HYDROLOGIC-HYDRAULIC STUDY FINAL REPORT

DAM BREAK ANALYSIS FOR THE VALENCIANO WATER SUPPLY RESERVOIR JUNCOS, PUERTO RICO

FOR

CSA GROUP, INC.
MEZZANINE SUITE
MERCANTILE PLAZA
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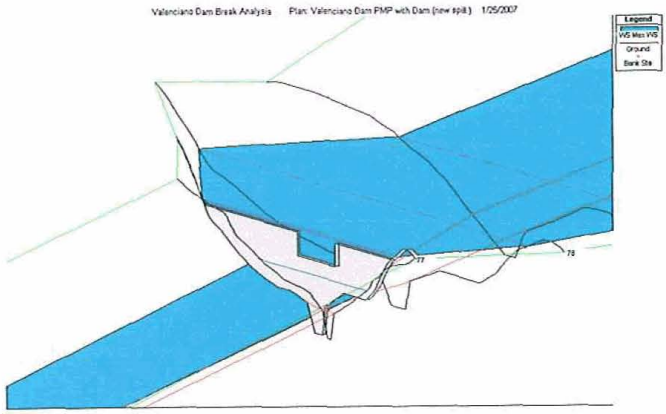

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I. INTRODUCTION

The firm of CSA Architects and Engineers, LLC, commissioned the study whose results are presented and discussed herein. The study was performed as part of the planning and design effort related to the development of a water supply reservoir in the Valenciano river, near Juncos, Puerto Rico.

Hydrologic and hydraulic analyses were conducted with the objective of assessing the inundation profiles downstream of the dam site resulting from the Probable Maximum Flood on the Valenciano river's drainage area. On the basis of current federal guidelines for dam safety, several scenarios were simulated in the study. These included dam-breach, no dam-breach, and sunny-day breach scenarios.

II. HYDROLOGIC ANALYSIS

2.1 Introduction

The objective of the hydrologic analysis was the derivation of the runoff hydrographs that constitute the baseline data input to the dam break analysis of the proposed Valenciano dam. A dam break analysis requires the application of the Probable Maximum Flood (PMF), obtained from either a statistical analysis of existing flow records or by performing a deterministic rainfall-runoff transformation with the Probable Maximum Precipitation (PMP). The latter approach was selected for the present study.

The PMP has been defined by the World Meteorological Organization (1986) as "*theoretically the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location at a certain time of the year*".

The runoff hydrograph derived from the PMP yielded the estimate of the PMF for the dam break analysis performed in this study

2.2 Methodology for PMP Estimation

Several procedures are available for estimating the PMP, which reflects the high degree of uncertainty associated with the estimation of this parameter. One of the most popular methods is

the moisture maximization approach, which estimates the amount of precipitable water in a vertical column of the atmosphere. However, this method requires an extensive climatological record to in order to yield adequate results. Such specific data may not be available at most locations.

Locally, the outdated Technical Paper No. 42 (TP-42) from the Weather Bureau (1961) provided estimates of the PMP based on a combination of results from the application of a hurricane model and a statistical method. The statistical method was based on the work of Hershfield (1961). In the TP-42, the applicability of Hershfield's method was limited by the relatively small rainfall record available at the time. Recently, NOAA has provided an online, updated rainfall atlas for Puerto Rico (Atlas 14) which provides rainfall frequency information at any location on the island. Rainfall frequencies of up to 1000 years are provided, but no recent PMP estimates are included in the report.

Because it is based on thousands of observations, Hershfield's model is one of the most popular methods, and is also one of the procedures suggested by the World Meteorological Organization (1986). Hershfield's method is the adopted methodology for the present study, mainly for three reasons: (a) the availability of an adequate rainfall record for the Valenciano basin, (b) the relative

simplicity of the formulation, and (c) the extensive real-world data base on which the method is based.

It needs mentioning that the estimation of the PMP is a controversial issue since, by definition, the concept implies that an upper bound rainfall depth exists at a site. This is an open question in hydrology to which much research effort has been dedicated. For practical purposes, it is adequate to estimate a PMP with a high return period, under the assumption of stationarity in the sense that no climatological variations will occur that will introduce trends in the statistics of the recorded rainfall process. Indeed, this assumption is implicit in any of the available PMP estimation methods.

According to Koutsoyiannis (1999), the return period of a PMP estimate based on Hershfield's method can reach up to 60,000 years, as suggested by an analysis of data from Athens, Greece, with a Generalized Extreme Value (GEV) distribution. Such a result implies that a PMP determined from Hershfield's method may lack an upper bound if the distribution has an infinite domain. Nevertheless, in view of all the attendant uncertainties associated with the estimation of events with high return periods, the method is quite adequate for practical purposes.

The general expression for the PMP based on Hershfield's method is given by

$$h_m = \bar{h}_n + k_m s_n \tag{1}$$

where:

h_m = is the maximum observed rainfall depth at the location of interest

$\bar{h}_n$ = mean of the series of *n* annual maxima

s_n = standard deviation of the series of mean annual maxima

k_m = frequency factor

The maximum predicted value of the frequency factor k_m is 15. The PMP is thus obtained as the rainfall depth at 15 standard deviations from the mean. In a subsequent study, Hershfield (1965) proposed a correction to the frequency factor based on rainfall duration.

Hershfield's method was applied to the Valenciano basin by computing the PMP at the rainfall stations within the basin and then selecting the most appropriate estimate. This follows the procedure outlined by Elíasson (1997), in which the highest station PMP estimate available at a basin is used for design purposes. This accounts for the fact that even though the rainfall at nearby stations may be correlated, it may nevertheless be possible for the annual maximum series to yield small correlation coefficients,

although this may not actually be the case in regions with heavy orographic influence.

A rough estimate of the return period of the PMP can be obtained by fitting a probability density function (pdf) to the recorded peak annual rainfall data. As argued by Papalexiou and Koutsoyiannis (2006), and Koutsoyiannis (1999), the Generalized Extreme Value (GEV) distribution provides a good fit to a given station's annual rainfall series.

The use of the GEV distribution is pretty well established in hydrology. The novelty in applying the GEV model to PMP studies resides in the observation that this supposedly bounded variable can be described by a pdf that can be unbounded at its upper end, that is, it can reach up to infinity. The theoretical aspects, and implications, of this issue are beyond the scope of this study. The use of the GEV model in the present study is limited to the practical consideration of providing a ballpark estimate of the return period of the PMP while being aware that, by definition, this variable could theoretically be only described, if it were at all possible, by a bounded distribution.

The GEV distribution is usually expressed in its cumulative form, that is, as a cumulative density function (cdf). The general expression of the cumulative GEV is given as (Maidment, 1993):

$$F(x) = \exp\left\{-\left[1 - \frac{\kappa\left(x - \xi\right)}{\alpha}\right]^{1/\kappa}\right\} \qquad for\ \kappa \neq 0 \qquad (2)$$

where:

F(x) = cumulative probability, that is, P[X ≤ x]

ξ = the location parameter

α = the scale parameter

κ = the shape parameter

Since it defines its boundedness, the shape parameter is the most important statistic of the distribution. A thorough discussion of the nature and application of the distribution is beyond the scope of the present study, and can be found elsewhere. Suffice it to say that by fitting the distribution to an annual series, the return period (years) of any event magnitude x can be computed from $T = 1/[1-F(x)]$.

2.3 Methodology for PMF Estimation

The PMF is estimated from the PMP through a rainfall/runoff transformation. The hydrologic analysis was performed with the Watershed Modeling System (WMS) from Scientific Software Group (2002). WMS is a comprehensive catchment analysis and modeling package which allows the precise computation of a basin's

morphological attributes from its Digital Elevation Model (DEM). These are then used, in turn, to construct the input parameter set to a variety of hydrology interfaces, one of which is the Hydrology Modeling System (HMS) from the U.S. Army Corps of Engineers (2006). The HMS model was used in the present study to perform the rainfall/runoff transformation for the computation of the PMF.

Within HMS, the NRCS Curve Number method (CN) is used to convert PMP and other rainfalls into PMF and other runoff hydrographs. The CN is estimated from land use and soil type coverages for the study area. Other parameters, such as the lag-times and times-to-peak, are computed directly by WMS.

The computation of runoff hydrographs requires specifying a rainfall temporal distribution, which is itself dependant on the selected rainfall duration. For the present study a 24-hour PMP was determined, in turn requiring a 24-hour distribution. In most design studies a standard Type-II rainfall distribution is applied. This distribution allocates most of the rainfall within a central 6-hour period of the storm. It is unknown whether the PMP will actually be distributed in such a fashion.

In order to consider perhaps a more realistic, and critical, rainfall distribution pattern, the quartile distributions presently available in NOAA's Atlas 14 were considered. Atlas 14 provides distributions for four quartiles, the most critical of which, in

terms of producing the largest rainfall during the initial stages of a storm, is the First Quartile distribution. Several sub-distributions exist within each quartile, classified according to a level of exceedance which ranges between 10% and 90%. Since they are the most critical, the study considered distributions from the First Quartile.

2.4 Results for the PMP Estimation

The PMP estimate is based on the analysis of the available records at weather stations nearest to the Valenciano river basin. The description of these stations is presented in the following table, while their specific location is plotted in Exhibit1 of the Exhibits section.

Table 1. Rain Gaging Stations Near the Valenciano River

Station Name	Distance to Valenciano Basin (km)	Period of Record	Record Length (years)
Juncos 1 SE	7	1931 - 2006	74
Cubuy	11.4	1973 - 2003	30
Gurabo Substation	14.5	1971 - 2003	33
San Lorenzo Farm 2N	7.2	1925 - 1988	46[a]
Las Piedras 1N	5.3	1973 - 2003	31

a- Reflects a number of missing years

Of the stations presented in Table 1, only that of Juncos has an adequate record length, but it is not the nearest station. Although one of the closest, San Lorenzo has a number of missing years and no data are available after 1988. The reasons for the gap in the data are unknown, and as such the consistency and homogeneity of the record may be cast in doubt. For this reason the San Lorenzo data was excluded from the analysis. The available rainfall data for the selected Juncos 1SE station is presented in Appendix A.

A question arises as to the interdependence of the data for the listed stations and whether these can be treated as independent variables. A correlation analysis may not yield reliable results in the present case because of marked differences in record lengths of the principal stations. Such an analysis was not performed in the present study. However, analysis of the data indicates that several of the major events at each station occurred concurrently. Some of them, such as the 1996 events, are related to the passage of hurricanes, such as hurricane Hortense for that year. This would corroborate the existence of a non-negligible degree of interdependence between the series.

While no definite procedure exists for determining the minimum inter-station distance as a means of justifying the independence assumption, Elíasson (1997) has suggested the use of 30 km as a

guideline. If such is the case, then the listed stations near the Valenciano river are in all likelihood interdependent and thus correlated, even more so if the meteorological process is heavily influenced by orographic patterns, as is indeed the case here.

A lack of independence implies that the annual rainfall series cannot be grouped to increase the sample size and thus produce more robust statistical parameter estimates. Thus, following the suggestion from Elíasson (1997), the PMP analysis is performed for each station and the most appropriate value designated as the design value for the study.

As discussed earlier, the Hershfield method was deemed adequate for the stated objectives of the Valenciano reservoir design project. The method requires the estimation of annual rainfall statistics, mainly the mean and standard deviation. Hershfield's frequency factor is then applied to obtain the PMP. In a refinement of the method, Hershfield (1965) provided a correction to the frequency factor based on the mean annual maximum rainfall.

The following table presents the results of the statistical analysis, with the estimation of the 24-hour PMP and the maximum recorded rainfall at each station:

Table 2. Results of the PMP Analysis for Valenciano River

Station Name	Mean (inches)	Standard Deviation (inches)	Frequency Factor k_m	PMP (inches)	Maximum Recorded (inches)[a]
Juncos 1 SE	4.84	3.15	14.9	51.8	20.55
Cubuy	7.03	3.22	12.8	48.2	14.7
Gurabo Substation	5.12	2.28	14.6	38.4	12.1
Las Piedras 1N	7	3.88	12.8	56.7	19.4

a-The maximum recorded rainfall at the discarded San Lorenzo station was 13.73 inches. The maximum historical 24-hour rainfall ever recorded was 71.85 inches, at La Reunión (Maidment, 1993). From a general overview of Puerto Rico stations it is apparent that no rainfall over 30 inches within a 24-hour period has ever been recorded. One of the highest ever recorded in Puerto Rico was 22.90 inches, measured at the Cerro Maravilla station in 1985.

The maximum PMP turned out to be that from the Las Piedras station. However, the Juncos record length would argue that it is the most reliable and the one less subject to estimation errors. This is an issue that cannot be resolved in the present study, and one which would require a profound statistical analysis of the data.

In the absence of any other criteria, the recommended PMP value would be 52.0 inches, based on the Juncos data, which must be considered the most statistically reliable of the set, if only due to its record length.

Neither station transposition nor area reduction factors were applied to the PMP. The Valenciano drainage area is relatively

small, and it is entirely probable that the PMP could be evenly distributed over the whole catchment.

2.5 Return Period of the PMP

Estimating the return period of the PMP contradicts the concept of the PMP in the sense that it represents an upper bound of a process that cannot stretch to infinity. In using Hershfield's method, no upper bound of the PMP is prescribed by using a finite multiple of sample standard deviations. Koutsoyiannis (1999), using data from Greece, estimated that Hershfield's method could imply an event return period of 60,000 years. However, this need not necessarily be the case for all locations, particularly in the tropical regions.

Although it may not be appropriate to formulate the concept of PMP within an event return period framework, it is nevertheless instructive to estimate the implied return period of the PMP from a probability distribution that has been fitted to the annual rainfall series.

As discussed previously, the Generalized Extreme Value (GEV) distribution is applicable to annual rainfall maxima, and is used in the present study to assess the possible frequency of the PMP as computed with Hershfield's method. The GEV distribution is a three-parameter model. The parameters were estimated from sample data by using the method of Probability Weighted Moments (PWM).

The PWM estimation procedure has been found particularly applicable to short-duration series. This estimation procedure is related to the L-Moments estimation procedure, which is presently widely used in flood frequency estimation studies.

The parameter estimation procedure used in fitting the GEV model to the sample data is described in Maidment (1993), and Kotz and Nadarajah (2000). The GEV was fitted to the Juncos data, which has the most extensive record of all the nearby stations. The results of the distribution fitting are presented in Figure 1. A reasonably good fitting was obtained for the Juncos data. The PMP is beyond the data range and its return period must be estimated from the inverse of the complement of the cumulative density function, as defined by Equation (2).

Comparison of the return periods from the GEV distribution with those proposed in NOAA's Atlas 14 may occasion some illustrative observations. Atlas 14 estimates return periods only up to 1000 years and it will not be possible to provide a direct comparison with the GEV-fitted model, but the trend may be noted and can yield some indication of the range within which the return period of the PMP may be located. The return periods of the GEV model and the Atlas 14 are compared in Table 3.

Table 3. Comparison of 24-Hour Rainfall Return Periods[a]

Return Period (years)	Atlas 14 Rainfall (inches)	GEV Model (inches)
100	17.5	19.2
200	20.3	25.2
500	24.4	36
1000	27.8	47.4
1500	(not computed)	55.6
2000	(not computed)	62.3

a- At the Juncos 1 SE station

The GEV model estimates in Table 3 are substantially higher than those from Atlas 14, although they have been found to be much closer for the lower return periods (not shown). It must be noted that the GEV distribution approach used only the annual series available at the Juncos station, and with a different data plotting position from that of the atlas. Atlas 14 is based on a comprehensive regional data analysis effort that provided reliable parameter estimates for return period estimation, and must be considered the most accurate.

Using the GEV distribution, the estimated return period of the recommended PMP of 52 inches amounts to almost 1270 years. Given the level of uncertainty associated with extended extrapolations of small-sample trends, the estimation of the return period of such extreme events is an unreliable endeavor. The most that can be

said, when considering the comparisons depicted in Table 3 and given that Atlas 14 provides reasonable reliable return period estimation, is that the actual return period of the PMP will surely be much greater than 1270 years. This is a welcome result since an extremely high return period, such as 60,0000 years at some locations (Kuotsoyiannis, 1999), has been associated with the PMP estimated from Hershfield's method.

2.6 Hydrology Analysis Methodology

The hydrologic study was conducted with the Watershed Modeling System (WMS) from Scientific Software Group (2002). WMS is a comprehensive hydrology package that allows the analysis of morphological catchment attributes based on a Digital Elevation Model (DEM) of a fluvial system. The model has several hydrology interfaces, one of which is the Hydrology Modeling System (HMS) from the U.S. Army Corps. WMS extracts a catchment's morphological data from a DEM to create an input data file for the hydrologic interface, in this case the HMS. The DEMs, available from the U.S. Geological Survey, are traced within a 30 m × 30 m grid size resolution.

The hydrology model used in the study is the Curve Number (CN) method from the Natural Resource Conservation Service (NRCS), as programmed in HMS. The CN method requires land use and soil type information for estimation of the rainfall transformation

parameters. Other required parameters, such as the runoff hydrograph's lag-time are also computed by WMS.

The hydrologic analysis is based on a 24-hour rainfall distribution. No specific rainfall temporal pattern for the PMP has been proposed to date. It was assumed that either the Type-II distribution or the Quartile distributions from Atlas 14 would be appropriate.

The results of the hydrologic analysis, mainly the resulting runoff hydrographs, constitute the baseline data for the hydraulic study and dam break analysis.

2.7 Hydrologic Analysis

A hydrologic analysis was performed for all the streams associated with the Valenciano river. The PMP was applied exclusively to the Valenciano basin as far downstream as the proposed dam location. The major system in the area is the Grande de Loíza river, which feeds the Carraízo reservoir downstream of the Valenciano river. Exhibit 2 depicts the study area on a portion of the topographic survey map of the region. The basin configuration used in HMS, including the sub-basin identification, are also shown in the exhibit.

The Valenciano river catchment was modeled with the PMP as well as with rainfall with other return periods. Only Valenciano was modeled with the PMP, following the Federal Guidelines for Dam

Safety from FEMA (2004). Rainfall for the other return periods were obtained from Atlas 14 by selecting the 24-hour rainfall depth nearest the centroid of each watershed. The resulting rainfall depths are listed in the following table.

Table 4. 24-Hour Sub-Basin Rainfall Depths (Inches)

	Return Period			
Sub-Basin	PMP	100-Year	50-Year	25-Year
Valenciano river at damsite	52.0	19.3	16.4	13.7
Gurabo river	-	19.5	16.6	14.0
Gurabo river downstream of confluence with Valeciano	-	16.0	13.7	11.5
Grande de Loíza river	-	20.5	17.3	14.4
Turabo river	-	14.8	12.8	10.8
Cagüitas river	-	13.7	11.8	10.0
Bairoa	-	15.1	12.9	10.9
Carraízo		15.6	13.4	11.3

Sub-basin hydrologic parameters were determined from WMS following the procedure described earlier. The CN parameter was determined as an area-averaged value based on current land use distribution and general soil type characteristics. Aside from the major urban centers, most of the catchments are vegetated. A CN value of 80 was assigned to the sub-basins, save for the Valenciano basin, which was assigned a value of 94 to reflect AMC-III

conditions. AMC-III conditions were assigned in conformity with the definition of the PMF, which requires the consideration of extreme conditions for runoff generation as per its definition. The soil type and land use data used to compute the CN for the Valenciano drainage area is described in Appendix B.

The hydrograph lag time, being the other major parameter, was computed by WMS for each sub-basin. The lag time was computed with Kirpich's equation (Maidment, 1993). This particular equation was selected because it provided reasonable results, in contrast to the standard NRCS time lag equation, which yielded some unreasonable results. The computed drainage areas and lag-times for each sub-basin are listed in the following table.

Table 5. Sub-Basin Areas and Lag-Times

Sub-Basin	Drainage Area (mi^2)	Lag-Time (hours)
Valenciano river at damsite (3B)[a]	14.78	2.11
Downstream of Valenciano dam (2B)	7.72	1.35
Gurabo river (1B)	25.74	3.48
Gurabo river downstream of confluence with Valenciano (4B)	21.47	3.28
Grande de Loíza river (10B)	55.56	4.27
Turabo river (11B)	32.43	2.87
Cagüitas river (9B)	18.66	3.43
Bairoa (7B)	7.18	2.96
Carraízo (12B)	20.36	3.18

a- Sub-basin ID used in hydrologic model

The next step was distributing the rainfall depths according to the Type-II and Quartile distributions from Atlas 14. With the Quartile distribution, the First Quartile, 20% and 30% exceedance level temporal distributions were considered in the analysis. The distribution yielding the highest peak flow rate was selected as the recommended result. It turned out that the Type-II storm distribution yielded the highest flow rate.

Hydrographs generated with HMS were routed downstream of the Valenciano dam site down to the Carraízo lake area. The peak flows obtained from this analysis are presented in Table 6.

Table 6. Estimated Peak Flows for the Study Area

	Peak Flows (m^3/s)		
Return period (years)	At Valenciano river dam site	At confluence with Gurabo river	At Carraízo dam site
PMF	2757	4074	7835
100	957	2301	6412
50	797	1919	5335
25	649	1565	4338

The entries in Table 6 for the PMF flows at Gurabo and Carraízo were obtained by combining the PMF flood for Valenciano with the 100-year floods from the other streams. These 100-year floods were also convoluted with the 100-year flood from Valenciano and presented as the 100-year flows. The results of the 100-year

simulation are used in the calibration of the hydraulic model with the regulatory flood profile. The simulation output files for the hydrologic analysis are presented in Appendix C.

The runoff hydrographs from the PMF analysis constitute the baseline input data for the subsequent dam-break study.

III. HYDRAULIC STUDY

3.1 Introduction

The hydrologic study presented and discussed the results of the flood flow analysis of the basins related to the Valenciano dam project. The results of that study constitute the boundary conditions of the unsteady flow simulation of the hydraulic study.

Several dam break scenarios were modeled in the hydraulic analysis. The basis for these scenarios are the recommendations from FEMA's guidelines for dam safety (Federal Emergency Management Agency, 2004).

3.2 Methodology

The dam break analysis was performed with the deterministic HEC-RAS model from the Army Corps of Engineers (2005). Recent versions of HEC-RAS have the capability of performing dam break analyses. Indeed, recent studies have demonstrated that HEC-RAS can yield similar results as those obtained from the National Weather Service's FLDWAV model (Gee and Brunner, 2005).

Dam breaks are modeled in HEC-RAS as breaches of inline structures. An inline structure is placed in a specific cross section, and the dam break option in the flow analysis scenario is activated for the appropriate simulation. The river drainage system is described in the model in the form of cross sections.

Cross sections were traced from aerial survey topographic data by the contracting firm of CSA Architects and Engineers. Cross sections culled from aerial surveys may lack the resolution that would allow the accurate tracing of the river channel. In the present case, the cross section data was carefully analyzed to yield a reasonably consistent data base that would reduce the chance of obtaining unstable and unrealistic solutions.

In the particular case of the Valenciano river we have a relatively small stream which is a tributary to a larger basins, the major system being the Loíza river. As discussed above, simulating the entire drainage system of Lake Carraízo would have required a major surveying effort to achieve a working hydraulic model. A compromise was achieved by simplifying the stream network to comprise the Valenciano river and the Gurabo river downstream of their junction as a continuous stream. The upper Gurabo river was treated separately. The unsteady flow analysis was performed for the Valenciano and lower Gurabo channels, while for the upper Gurabo river a separate backwater analysis was performed upstream from the junction with Valenciano. This approach is more conservative since the runoff hydrograph from the upper Gurabo basin accrues without attenuation at the junction with the Valenciano river, and is described as an inflow boundary condition in the unsteady flow model.

Downstream, the unsteady flow analysis was performed up to a location about 600 meters upstream from the confluence of the Gurabo and Loíza rivers. The contributing drainage areas along the channel were represented as boundary elements at which input runoff hydrographs were specified.

Several scenarios were analyzed according to either the calibration requirements or the recommended dam break scenarios from the FEMA guidelines. These can be summarized as follows:

(a) Simulation without the Valenciano dam to define the natural condition for calibration with the regulatory flood profile from the current FEMA flood map, using the 24-hr, 100-year flood;

(b) Simulation with the Valenciano dam, without dam break, with the 24-hr, 100-year flood, and the Probable Maximum Flood (PMF) for the Valenciano basin, with the 24-hr, 100-year flood at the other boundaries;

(c) Simulation of the dam break with the PMF and 100-year flood for the Valenciano basin, with the 24-hr, 100-year flood at the other boundaries.

(d) Simulation of the sunny-day breach with no inflow to the Valenciano reservoir, and with the 100-year flood at the other boundaries.

The geometric description of the Valenciano reservoir was supplied by the contracting firm. Basically, the dam is a concrete structure with a gated overflow spillway. Breach parameters for concrete slab structures are not well defined (Wahl, 1998). Most studies recommend prescribing a nearly instantaneous breach formation for concrete gravity structures (National Weather Service, 2000) and this particular approach was followed in the present project.

3.3 Simulation Model Data

The basic model input data consists of channel cross sections and the flow data from the hydrologic analysis. A total of 28 cross sections were assembled to define the channel system, comprising the Valenciano and Gurabo rivers downstream to a point about 600 meters upstream of the confluence with the Loíza river. The cross section location layout used in the HEC-RAS model is displayed in Exhibit 3. Listed in the following table are the cross section identification numbers used in the model.

Table 7. Cross Section Model Identification

Lower Gurabo River		Valenciano river		Upper Gurabo river	
Station	Model ID	Station	Model ID	Station	Model ID
00+00.00	3	140+90.14	58	00+00.00	22
10+49.13	4	149+61.11	67	03+72.00	23
18+73.47	5	154+93.11	69	06+13.48	24
39+54.87	6	161+57.31	73.04[a]	13+33.43	25
58+73.77	7	168+07.28	77[b]	17+94.43	26
71+34.74	10	169+01.92	78	26+14.43	27
109+92.07	14	170+52.51	79	30+60.98	28
117+79.17	15	173+86.03	80	37+94.98	29
127+61.26	17	213+42.94	100[c]	42+73.29	30
133+47.11	21[d]				

a- Interpolated cross section; b- Dam located just upstream of this cross section; c- outside the coverage of Exhibit 3; d- at junction of Valenciano and upper Gurabo rivers

Runoff hydrographs from the hydrologic study serve as inflow boundary conditions of the hydraulic model. Presently, four runoff hydrographs are used at the boundaries of the network. At the first downstream cross section a normal depth boundary condition was assumed defined by the energy slope derived from the regulatory flood profile. Runoff hydrographs impinging on the river downstream of the dam are treated as lateral inflow hydrographs at specific sections. Figure 2 depicts the PMF contributing flood

hydrograph to the Valenciano dam. The locations of the hydrograph boundary conditions are presented in schematic fashion in Figure 3.

The Valenciano dam is a concrete structure with a gated spillway and a non-overflow section, as described in the following table.

Table 8. Description of Valenciano Dam

Feature	Measure (m)
Spillway elevation	95.4
Crest length	72.5
Non-overflow section elevation	103

The proposed spillway section will hold flow control gates. In the simulation it was assumed that such gates would be open. Figure 4 depicts the representation of the Valenciano dam in the HEC-RAS model.

The dam breach analysis was performed following recommended guidelines for concrete dam failures, which essentially consist of prescribing a nearly instantaneous structural collapse. Most of the available parameter estimates are defined for earthen dams while no reliable generalizations are available for concrete structures. Several dam breach scenarios were simulated to assess the sensitivity of the resulting water surface profiles to

variations in breach parameters. The most critical scenario was adopted.

Simulation of the dam breach was performed by assuming a trapezoidal or rectangular failure section arising from the bottom of the channel cross section at the dam site. Several scenarios were analyzed to permit the identification of the most critical one following the criteria of the highest tailwater elevation at the dam. While these analyses will be discussed in the following section, the following table presents, in advance, the basic breach parameters proposed in the study.

Table 9. PMF Dam Break Parameters

Parameter	Magnitude
Center station	188 m
Final bottom width	16 m
Final bottom elevation	73.3 m
Left side slope	1:0.5
Right side slope	1:1.0
Full formation time	5 min

The side slopes of the breach were set to conform to those of the cross section were the dam was located. The final bottom width is that of the channel section at the dam.

Performing the unsteady flow simulations required using a hot-start, or restart file for achieving stable runs. The use of such files is a typical feature of unsteady flow analysis packages and they are required in many situations in order to avoid runaway instabilities and divergent results. They provide baseline results for the unsteady flow analysis and help prevent such problems as may arise from channels going dry at some stages or wildly oscillating results from rapidly changing flows, among others. All hot-start files in the model were created with a steady flow of 100 cubic meters per second.

3.4 Simulation Results

Before performing any simulations, the model was calibrated with the regulatory flood profile from the current FEMA flood map for the Carraízo area. The calibration permitted the estimation of the roughness parameters of the stream channel and overbank areas within the region covered by the insurance study. Except for a stretch in the vicinity of Juncos, the model calibrated exceedingly well with the regulatory 100-year flood profile.

The calibration yielded somewhat lower roughness factors than those used in an earlier Valenciano dam break study (Infrastructure Financing Authority, 2001). No discussion was provided in the referenced report as to the provenance of the roughness estimates used therein. The lower roughness estimates of the present study

produced correspondingly lower flood elevations than those of the cited study. However, the estimates obtained through calibration with the regulatory flood profile reflect field conditions from an official, comprehensive study and should be considered representative.

The breach was constrained by the bottom width of the channel at the cross section where the dam was located. The break was initiated at a time when the water elevation at the dam was near the maximum water elevation which had resulted from the no-breach scenario. Given that the simulation originated at 0000, the breach starting time was set at 1300 hours. The peak flow rate occurs at 1400 hours. However, 90% of the peak flow rate is achieved by 1325 so that the higher runoff rates are observed much earlier than the peak time. Figure 5 depicts the full model breach formation.

While the Federal guidelines suggest initiating the breach at the peak flow, in the present case the highest tailwater elevations resulted when initiating the breach at 1300 hours. This may be due to a number of reasons, including particular programming features of the hydraulic model. A number of trial simulations were performed, varying breach initiation times, water elevations at failure, and simulation time interval. The most agreeable and stable solution proved to be that with the 1300 hours breach.

The Valenciano dam was assumed to be initially full for the analysis of the dam break scenarios. Actually, due to the restart file option, a relatively small head will exist at the spillway at the onset of the inflow runoff hydrograph. This was a necessary condition since a zero flow situation in an unsteady flow analysis may produce numerical instabilities, particularly in steep reaches with supercritical flow such as the one existing downstream of the Valenciano dam.

To test the model sensitivity, several breach scenarios were simulated by varying breach geometry and breach formation times. Several of the more relevant scenarios for varying breach geometry are presented in the following table.

Table 10. PMF Dam Break for Varying Breach Geometry[a]

Left Side Slope	Right Side Slope	Bottom Width (m)	Tailwater Elevation (m)
1:0[b]	1:0	10	86.91
1:0	1:0	16	88.42
1:0.5	1:1	16	90.94
1:0.5	1:1.35	16	90.94

a- For a 5 min breach; b- defines a rectangular breach

The above results show that the breach bottom width has some effect on the resulting tailwater elevation. Side slopes are different in order to conform to the channel bank slopes. Varying

the right side slope above, to a certain extent, did not change the resulting tailwater elevation to a noticeable degree. Barring the effects of uncertainties and other factors, the results in Table 10 suggest that a trapezoidal breach with the given dimensions is adequate for the stated purpose.

The effect of the breach formation duration can also be generally assessed through a sensitivity analysis. Results of such an analysis, using the recommended breach parameters from Table 9, are presented in the following table:

Table 11. PMF Dam Break for Varying Breach Formation Times[a]

Breach Formation Time (min)	Tailwater Elevation (m)
0 - 5	90.94
10	89.05
15	88.99
20	88.83

a-For the breach geometry in Table 9

The results in Table 11 are less conclusive than those in Table 10. Although it is evident that the smaller breach initiation time yields a higher tailwater, the elevation is not very sensitive to higher durations. It must be stressed that long breach-formation times are apparently atypical for concrete dams, and it may be appropriate to assume very small formation times.

However, it needs pointing out that the time interval specified for the analysis may at times blur the time differences between breach formation options. For example, with a computation interval of 10 minutes, the 15 and 20 minute breaches will both occur within a 20 minute simulation period.

There were a number of other technical issues associated with the modeling of the dam breach scenarios which had to be addressed in the study, including programming constraints within the model itself. These were given due consideration in the analysis to produce the end results that are finally proposed.

The 100-year, PMF, and sunny-day breach scenarios were simulated with the breach parameters in Table 9 to compute the resulting water surface profile for each. The results of this analysis at several critical locations downstream of the Valenciano dam are summarized in the following table.