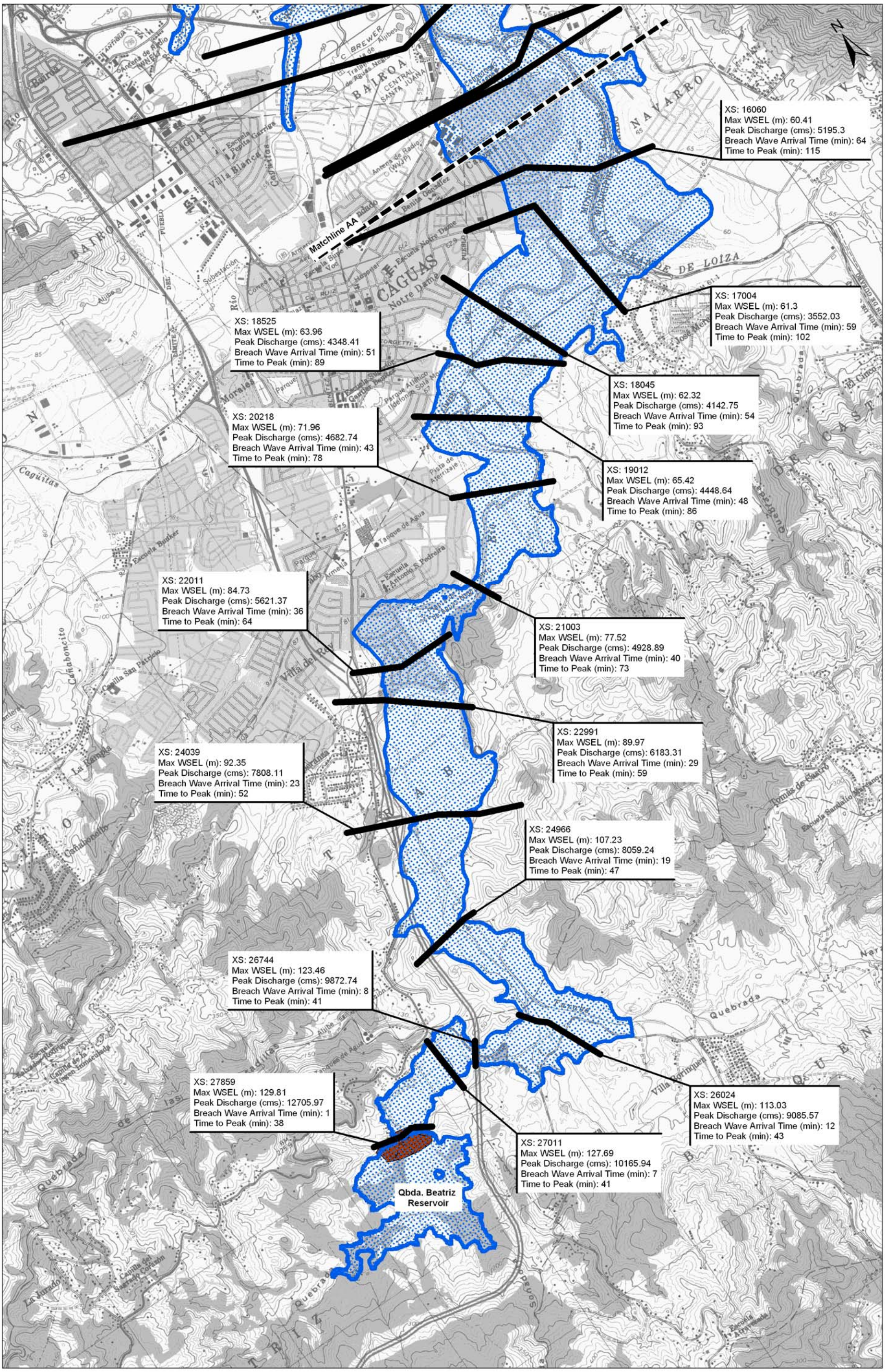
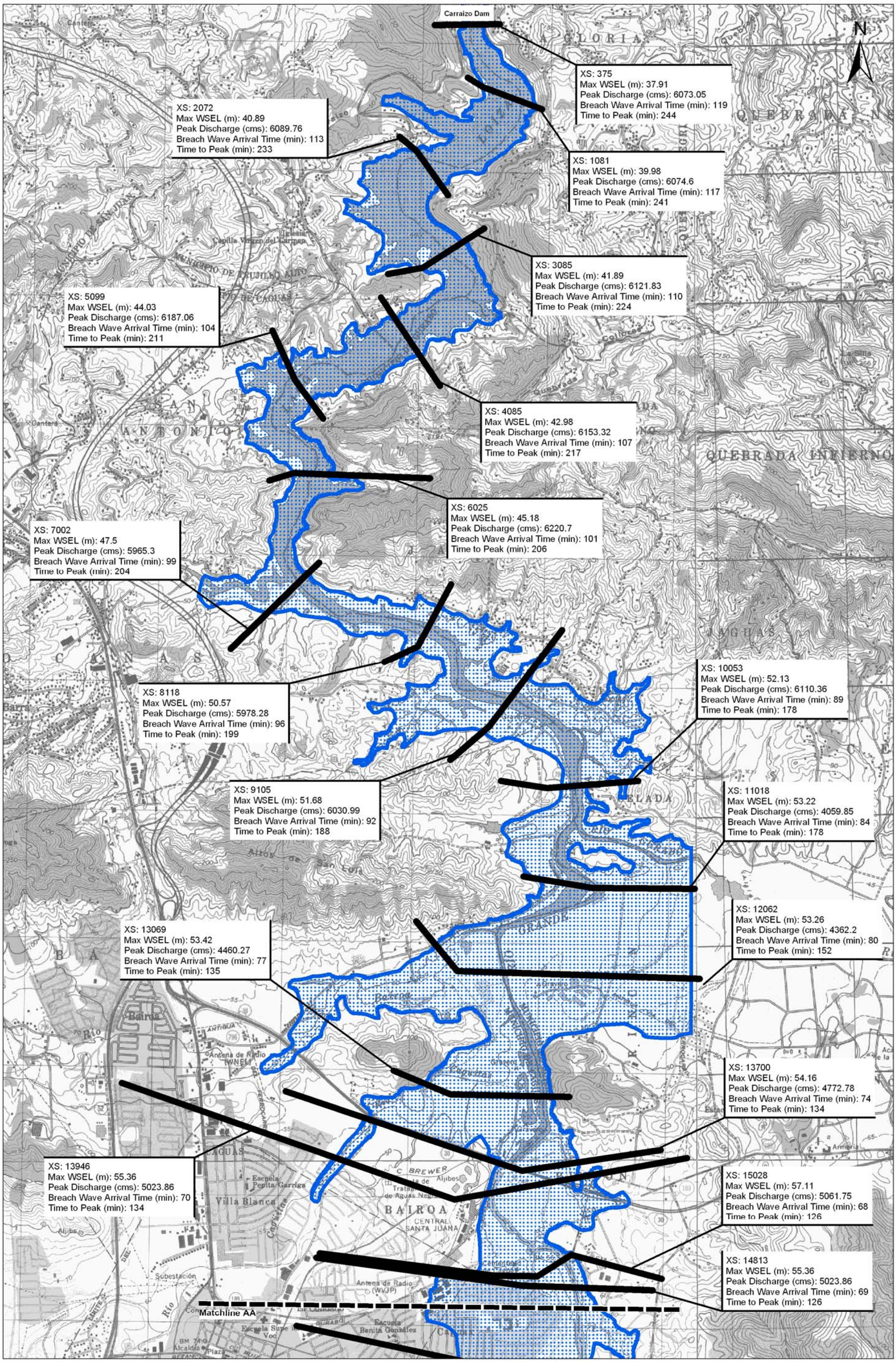


Figure 12: Flood limits: 100-yr event



Legend

Flood Limits

0 0.25 0.5 1 Kilometers

Figure 13: Flood limits: 100-yr event

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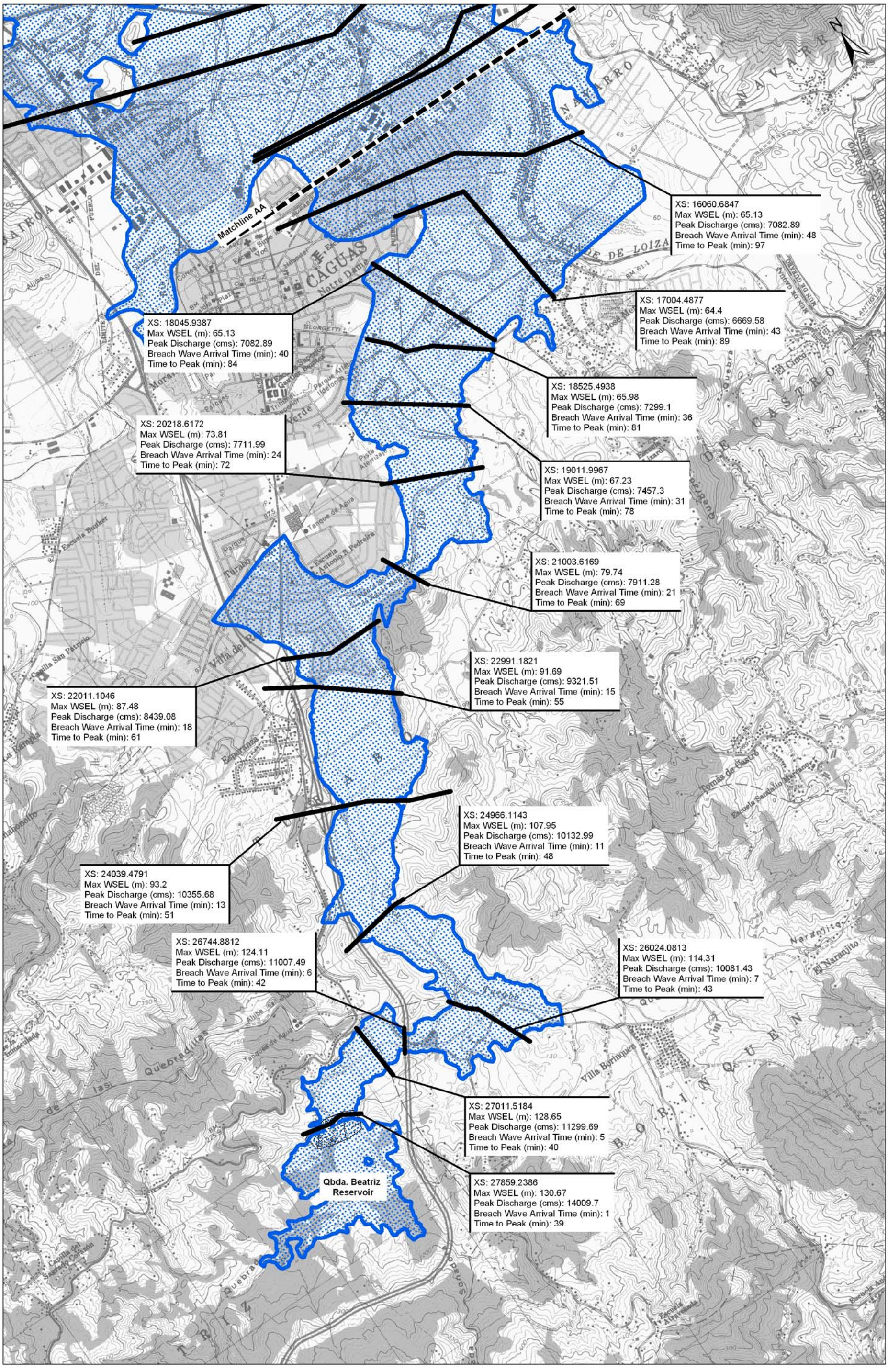
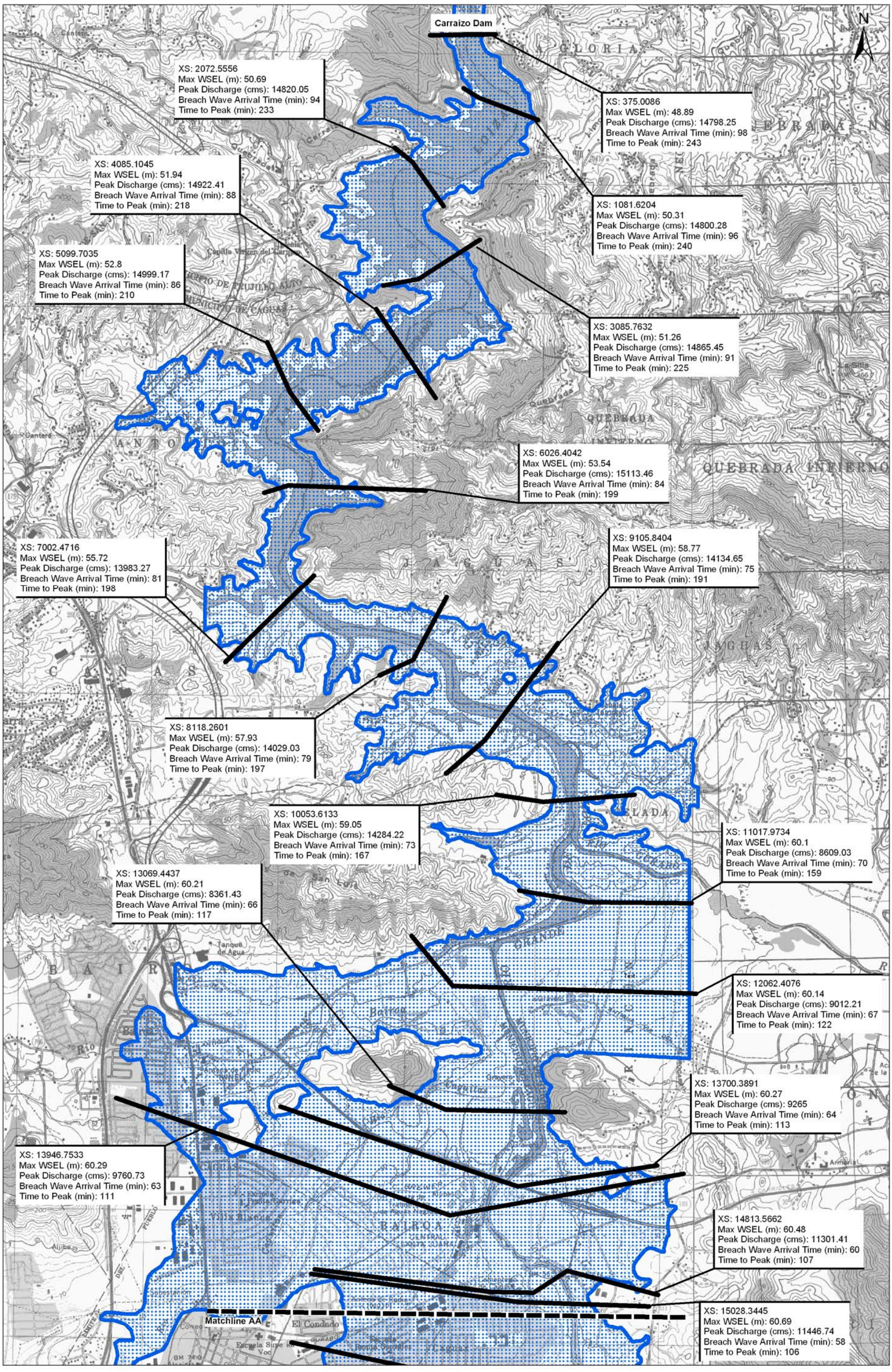


Figure 14: Flood limits: PMP event



Legend

Flood Limits

0 0.25 0.5 1 Kilometers

Figure 15: Flood limits: PMP event

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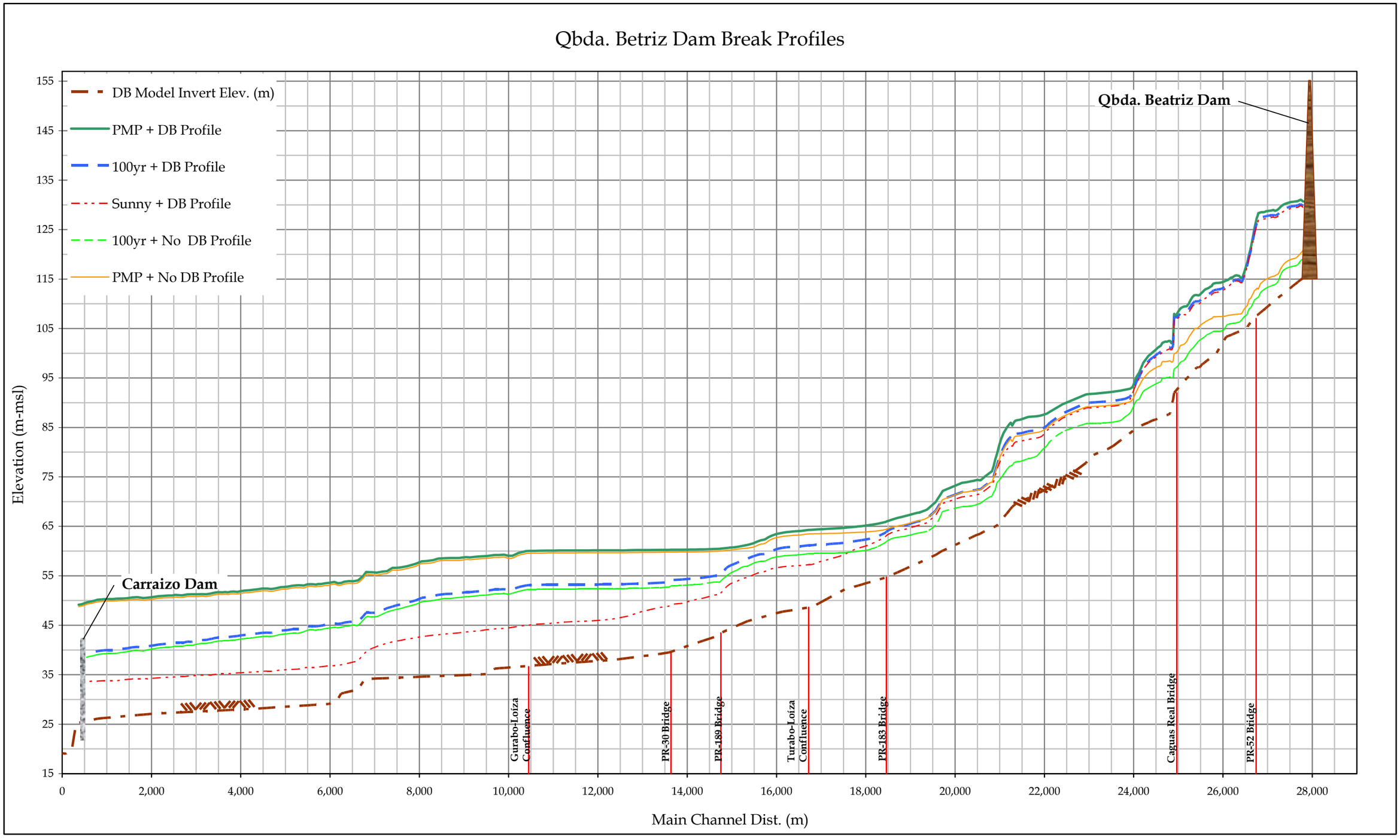

Figure 16: Water surface profiles