APPPENDIX

HYDROLOGY AND HYDRAULICS

DETAILED PROJECT REPORT
ENVIRONMENTAL IMPACT ASSESSMENT

LEVEE SYSTEM ALONG THE CAÑO MADRE VIEJA AREA FOR THE DISCOVERY BAY RESORT AND MARINA DEVELOPMENT RIO CULEBRINAS AT AGUADILLA AND AGUADA PUERTO RICO

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I. INTRODUCTION

Two hydrologic-hydraulic analyses were performed for the marina and resort development project. The first is a comprehensive analysis of the impact of the proposed levee system in the Caño Madre Vieja sector of the Río Culebrinas basin, and the second is the interior drainage analysis related to this same levee system. The study performed for the levee system is the basis of the Flood Insurance Study (FIS) map revision request to be submitted to the Federal Emergency Management Agency (FEMA) via a Conditional Letter of Map Revision (CLOMAR) solicitation, and is also the basis of the Environmental Impact Statement (EIS) of which this document is to be a part of.

This appendix includes the results of both the levees and interior drainage studies as well as additional analyses pertaining to EIS related issues. The impact of the residual flooding related to the levee system is also assessed.

II. DESCRIPTION OF STUDY AREA

Río Culebrinas is located in the northwestern part of Puerto Rico. It is an intermediate-sized basin coursing along several municipalities. Río Guatemala, Río Caño, and Río Sonador as well as several named and unnamed creeks are all tributaries to Río Culebrinas. Amongst the named creeks are the Grande, Las Marías, Los Morones, Lasalle, Viejo, El Salto, and Salada. The basin is mostly forest and pasture with some urban development related to the towns of San Sebastián, Aguada, and Aguadilla, also including commercial and industrial areas, housing projects, and rural developments. Sugar cane is still grown in the coastal plain but increasing development has encroached into this and other formerly cultivated areas.

The climate of the region is tropical, with a rainy season lasting from June to December, although it is not uncommon to observe significant downpours outside this season. Average annual rainfall in the basin ranges from 70 inches (1,778 mm) along the coast up to 100 inches (2,540 mm) in the mountainous area.

The Río Culebrinas' coastal plain is bounded on the east side by Highway PR-2 (Hwy-2), which connects the hills straddling the flood valley on its north and south sides. Figure 1 shows the photographic image of the project area on which several relevant features are identified. A significant feature of the valley is Caño Madre Vieja, an old outlet of Río Culebrinas which winds its way to the coast northwest of the Espinar area, in the Parque Colón sector of Aguadilla. Caño Madre Vieja is nearly stagnant as its outlet is frequently blocked by sand dunes. The residential area of Espinar is located near the middle of the coastal valley and, in effect, divides the flood of Río Culebrinas.

The proposed project is to be located in the Caño Madre Vieja sector. It consists of two levees, one labeled the Aguadilla levee and the other the Aguada levee, after the municipalities within which they are located. The Aguadilla levee will run west of this town's built-up area, from Hwy-2 up to an existing coastal road. The Aguada levee will wind along the eastern fringe of the Espinar sector, from a high-ground location near the southeastern tip of the sector up to a location near the coastline. The study area comprises the floodplain west of Hwy-2.

Additionally, project will include two entrance jetties to protect the north perimeter of the marina basin.

III. FLOODING HISTORY

A significant number of flooding events have been recorded in Río Culebrinas. The most memorable recorded event, in terms of extent of flooding, is the so-called Eloísa flood of September 16, 1975, resulting from a stationary depression. This was the most severe flooding in the basin since the 1956 flood associated with hurricane Santa Clara, and to date is still the largest on record. The return period of the Eloísa event is hard to ascertain since the systematic period of record is relatively short and discontinuous, and therefore no reliable flood frequency estimates can yet be computed.

Systematic recording of flows by the U.S. Geological Survey (USGS) has been underway since 1968 at three locations within the Culebrinas basin. The gaging station with the best record is USGS 50147800, located at Road PR-404 in the municipality of Moca. A drainage area of 71.2 square miles (184.5 square kilometers) is associated with this station. Table 1 lists the major annual flood events recorded at the Moca gaging station since 1968.

The Federal Emergency Management Agency (FEMA) has performed a Flood Insurance Study (FIS) on Río Culebrinas (Ref. 1) that has served as the basis of the regulatory flood map from the Puerto Rico Planning Board (PRPB). The portion of the regulatory flood map pertaining to the study area is depicted in Plate 1. The current flood map is the baseline condition against which the hydraulic impact of all proposed projects must be assessed.

IV. HYDROLOGIC STUDY

The hydrologic study was performed for the entire Río Culebrinas' drainage area, and flood hydrographs were computed at selected locations of the river. The study was performed following FEMA guidelines for FIS revisions. In the existing FIS study, a hydrologic analysis was performed as far downstream as Highway 2, at which runoff hydrographs were computed that served as the inflow boundary condition for a two-dimensional hydraulic study of the floodplain. Upstream of Highway 2, a one-dimensional flow analysis was performed to trace the water surface profile. A similar approach has been followed in the present study.

FEMA requirements dictate that a FIS revision request should either duplicate the original study or propose new results based on improved modeling or new field data reflecting different baseline conditions with respect to the original study. Given the size of the drainage area and based on an analysis of the present land use patterns, a revision of the hydrologic results is not warranted. Although in the present study the flow estimates at given locations will exhibit some variation with respect to the FIS flows, these are not deemed significant and are within the expected level of error of such analyses.

The major results of the hydrologic study are the resulting flood hydrographs for various return periods. These hydrographs serve as the inflow boundary conditions of the hydraulic study.

4.1 Methodology

The hydrologic study was performed with a rainfall/runoff model. Rainfall depths of various return periods were converted to runoff via the Natural Resource Conservation Service's (NRCS) Curve Number (CN) method, and the standard NRCS unit hydrograph used to generate the resulting flood hydrographs. Hydrographs generated at watershed subdivisions were then routed downstream to the outlet, this being the Highway 2 bridge at the neck of the Río Culebrinas' coastal floodplain.

The hydrologic simulation of the Culebrinas basin required the determination of numerous morphological parameters. These parameters were computed with the Watershed Modeling System, or WMS (Ref. 2). WMS is a comprehensive landform analysis package that can glean morphological attributes from a basin when its attendant topography is in the form of a Digital Elevation Model (DEM). The package includes several hydrology interfaces, one of which is the U.S. Army Corps' Hydrology Modeling System, or HMS (Ref. 3). The NRCS method was applied via the HMS model. DEM coverage for the entire Culebrinas basin was obtained from the U.S. Geological Survey (USGS) and used to trace drainage subdivisions, compute drainage areas, land slope, and other parameters.

Hydrologic parameters, such as the CN, were computed from a digitalized overlay of land use and soil type distribution over the Culebrinas basin. The computations were also performed with the WMS model. Drainage area lag times were computed with WMS as well by selecting an available equation for which the model would, in turn, calculate the required physical parameter.

4.2 Watershed Data and Parameters

The Río Culebrinas drainage area was subdivided into 14 sub-basins based on fluvial and morphological characteristics. At each sub-basin the requisite morphological and hydrologic parameters were computed with the WMS model from the available DEM overlay for the Culebrinas area. The DEM coverage of the basin was in turn digitally superposed on the topographic quadrangle to identify land features and fine-tune the drainage divide subdivisions traced by WMS. Drainage subdivisions and other features are identified and labeled in Plate 2.

Land use and soil type data was available in the form of digital map overlays. Each soil type was assigned the appropriate runoff potential index (A, B, C, or D) based on the soil classification listing available in TR-55 (Ref. 4). The distribution of soil types over the drainage area, in terms of the runoff potential index, is presented in Plate 3. It is seen in the plate that soil types B and D predominate. Soil type B is characterized by a relatively good infiltration potential, referring to gravely or sandy soils, while type D have a poor infiltration potential, as would be the case for clayey and loamy soils.

Land use distribution over the drainage area is depicted in Plate 4, using the classification scheme of the NRCS Curve Number tables. The area is mostly grassy or rangy, with woody-type vegetation in the upland areas. The lowland along Río Culebrinas is cultivated with sugarcane, and it has been assumed that this type of crop would be of the grassy type within the peak of the rainy season.

The digital land use/soil type maps were superposed on the drainage subdivisions from the DEM coverage to produce land use/soil type matrixes from which the composite Curve Number (CN) parameter used in the NRCS model was computed by WMS. Table 2 lists the drainage area and computed CN, for both AMC-II and III moisture conditions, for each sub-basin. The total drainage area amounts to 102.6 square miles (266 square kilometers).

Construction of the NRCS unit hydrograph requires the estimation of a watershed's lag-time. The lag-time for each sub-basin was computed with the NRCS equation:

$$t_L = \frac{L^{0.8}\left(S+1\right)^{0.7}}{1900\,Y^{0.5}} \qquad (1)$$

where t_L = lag-time (hr); L = hydraulic flow length (ft); S = maximum water retention potential (inches), computed from S = 1000/CN − 10; Y = average watershed slope (%).

The computed lag-times for each of the Río Culebrinas sub-basins are listed in Table 3. The lag-times are used in the derivation of the synthetic unit hydrograph for each sub-basin. The unit hydrograph is used to generate the resultant composite hydrograph via the convolution process with the incremental runoff resulting from the rainfall-runoff transformation. The 24-hour precipitation accumulation is distributed according to the standard Type-II rainfall distribution, and the rainfall-runoff transformation computed incrementally.

Precipitation depths over each sub-basin were obtained from the Weather Bureau's Technical Paper No. 42 (Ref. 5), or TP-42. At the time of the performance of the hydrologic analysis, TP-42 was the standard source for rainfall frequency data. An additional justification for using TP-42 is the requirement of relating the present study to then existing FEMA's baseline flood insurance study. The selected precipitation depths for various return periods at each sub-basin are listed in Table 4.

4.3 Hydrologic Modeling Results

Prior to modeling the hydrologic response of the drainage area, the assembled precipitation data is converted to runoff depth using the NRCS transformation. This transformation is applied to the standard Type-II, 24-hour cumulative rainfall.. The NRCS rainfall-runoff transformation is given as follows:

$$R = \frac{\left(P - 0.2S\right)^2}{P + 0.8S} \qquad (2)$$

where R = runoff depth (inches); P = cumulative rainfall (inches); S = retention potential described in Equation (1).

The runoff depths are applied to the computed unit hydrograph to perform the requisite convolutions needed to produce the various runoff hydrographs for each sub-basin. This is accomplished with the hydrologic model.

Application of the WMS/HEC-HMS hydrology model to the drainage network of Río Culebrinas yielded runoff hydrographs for various return periods. The hydrographs generated at each subdivision were routed through the channel network down to the selected outlet at the Hwy-2 bridge. Flood hydrographs constitute the flow boundary conditions of the hydraulic model. Hydrographs were generated for various return periods, the most relevant of which were the 100 and 500-year events.

The hydrograph peak flows of the present study are compared to those of from FEMA's FIS study in Table 5. A rather close agreement is observed between the peak flows of the present study and those from the FIS. It is for this reason that the present study has not been presented as a revised hydrologic analysis.

V. HYDRAULIC STUDY

The hydraulic study was performed with a two-dimensional, unsteady flow model, in conformity with the requirement of using a similar or improved methodology than that used by FEMA for the Río Culebrinas FIS. The overall objective of the hydraulic study is the assessment of the impact of the proposed levee system on the flooding characteristics of Río Culebrinas. Specifically, the study sought to (a) assess the feasibility of constructing the levee system, (b) amend the regulatory flood map to include the proposed conditions, and (c) analyze the interior drainage system to minimize the impact of residual flooding and provide mitigation against flooding for the interior areas.

5.1 Methodology

The hydraulic analysis was conducted with the unsteady, two-dimensional Flo-2D model (Ref. 6). Flo-2D utilizes a finite-difference solution of the full dynamic unsteady flow equations. It is capable of simulating a varied range of unsteady flow applications, and is one of the models allowed by FEMA for use in FIS revision studies.

Data for the Flo-2D model was gleaned from a digital aerial survey topographic map supplied by the firm of Hernández Virella & Associates. The model constructed a two-dimensional grid of the Culebrinas flood valley downstream of Hwy-2 down to the coastline. This grid included the Río Culebrinas and Caño Madre Vieja areas. Inflow hydrographs were specified at the boundary nodes of the grid and routed through the network out into Aguadilla Bay.

The procedure called for calibrating the model with the current FIS regulatory flood in order to ascertain the hydraulic model parameters, particularly the surface roughness values.

After calibration, two sets of simulations were performed, one pertaining to the existing field condition, that is, without the levees, and the other to the proposed condition, with the levees. Existing conditions are those pertaining to the FIS flood, with the updated field data from the aerial survey.

The results of interest in the simulations are the water surface profiles and the flow velocity field within the levees. The water surface profile upstream of the levees should not increase appreciably above that of the FIS profile. Other variables, such as levee height and freeboard are dependent on the water surface profile from the hydraulic simulation.

5.2 Levee Description

Two levees are proposed, one labeled the Aguadilla levee to the north, and the other the Espinar levee to the south, straddling the Caño Madre Vieja, with the alignment as depicted in Plate 5. Plate 6 depicts the levee alignment on the topographic map of the plain and includes other related design features. The Aguadilla levee has some similarity to that proposed by the U.S. Army Corps of Engineers (USACOE) in an earlier study (Ref. 7). The Espinar levee is tailored to accommodate the Marina concept.

Levee heights were determined from the results of the hydraulic simulation with the 100-year flood runoff hydrograph and in compliance with 44 CFR Ch. 1, Section 65.10 of the NFIP regulations. Incremental depths were added to the flood profile to account for freeboard, wave runup, wind setup, wave height, and overtopping considerations.

5.2.1 Aguadilla Levee

The Aguadilla levee extends from Hwy-2 to the coastline in the Parque Colón area, running between the San Carlos parochial school and the old municipal baseball park, for a length of about 1.76 kilometers. East of the Aguadilla levee lies the southern part of the municipality of Aguadilla. This area includes the Victoria and García housing projects. The levee will cut across the Caño Madre Vieja stagnant watercourse at several locations. It will also cut across Road PR-115, which will require a ramp and maybe some additional drainage facilities.

The current Aguadilla levee alternative consists of an earthen embankment with 1:2 side slopes and a top width of about 11.5 meters, as depicted in Plate 7, with an average levee height of 3.4 meters. A drainage channel will run along the exterior section which will feed eight flap-gate culvert openings for draining the exterior area of the levee.

5.2.2 Espinar Levee

The Espinar levee is more sinuous than the Aguadilla levee. It extends for a distance of about 2.15 kilometers, beginning at a high point south of the Espinar community and ending near the coastline at a point southwest of the present Caño Madre Vieja outlet. The Espinar community is located to the southwest of the levee. A coastal mangrove is located north of the Espinar community, which is drained by a small outlet channel into Caño Madre Vieja between station locations 00+00.00 and 02+00.00, as depicted in Plate 5.

The current Espinar levee alternative consists of a stepped embankment made up of a bulkhead wall and a retaining concrete wall along a promenade area, as depicted in Plate 8. The major opening in the wall is the outlet of the small drainage channel north of the Espinar community. A twin-barreled flap-gated culvert opening is proposed at this location.

5.3 Simulation Analysis and Results

The hydraulic analysis of the proposed levee system was performed with the Flo-2D model. It sought to determine the impact of the proposed development on the 100-year water surface profile from the regulatory FIS study for Río Culebrinas. As such, the main objective was to design a system that would not impact the regulatory profile upstream of the encroached segment of the floodplain wherein the levees will be constructed as well as to provide the maximum attainable level of flood mitigation for the interior areas.

5.3.1 Model Calibration

FIS flood map amendment procedures require a Duplicate Effective Model of the regulatory profile, against which a map revision request must be evaluated. To this end the hydraulic model was calibrated to reproduce, as closely as possible, the FIS flood elevations. The end result of the calibration is the matrix of roughness values at each cell of the finite-difference grid of the Culebrinas floodplain.

5.3.2 Water Surface Profiles

The regulatory flood profile is only affected by the levee system within the exterior area and does not extend upstream of Hwy-2. Likewise, the flood profile along Río Culebrinas north of Hwy-2 is also unaffected. Figure 2 depicts the comparison between the regulatory flood profile and the proposed profile along the Caño Madre Vieja sector, where it is evident that the relative impact of the levee system largely peters out within 2 kilometers from the outlet.

The resulting water surface elevations within the levees are listed in Table 6. The major computed difference between the FIS and proposed water surface profiles is in the order of 1.47 meters, at a distance of about 400 meters upstream from the coast. By the environs of Hwy-2 the difference in elevations is 0.01 meters, which is considered negligible and well within the expected error limits of the simulation.

5.3.3 Levee Profiles

Levee elevations were ascertained from the computed water surface profiles, wind wave, wave runup, wind setup, and freeboard specifications. Wave runup, wind setup, and wave heights were computed with several procedures available in Sentürk (Ref. 8). The results pertaining to this analysis are presented in Table 7. A wind speed similar to that suggested by the U.S. Army Corps (Ref. 7) was used to compute the wave parameters.

The resulting levee profiles are depicted in Figures 3 and 4, including the regulatory and proposed water surface elevations as well as the location of drainage structures across the levee embankments. Proposed longitudinal levee crest elevations are listed in Table 8. Average levee height for the Aguadilla levee is 3.42 meters while for the Espinar levee it is about 3.46 meters. Figure 5 shows the levee profiles proposed in the study compared to those from the Corps of Engineers design. Also included are the water surface profiles from the simulation.

To account for the possibility of levee overtopping, a minimum freeboard was provided at the lower end of the levees, which was then increased upstream in order to constrain this effect to the lower reach of the embankment.

VI. INTERIOR DRAINAGE ANALYSIS

The interior drainage areas are those lying outside the levee enclosure area, labeled herein as the Espinar and Aguadilla areas. A separate hydrologic study was performed for the interior drainage design while considering several control alternatives. The objective was the determination of the level of residual flooding expected as a result of the construction of the levees. While some level of residual flooding is acceptable, it was desired to mitigate as much as possible the level of flooding so as to provide additional benefits to the neighboring communities.

6.1 Methodology

The hydrologic analysis was performed using the same tools as those used for the Río Culebrinas basin study discussed earlier. Runoff hydrographs for the Espinar and Aguadilla areas were computed with the WMS model. In the case of the interior drainage areas, certain simplifying assumptions were required due to the nature of the terrain. Particularly for the Aguadilla sector, the area is heavily urbanized and it was not possible to model accurately the serpentine and complex flow paths existing within the built-up area. The system was simplified for representation in the model.

The hydraulic analyses of the drainage alternatives were performed with the Interconnected Pond Routing (ICPR) package (Ref. 9). An interior flood hydrology analysis requires the dynamic modeling of the system, in which the time-varying stages of the exterior and interior floods are applied concurrently on the interior drainage system in order to trace the extent of the resulting residual flooding. The exterior area was simulated as time-varying stage against which the interior drainage system would operate.

Given the differences in catchment area and wave travel times, it is extremely unlikely that the peak stages from the interior and exterior areas will ever coincide to produce a so-called "worst-case scenario". For example, the peak 100-year floods from both the interior and exterior will have a very small probability of coinciding. It is expected that the interior areas will respond faster than the river basin.

6.2 Description of Interior Drainage Areas

6.2.1 Aguadilla Interior Area

The Aguadilla interior area comprises the southern sector of the town and a portion of the hilly area east of Hwy-2. These hills, mostly karstic in nature, are characterized by numerous sinkholes and depressions. The drainage divides of the Aguadilla and Espinar interior areas are traced in Plate 9. Tracing drainage areas within a karst topography is highly uncertain due to the nature of this terrain, and thus a given trace is perforce only an estimate.

Runoff draining from the catchment will concentrate along the eastern face of the Aguadilla levee, in the undeveloped patch of land running alongside it. A field survey has identified three significant concentrated runoff avenues into this area. The first is a narrow street running uphill east of Hwy-2 via an underpass in front of the Aponte public housing project. Runoff flowing along this street is intercepted by a cross inlet across from the Aponte housing project and conveyed via a culvert to the undeveloped area west of it, near the northern tip of the Aguadilla levee.

The second is a culvert draining the Monte Brujo sector east of Hwy-2 which drains directly into Caño Madre Vieja along the boundary between the Urbanización Victoria and Urbanización García housing developments. This culvert courses beneath the access road (Juan Santos Street) to Hwy-2 at the Victoria sector, crossing the highway and extending for a short distance to the east of it.

The third is a small gully crossing an undeveloped patch of land in the Victoria sector, west of Hwy-2 and just north of the southern tip of the Aguadilla levee. The gully crosses Road PR-111 near the fork branching off to Road PR-115. It drains part of Hwy-2 and an area to the east of it.

Additional surface runoff from other adjacent area drains directly into the sector east of the Aguadilla levee. The Urbanización García project has no storm sewer facilities beyond street gutters which drain overland into the Caño Madre Vieja sector.

6.2.2 Espinar Interior Area

The Espinar interior area is smaller than that of Aguadilla. It comprises part of the Espinar residential community and the area where the marina and resort project is to be located. In this definition the flooded area along the Río Culebrinas' main course is excluded since it is not affected by the proposed project. The Espinar sector has no stormwater sewer facilities. Surface runoff flows overland into the Caño Madre Vieja area.

6.3 Data and Results

Because the Aguadilla and Espinar interior areas are so geographically close, the rainfall depths selected for the analysis will be the same. Rainfall depths for the selected return periods are listed in Table 9. The hydrologic parameters of each interior area are presented in Table 10. The CN parameter was computed in a similar fashion as that of the Río Culebrinas' basin. The results of the hydrologic analysis are listed in Table 11 as the peak flow for a given return period. As the results demonstrate, the Aguadilla area is the major runoff contributor to the levee interior flooding.

Determining the extent of residual flooding required choosing a particular combination of return periods for the interior and exterior stages. Usually, a 10 or 25-year return period for interior flooding is considered acceptable. However, in the interest of providing a more extreme scenario for the proposed project, a 100-year interior flood was selected, as well as the required 10-year flood, combined with a 50-year flood stage in the exterior area. The joint occurrence of 100-year stages is considered unlikely.

Several interior flood management alternatives were assessed as part of the hydrologic study. These included such activities as providing drainage outlets to the ocean and pumping stations. Some were technically infeasible, while others were considered too costly and impractical, as was the case with pumping stations. The most practical and cost-efficient alternative is that of providing gravity-outlets in the form of culverts for drainage into the levee exterior zone.

The resulting interior flooded areas area can be compared with the existing regulatory flooding extent in a revised flood map depicted in Plate 10. The levee system entails a significant reduction in the extent of flooding for the regulatory event. The quantitative description of the reduction in expected flood levels is explained below.

6.3.1 Aguadilla Levee

A total of eight 60-inch (1,524 mm) pipe culvert outlets are proposed for the Aguadilla levee. The location of these outlets was selected along the points at which runoff from the interior area naturally concentrates. The proposed locations for these outlets have already been indicated in Plate 6, along the Aguadilla levee alignment. Design parameters for each drainage pipe are listed in Table 12.

The residual flooding from the 100-year and 10-year rainfalls at the Aguadilla interior area was determined by simulating of the levee system against the time-varying flood stage of the levee interior area. The extent of the residual flooding along the Aguadilla levee is traced in Plate 11. As simulated in the hydraulic model, interior flooding is mainly the level-pool flooding related to the drainage outlets. The interior area has been treated as basically a storage area for the culverts. The peak stage for the 100-year interior area flooding reached only 2.94 meters, while that of the 10-year flood stood at 2.77 meters.

Numbered locations in Plate 11 refer to water elevation and depth data that are listed in order to provide a quantitative measure of the degree of flood relief resulting from the proposed levee system. Table 13 lists the water surface elevations at the locations indicated in Figure 16, including the difference in elevations resulting from the construction of the levee. A significant reduction in water elevations is obtained at most of the locations, particularly within the residential areas. At a small number of non-residential locations near the coast, a slight increase not exceeding 0.14 meters (5.5 inches) is obtained. These slight differences may well fall within the expected error tolerances of the simulation.

A lateral profile of the interior flood at a location in the Aguadilla levee (see Plate 11 for the location) is presented in Figure 6. The dramatic reduction in the extent and depth of flooding is evident in the profile.

6.3.2 Espinar Levee

Two 60-inch culverts are proposed for the Espinar interior drainage area, as located in Plate 6. Design parameters for these culverts are presented in Table 14.

As with the Aguadilla levee, the residual flooding from the 100-year and 10-year rainfalls at the Espinar interior area were determined from the hydraulic simulation of the levee system against the time-varying flood stage of the levee interior area. The extent of the residual flooding along the Espinar levee is traced in Plate 12. The peak stage for the 100-year interior area flooding reached only 2.37 meters, while that of the 10-year flood stood at 2.14 meters.

Numbered locations in Plate 12 refer to the elevations listed in Table 15. Significant beneficiary differences in water surface elevations are achieved in the Espinar area as a result of the construction of the levee. As such, the levee will reduce flood proofing requirements in this sector. A lateral flood profile along the Espinar interior area (located in Plate 12) is depicted in Figure 7, showing the significant reduction in the extent of flooding achieved by the levee system.

6.3.3 Flooding at the Tablonal Area

The other significant area flooded by Río Culebrinas is the Tablonal sector, located south of the river and west of Road PR-115. A sizable portion of the community thereon is highly susceptible to frequent flooding due to its proximity to the river. This area is only slightly affected by the marina project, as evidenced by the comparison between water surface elevations from the FIS and the present study at several locations. Plate 13 identifies the locations where the flood elevations are compared, while Table 16 lists the numerical comparisons. At only one location is a difference observed that would slightly exceed 6 inches, amounting to less than a 13 % change in flood depth.

It is worth noting that the water surface elevations proposed in the U.S. Army Corps levee study (Ref. 7) show an average increase of about 9.8 inches within the Tablonal sector, albeit with a different simulation approach and conceptualization. As such, the proposed marina project provides the benefit of a lower impact.

Due to its proximity to the main course of Río Culebrinas, the Tablonal sector would benefit from the implementation of some flood-proofing activity for those housing units likely to be affected by flooding.

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TABLES

Table 1. Major Recorded Annual Floods at Río Culebrinas

Date	Flow (ft^3/s)
September 16, 1975	41,200
September 22, 1998	36,900
May 17, 2003	31,800
September 26, 2004	33,100
November 17, 1968	30,00
October 4, 1993	28,400

Table 2. Río Culebrinas' Sub-Basin Areas and Curve Numbers

		Curve Number	
Sub-Basin	Drainage Area, mi^2	AMC-II	AMC-III
A	12.63	79.3	93.6
B	19.39	75.3	91.2
C	4.75	72.6	89.1
D	11.83	72.6	89.1
E	2.56	74.3	90.4
F	5.45	72.7	89.2
G	4.48	73.5	89.8
H	1.21	73.9	90.1
I	6.58	72.1	88.7
J	12.44	71.4	88.1
K	2.62	78.4	93.0
L	6.85	75.0	91.0
M	3.51	79.7	93.8
N	8.34	80.2	94.1

Table 3. Río Culebrinas' Sub-Basin Lag Times

Sub-Basin	Lag Times, hrs	
	AMC-II	AMC-III
A	1.61	0.95
B	2.19	1.27
C	1.34	0.79
D	2.18	1.28
E	1.05	0.61
F	1.18	0.69
G	1.22	0.71
H	0.77	0.45
I	1.36	0.80
J	1.47	0.86
K	1.06	0.62
L	1.34	0.78
M	1.18	0.69
N	1.22	0.72

Table 4. 24-hr Rainfall (inches) for Río Culebrinas Sub-Basins

	Percent Chance Storm						
Sub-Basin	.02	1	2	4	10	20	50
A	12.5	9.00	8.00	7.50	7.00	6.00	4.50
B	12.5	9.50	9.00	8.10	7.25	6.20	4.63
C	12.5	9.00	8.00	7.50	7.00	6.00	4.50
D	12.5	9.00	8.25	7.75	7.00	6.00	4.50
E	12.5	9.00	8.00	7.50	7.00	6.00	4.50
F	12.5	9.00	8.10	7.50	7.00	6.00	4.50
G	12.5	9.00	8.00	7.50	7.00	6.00	4.50
H	12.5	9.00	8.00	7.50	7.00	6.00	4.50
I	12.5	9.00	8.10	7.50	7.00	6.00	4.50
J	12.5	9.00	8.00	7.50	7.00	6.00	4.50
K	12.5	9.00	8.10	7.50	7.00	6.00	4.50
L	12.5	9.00	8.00	7.50	7.00	6.00	4.50
M	12.5	9.25	8.25	7.50	7.00	6.00	4.50
N	12.5	9.00	8.00	7.50	7.00	6.00	4.50

Table 5. Comparison of Río Culebrinas' Peak Flows at Highway-2 Bridge

Return Period (years)	Peak Flows (m^3/s)	
	Present Study*	FEMA FIS
500	5743	5759
100	4062	4063
50	3198	3200
25	1757	Not computed
10	1639	1642
5	1286	Not computed
2	784	Not computed

* - The 500, 100, and 50-year floods are for AMC-III conditions. The others are for AMC-II conditions.

Table 6. Caño Madre Vieja Resulting Water Surface Elevations for Existing and Project Conditions for 100-year Flood

REACH	BRIDGE	PARTIAL DISTANCE, METERS	STATION, METERS	PRE-PROJECT WATER SURFACE ELEVATION, METERS	POST-PROJECT WATER SURFACE ELEVATION, METERS	DIFFERENCE IN WATER SURFACE ELEVATION, METERS
Reach : Caño Madre Vieja						
		63.65	26+63.65	8.78	8.79	0.01
		100.00	26+00.00	8.78	8.79	0.01
		100.00	25+00.00	8.66	8.67	0.01
	PR-2	100.00	24+00.00	8.66	8.67	0.01
		100.00	23+00.00	7.97	7.98	0.01
		100.00	22+00.00	7.43	7.48	0.05
		100.00	21+00.00	7.12	7.20	0.08
		100.00	20+00.00	6.90	7.00	0.10
		100.00	19+00.00	6.79	6.90	0.11
	PR-418	100.00	18+00.00	6.47	6.65	0.18
		100.00	17+00.00	6.47	6.65	0.18
		100.00	16+00.00	6.17	6.44	0.27
		100.00	15+00.00	5.92	6.27	0.35
		100.00	14+00.00	5.92	6.27	0.35
	PR-115	100.00	13+00.00	5.60	6.19	0.59
		100.00	12+00.00	5.25	6.06	0.81
		100.00	11+00.00	5.25	6.06	0.81
		100.00	10+00.00	5.13	6.00	0.87
		100.00	09+00.00	5.13	6.00	0.87
		100.00	08+00.00	4.74	5.63	0.89
		100.00	07+00.00	4.74	5.63	0.89
		100.00	06+00.00	4.38	5.15	0.77
		100.00	05+00.00	4.38	5.15	0.77
		100.00	04+00.00	3.12	4.59	1.47
		100.00	03+00.00	3.12	4.59	1.47
		100.00	02+00.00	2.73	3.55	0.82
		100.00	01+00.00	2.41	2.86	0.45
		0.00	00+00.00	2.41	2.86	0.45

Table 7. Río Culebrinas Wave Runup and Wind Setup

LEVEE SEGMENT	LEVEE STATION	FETCH LENGTH (M)	WIND SPEED KM/HR	WAVE HEIGHT (M)	WAVE RUNUP (M)	WIND SETUP (M)	TOTAL (M)
AGUADILLA	6+67	502	48	0.25	0.30	0.01	0.55
ESPINAR	4+09	645	48	0.25	0.30	0.01	0.55

Table 8. Rio Culebrinas 100-year Levee Crest Elevations

LEVEE SEGMENT	ROAD RAMP	PARTIAL DISTANCE (M)	DISTANCE	LEVEE STATION (M)	WAVE RUNUP AND WIND SETUP FOR 48 KM/HOUR WIND (M)	TOTAL MINIMUM FREEBOARD (M)	LEVEE CREST ELEVATION (M)	AVERAGE GROUND ELEVATION (M)	LEVEE HEIGHT (M)	POST-PROJECT WATER SURFACE ELEVATION (M)	PRE-PROJECT WATER SURFACE ELEVATION (M)
AGUADILLA											
		19.59	1762.88	17+62.88	0.88	1.07	8.07	6.91	1.16	7.00	6.32
		80.41	1743.29	17+43.29	0.88	1.07	8.05	3.66	4.39	6.98	6.30
		100.00	1662.88	16+62.88	0.88	1.06	7.95	3.49	4.46	6.89	6.27
	PR-418	100.00	1562.88	15+62.88	0.88	1.22	7.92	2.79	5.13	6.70	6.19
		100.00	1462.88	14+62.88	0.88	1.22	7.74	3.00	4.74	6.52	6.07
	PR-115	100.00	1362.88	13+62.88	0.88	1.03	7.40	3.80	3.60	6.37	5.88
		100.00	1262.88	12+62.88	0.88	1.02	7.22	2.50	4.72	6.20	5.63
		100.00	1162.88	11+62.88	0.88	1.01	7.05	2.80	4.25	6.04	5.37
		100.00	1062.88	10+62.88	0.88	1.01	6.75	2.50	4.25	5.74	5.12
		100.00	962.88	09+62.88	0.88	1.00	6.40	2.50	3.90	5.40	4.86
		100.00	862.88	08+62.88	0.88	0.99	6.08	2.50	3.58	5.09	4.61
		100.00	762.88	07+62.88	0.88	0.98	5.77	0.50	5.27	4.79	4.37
		100.00	662.88	06+62.88	0.88	0.97	5.47	2.20	3.27	4.50	4.15
		100.00	562.88	05+62.88	0.88	0.96	5.16	0.50	4.66	4.20	3.91
		100.00	462.88	04+62.88	0.88	0.95	4.85	2.00	2.85	3.90	3.66
		100.00	362.88	03+62.88	0.88	0.95	4.56	2.00	2.56	3.61	3.41
		100.00	262.88	02+62.88	0.88	0.94	4.27	1.60	2.67	3.33	3.17
		100.00	162.88	01+62.88	0.88	0.93	4.06	1.70	2.36	3.13	2.91
		53.22	62.88	00+62.88	0.88	0.92	3.92	1.40	2.52	3.00	2.80
		9.66	9.66	00+09.66	0.88	0.92	3.92	2.50	1.42	3.00	2.80
		0.00	0.00	00+00.00	0.88	0.91	3.30	3.30	0.00	3.00	2.80
								Avg. Height =	3.42		
ESPINAR											
		94.0	2194.0	21+93.99	0.88	1.07	5.47	4.50	0.97	4.40	4.44
		100.0	2100.0	21+00.00	0.88	1.06	5.50	3.00	2.50	4.44	4.46
		100.0	2000.0	20+00.00	0.88	1.05	5.70	3.00	2.70	4.65	4.65
		100.0	1900.0	19+00.00	0.88	1.05	6.18	3.00	3.18	5.13	5.09
		100.0	1800.0	18+00.00	0.88	1.04	6.54	3.00	3.54	5.50	5.46
		100.0	1700.0	17+00.00	0.88	1.03	7.03	3.50	3.53	6.00	5.88
		100.0	1600.0	16+00.00	0.88	1.22	7.62	2.90	4.72	6.40	6.10
	PR-442	100.0	1500.0	15+00.00	0.88	1.22	7.59	4.50	3.09	6.37	6.12
		100.0	1400.0	14+00.00	0.88	1.22	7.54	3.50	4.04	6.32	6.11
		100.0	1300.0	13+00.00	0.88	1.22	7.49	3.90	3.59	6.27	6.03
		100.0	1200.0	12+00.00	0.88	1.00	7.11	4.00	3.11	6.11	5.77
		100.0	1100.0	11+00.00	0.88	0.99	6.92	2.50	4.42	5.93	5.44
		100.0	1000.0	10+00.00	0.88	0.98	6.67	2.30	4.37	5.69	5.19
		100.0	900.0	09+00.00	0.88	0.98	6.36	1.50	4.86	5.38	4.92
		100.0	800.0	08+00.00	0.88	0.97	6.07	1.50	4.57	5.1	4.69
		100.0	700.0	07+00.00	0.88	0.96	5.83	1.50	4.33	4.87	4.42
		100.0	600.0	06+00.00	0.88	0.96	5.56	1.00	4.56	4.6	4.08
		100.0	500.0	05+00.00	0.88	0.95	5.26	1.30	3.96	4.31	3.78
		100.0	400.0	04+00.00	0.88	0.94	5.00	1.50	3.50	4.06	3.53
		100.0	300.0	03+00.00	0.88	0.94	4.71	1.50	3.21	3.77	3.23
		100.0	200.0	02+00.00	0.88	0.93	4.46	1.50	2.96	3.53	2.96
		50.0	100.0	01+00.00	0.88	0.92	4.04	2.00	2.04	3.12	2.80
		50.0	50.0	00+50.00	0.88	0.92	3.92	1.50	2.42	3.00	2.80
		0.0	0.0	00+00.00	0.88	0.91	3.91	1.00	2.91	3.00	2.80
								Avg. Height =	3.46		

Table 9. 24-hr Rainfall (in.) for Discovery Bay IFH Watershed Study Area

Return Periods	Precipitation, in.
100-year	9.5
50-year	8.5
25-year	7.5
10-year	7.0
5-year	6.0
2-year	4.5
1-year	3.5

Table 10. Discovery Bay Watershed Hydrologic Model Simulation Parameters for Study Area

Sub-Basin	Area, mi^2	Lag times, hrs	CN
Aguadilla Levee	1.14	0.60	79.5
Espinar Levee	0.38	0.65	83.6

Table 11. Estimated Flood Flows for Discovery Bay IFH Watershed Study Area

Return Period, years	Peak Flows, m^3/s	
	Aguadilla	Espinar
100	74.9	25.1
50	65.0	22.0
25	55.1	18.8
10	50.2	17.3
5	40.4	14.1
2	26.0	9.5
1	17.0	6.4

Table 12. Discovery Bay Interior Drainage Structures Hydraulic Design Data Aguadilla Levee

ID	Station	Diameter (in.)	Length (m)	Slope	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Type of Control
A1	1+06.39	60	21.330	0.02344	0.500	0.000	FLAPGATE
A2	4+42.74	60	21.330	0.02344	0.500	0.000	FLAPGATE
A3	5+58.53	60	21.330	0.02344	0.500	0.000	FLAPGATE
A4	8+82.81	60	21.330	0.02344	2.500	2.000	FLAPGATE
A5	11+33.01	60	21.330	0.02344	2.000	1.500	FLAPGATE
A6	12+90.61	60	21.330	0.02344	2.000	1.500	FLAPGATE
A7	15+94.27	60	21.330	0.01360	2.790	2.500	FLAPGATE
A8	17+00.13	60	21.330	0.04219	3.500	2.600	FLAPGATE

Table 13. Aguadilla 10-yr and 100-yr Interior Flood Water Surface Elevations

ID	GROUND ELEVATION (M)	FEMA EFFECTIVE BFE (M)	PROPOSED WSE 100YR (M)	PROPOSED WSE 10YR (M)	100-YEAR DIFFERENCE (M)
1	2.64				
2	2.24	2.80	2.94	2.77	0.14
3	1.98	2.80	2.94	2.77	0.14
4	1.54	2.97	2.94	2.77	-0.03
5	2.19	3.10	2.94	2.77	-0.16
6	2.22	3.26	2.94	2.77	-0.32
7	2.70	2.80	2.94	2.77	0.14
8	2.00	2.89	2.94	2.77	0.05
9	2.30	2.80	2.94	2.77	0.14
10	1.50	2.80	2.94	2.77	0.14
11	1.40	3.03	2.94	2.77	-0.09
12	2.30	3.02	2.94	2.77	-0.08
13	2.10	3.06	2.94	2.77	-0.12
14	2.89	3.30	2.94	2.77	-0.36
15	2.64	3.41	2.94	2.77	-0.47
16	2.60	3.12	2.94	2.77	-0.18
17	1.80	3.34	2.94	2.77	-0.40
18	2.20	3.77	2.94	2.77	-0.83
19	2.30	4.01	2.94	2.77	-1.07
20	2.10	3.68	2.94	2.77	-0.74
21	2.10	4.01	2.94	2.77	-1.07
22	2.70	4.03	2.94	2.77	-1.09
23	2.80	4.25	2.94	2.77	-1.31
24	1.70	4.24	2.94	2.77	-1.30
25	2.60	4.49	2.94	2.77	-1.55
26	2.40	4.54	2.94	2.77	-1.60
27	2.10	4.63	2.94	2.77	-1.69
28	2.70	4.84	2.94	2.77	-1.90
29	2.30	4.98	2.94	2.77	-2.04
30	2.40	5.18	2.94	2.77	-2.24

Table 14. Discovery Bay Interior Drainage Structures Hydraulic Design Data Espinar Levee

ID	Station	Diameter (in.)	Length (m)	Slope (m/m)	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Type of Control
E1	1+14.09	60	15.240	0.09843	1.500	0.000	FLAPGATE
E2	1+14.09	60	15.240	0.09843	1.500	0.000	FLAPGATE

Table 15. Espinar Community 10-yr and 100-yr Interior Flood Water Surface Elevations

ID	GROUND ELEVATION (M)	FEMA EFFECTIVE BFE (M)	PROPOSED WSE 100YR (M)	PROPOSED WSE 10YR (M)	DIFFERENCE (M)
1	2.60	3.20	2.37	2.14	-0.83
2	1.60	3.24	2.37	2.14	-0.87
3	1.30	2.97	2.37	2.14	-0.60
4	1.70	3.20	2.37	2.14	-0.83
5	1.10	2.97	2.37	2.14	-0.60
6	1.30	3.28	2.37	2.14	-0.91
7	2.70	3.36	2.37	2.14	-0.99
8	2.20	3.80	2.37	2.14	-1.43
9	2.30	3.93	2.37	2.14	-1.56
10	2.10	4.15	2.37	2.14	-1.78
11	2.50	4.15	2.37	2.14	-1.78
12	1.60	2.54	2.37	2.14	-0.17
13	1.00	2.95	2.37	2.14	-0.58
14	1.90	3.20	2.37	2.14	-0.83
15	2.30	3.60	2.37	2.14	-1.23
16	2.60	3.73	2.37	2.14	-1.36
17	2.20	4.06	2.37	2.14	-1.69
18	2.30	4.19	2.37	2.14	-1.82
19	1.70	4.24	2.37	2.14	-1.87
20	1.50	4.41	2.37	2.14	-2.04
21	2.50	4.58	2.37	2.14	-2.21
22	2.60	4.60	2.37	2.14	-2.23

Table 16. Tablonal Community 100-year Flood Depths

ID	GROUND ELEVATION (M)	EXISTING BFE (M)	PROPOSED WSE 100YR (M)	DIFFERENCE (M)	DIFFERENCE (INCHES)	WATER DEPTH (M)
1	2.51	3.56	3.65	0.09	3.5	1.14
2	2.05	3.65	3.76	0.11	4.3	1.71
3	2.75	3.84	3.94	0.10	3.9	1.19
4	2.44	3.90	4.05	0.15	5.9	1.61
5	2.93	4.07	4.16	0.09	3.5	1.23
6	2.42	4.11	4.23	0.12	4.7	1.81
7	2.62	4.17	4.28	0.11	4.3	1.66
8	2.71	4.26	4.36	0.10	3.9	1.65
9	3.02	4.30	4.39	0.09	3.5	1.37
10	3.29	4.63	4.80	0.17	6.7	1.51
11	4.27	5.67	5.81	0.14	5.5	1.54
12	5.60	6.11	6.22	0.11	4.3	0.62

IX. FIGURES

Figure 1. Aerial View of Project Area

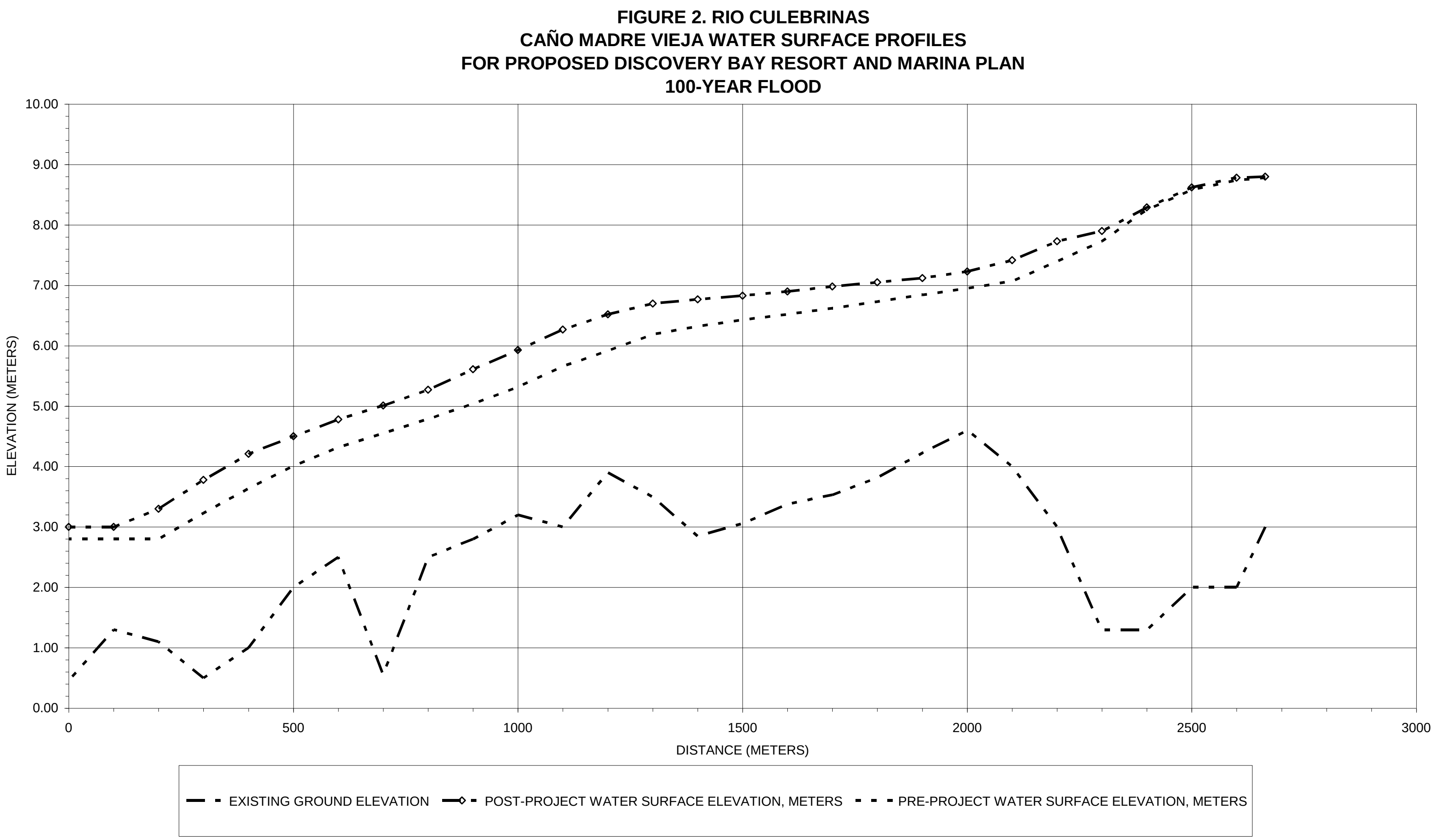


FIGURE 2. RIO CULEBRINAS
CAÑO MADRE VIEJA WATER SURFACE PROFILES
FOR PROPOSED DISCOVERY BAY RESORT AND MARINA PLAN
100-YEAR FLOOD
ELEVATION (METERS)
10.00
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
0
500
1000
1500
2000
2500
3000
DISTANCE (METERS)
EXISTING GROUND ELEVATION
POST-PROJECT WATER SURFACE ELEVATION, METERS
PRE-PROJECT WATER SURFACE ELEVATION, METERS

FIGURE 3. RIO CULEBRINAS
AGUADILLA LEVEE PROFILES
FOR PROPOSED DISCOVERY BAY RESORT AND MARINA PLAN
100-YEAR FLOOD

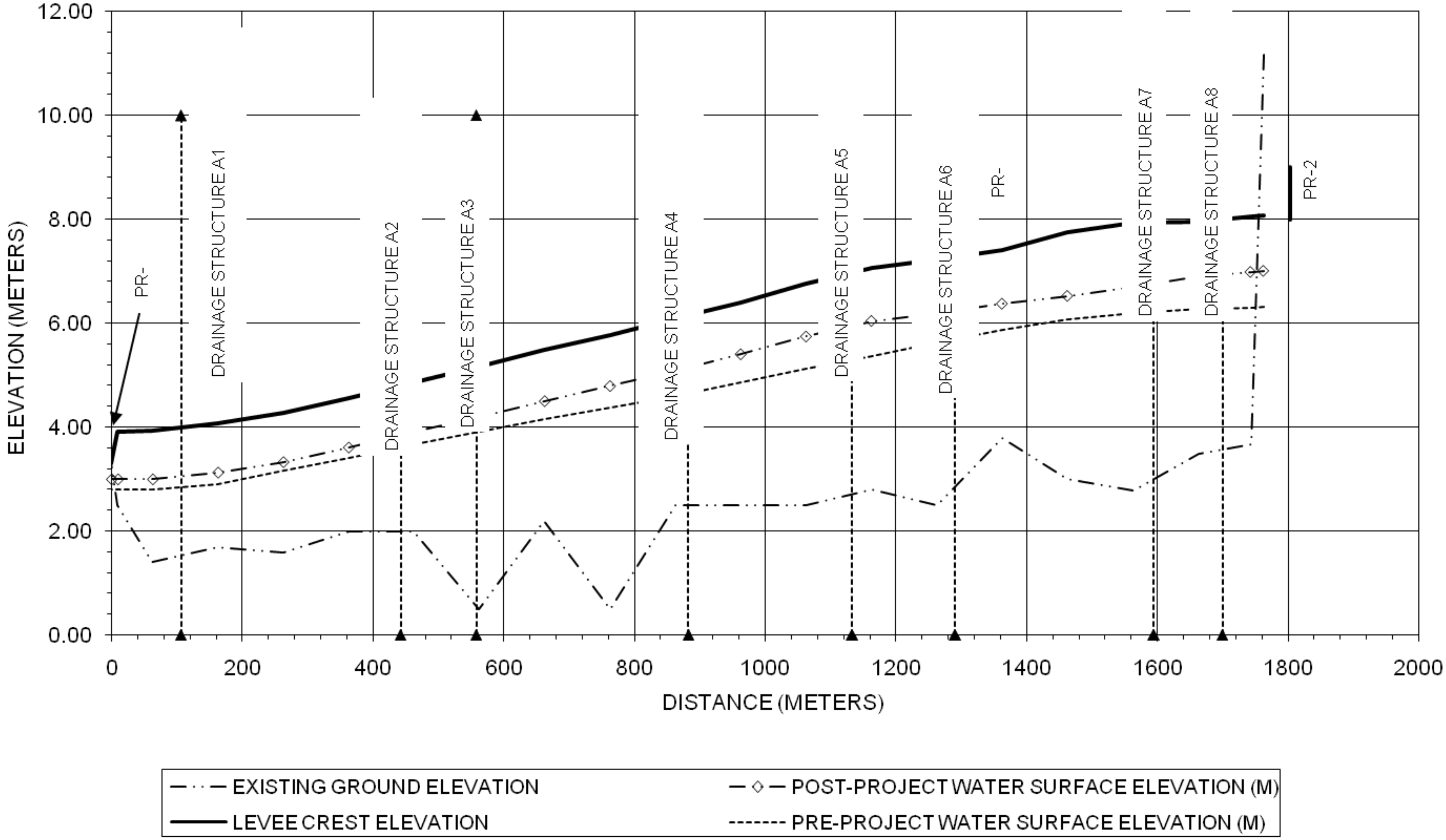


FIGURE 4. RIO CULEBRINAS
ESPINAR LEVEE PROFILES
FOR PROPOSED DISCOVERY BAY RESORT AND MARINA PLAN
100-YEAR FLOOD

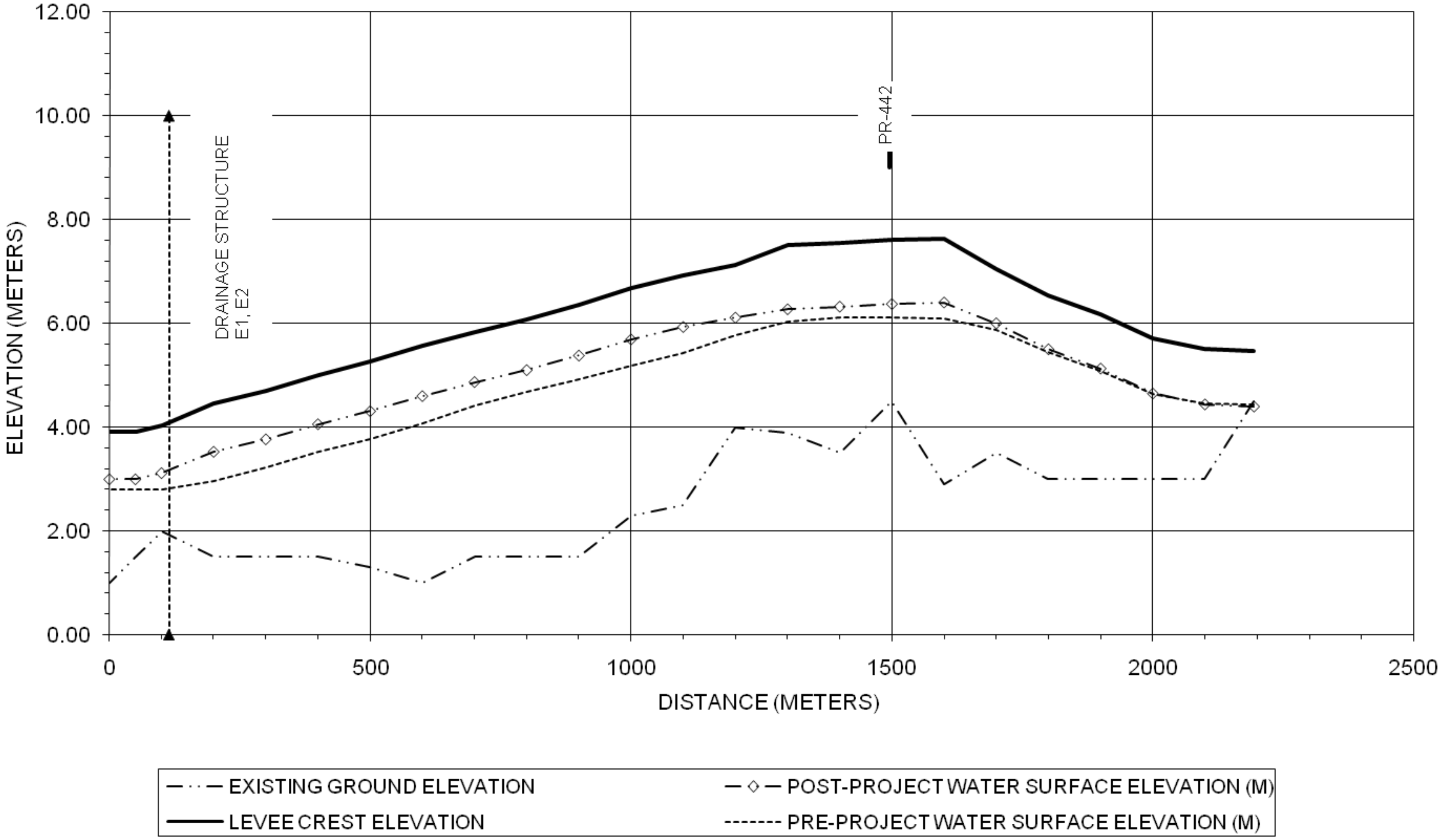
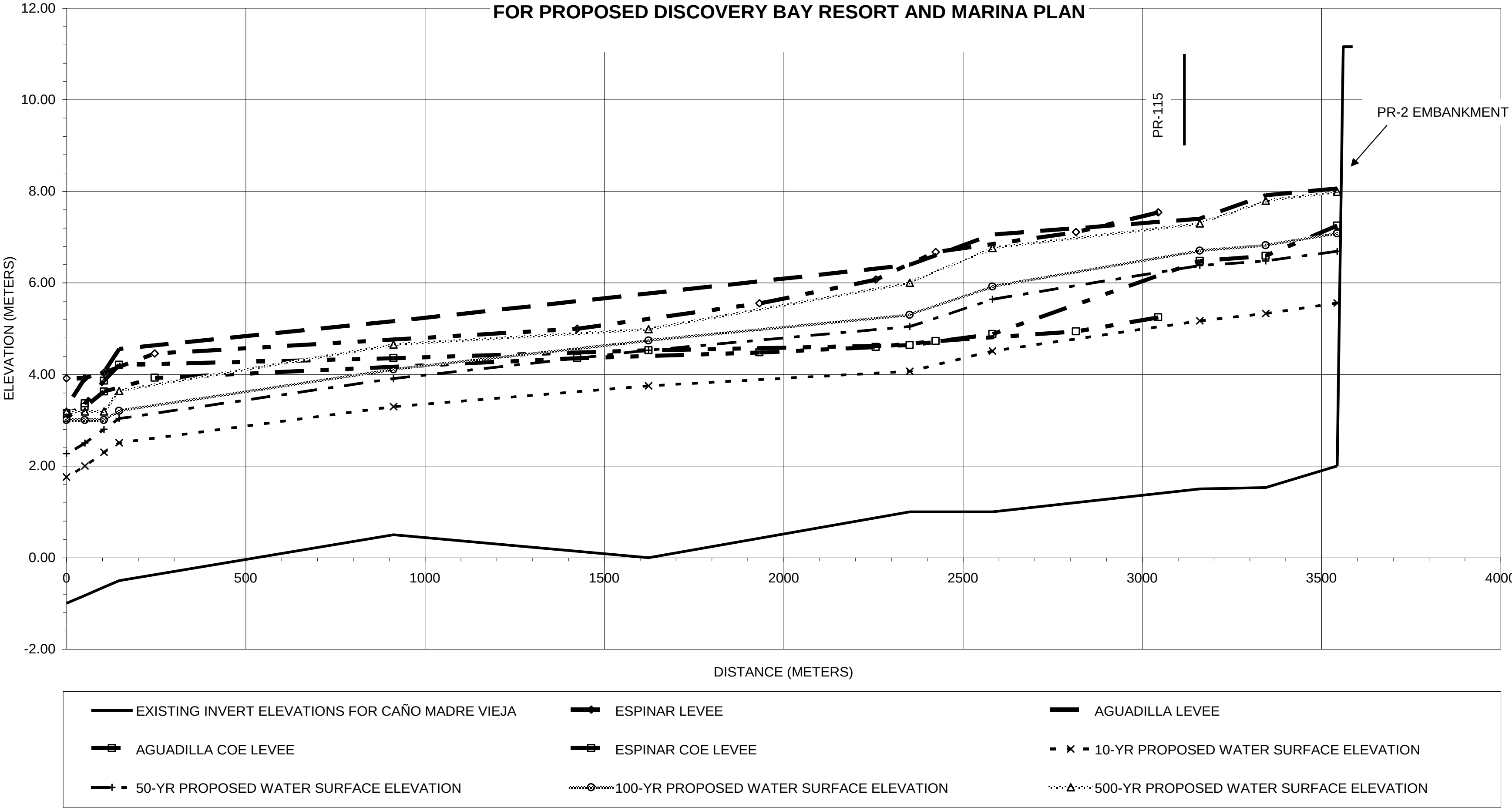


FIGURE 5. RIO CULEBRINAS
CAÑO MADRE VIEJA LEVEES PROFILE
FOR PROPOSED DISCOVERY BAY RESORT AND MARINA PLAN



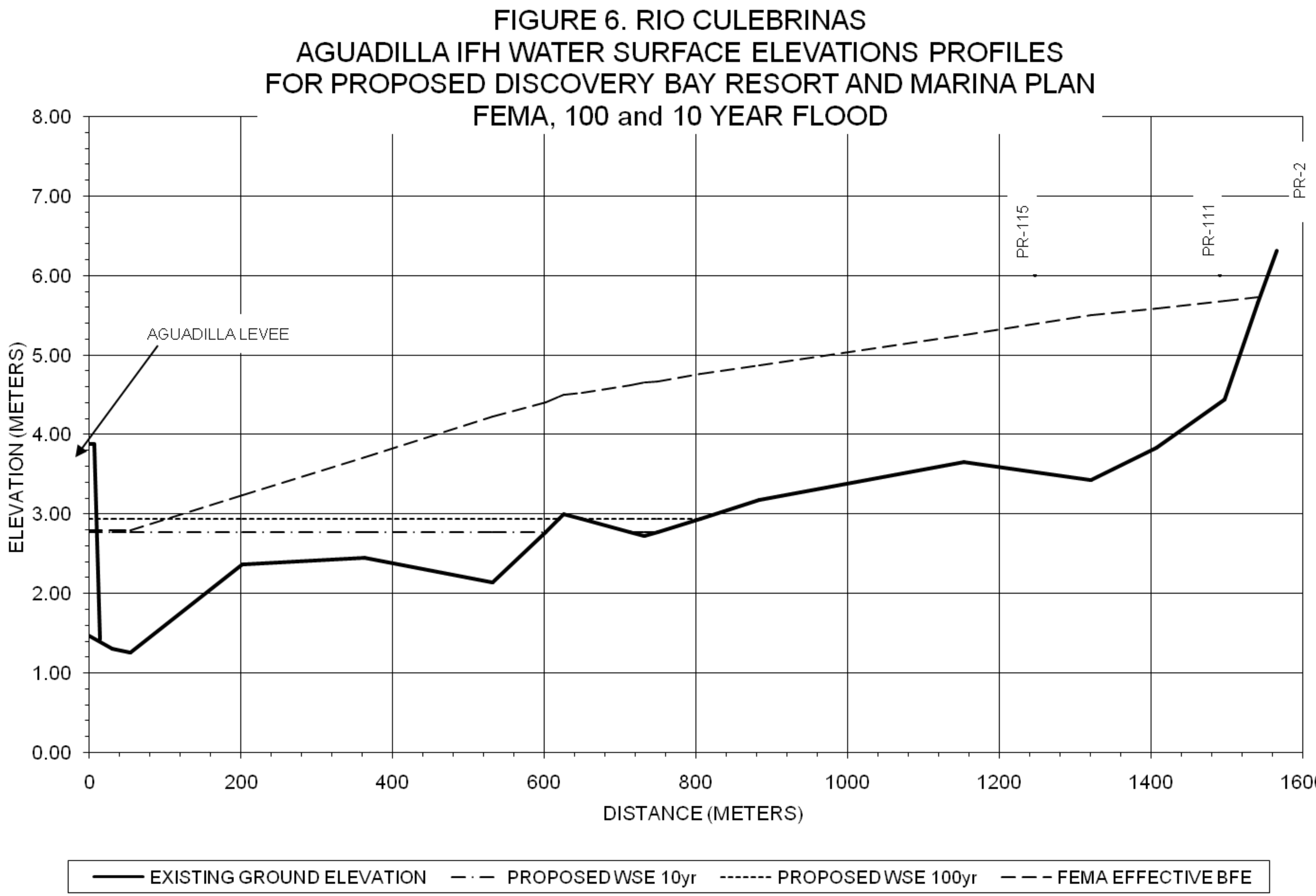


FIGURE 6. RIO CULEBRINAS
AGUADILLA IFH WATER SURFACE ELEVATIONS PROFILES
FOR PROPOSED DISCOVERY BAY RESORT AND MARINA PLAN
FEMA, 100 and 10 YEAR FLOOD
ELEVATION (METERS)
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
0
200
400
600
800
1000
1200
1400
1600
DISTANCE (METERS)
AGUADILLA LEVEE
PR-115
PR-111
PR-2
EXISTING GROUND ELEVATION
PROPOSED WSE 10yr
PROPOSED WSE 100yr
FEMA EFFECTIVE BFE

FIGURE7. RIO CULEBRINAS
ESPINAR IFH WATER SURFACE ELEVATIONS PROFILES
FOR PROPOSED DISCOVERY BAY RESORT AND MARINA PLAN
FEMA, 100 and 10 YEAR FLOOD

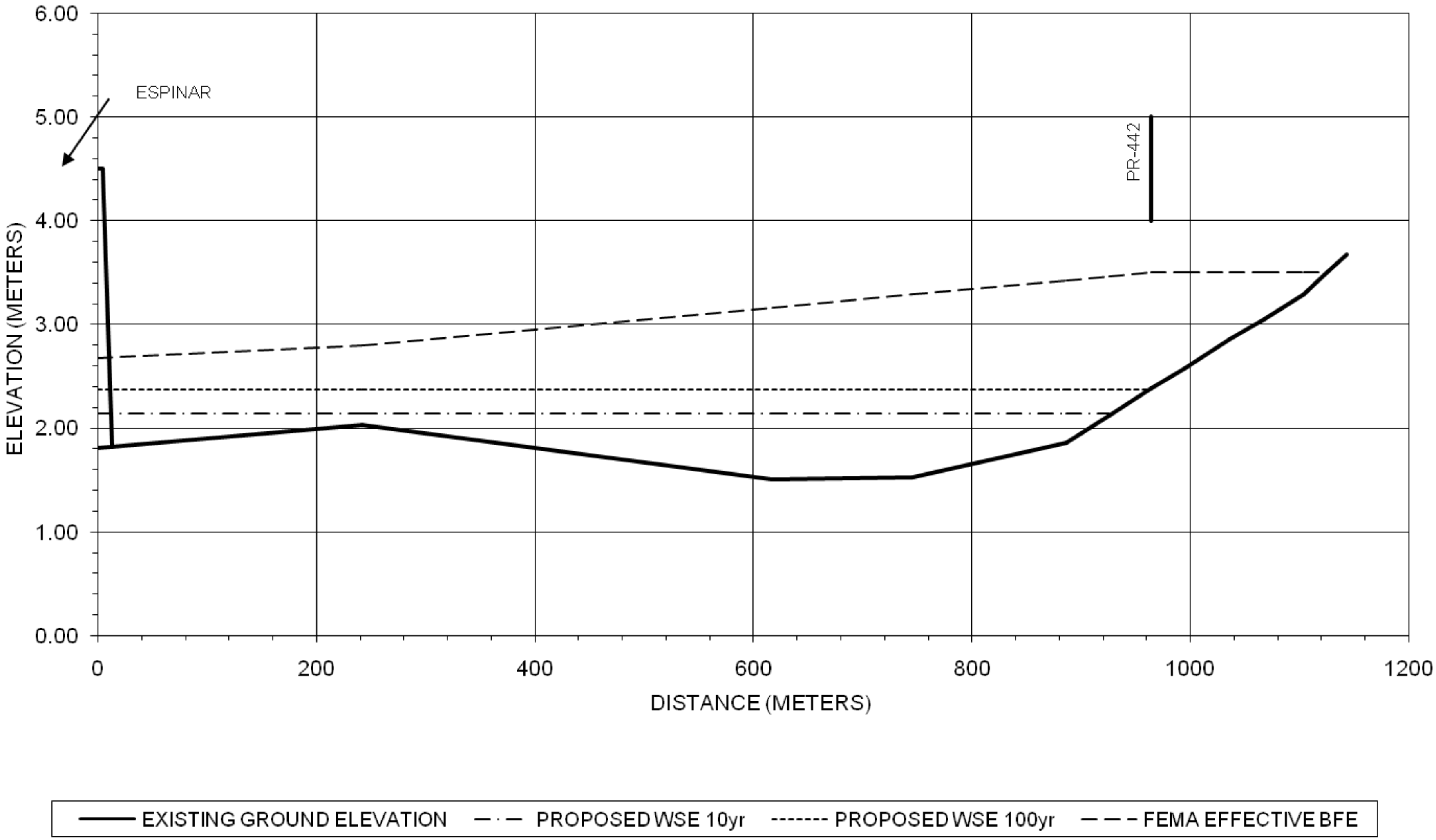


PLATE 1. FEMA FLOOD MAP
EXISTING CONDITIONS
DISCOVERY BAY RESORT & MARINA
AGUADA, PUERTO RICO

N
W E
S

LEGEND

BASE FLOOD ELEVATION

FLOOD AREAS
AE
X

FLOOD AREA
FLOODWAY

SCALE 1:20,000

0.5 0 0.5 1 1.5 Kilometers

0.2 0 0.2 0.4 0.6 0.8 1 Miles

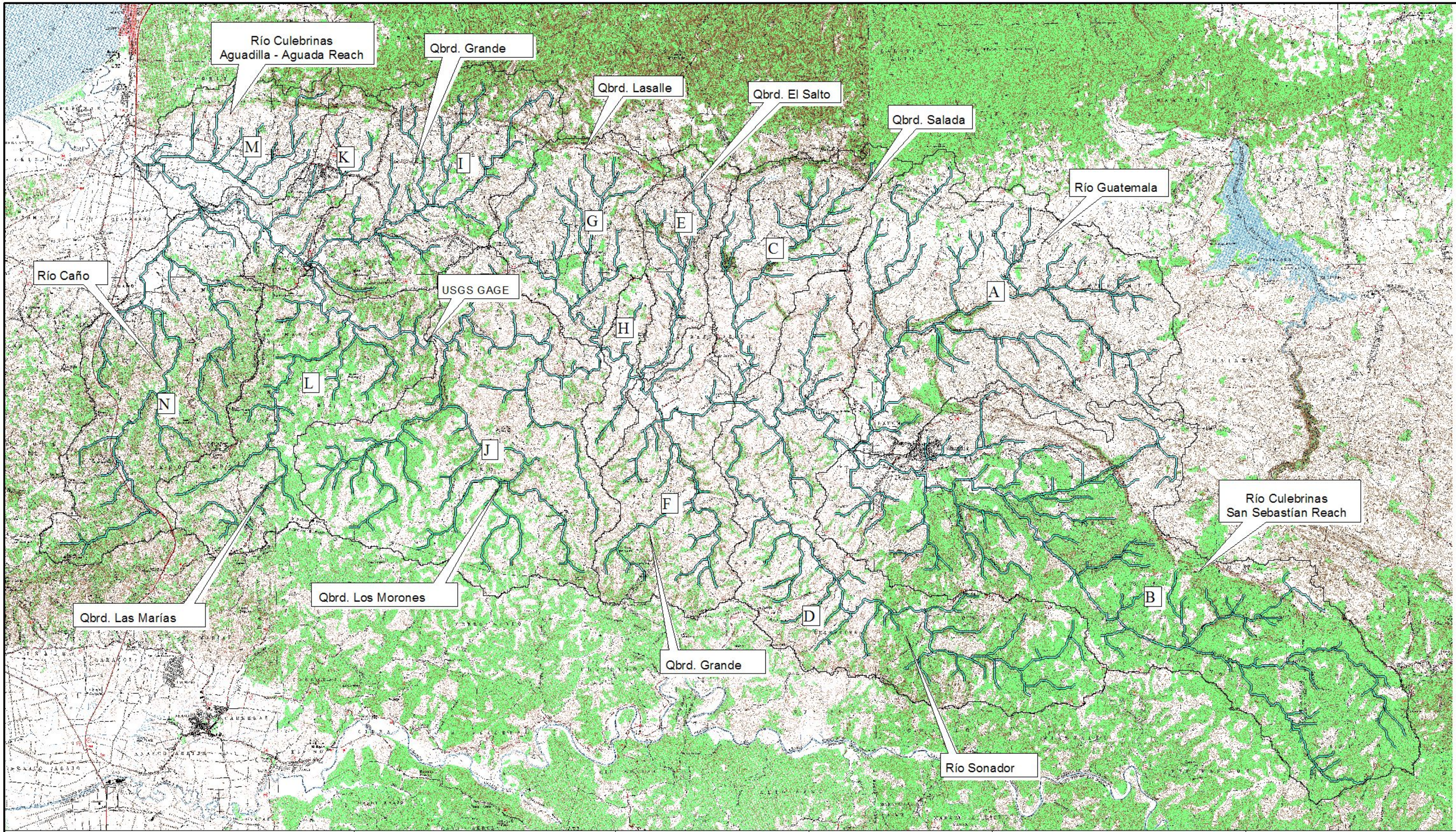


PLATE 2. DISCOVERY BAY RESORT AND MARINA
AGUADA, PUERTO RICO
RIO CULEBRINAS BASIN AND SUB-BASINS
BOUNDARIES AND RIVER NETWORK
WITH LOCATION MAP

N

LEGEND

Watershed Boundaries

River Network

SCALE 1:80,000

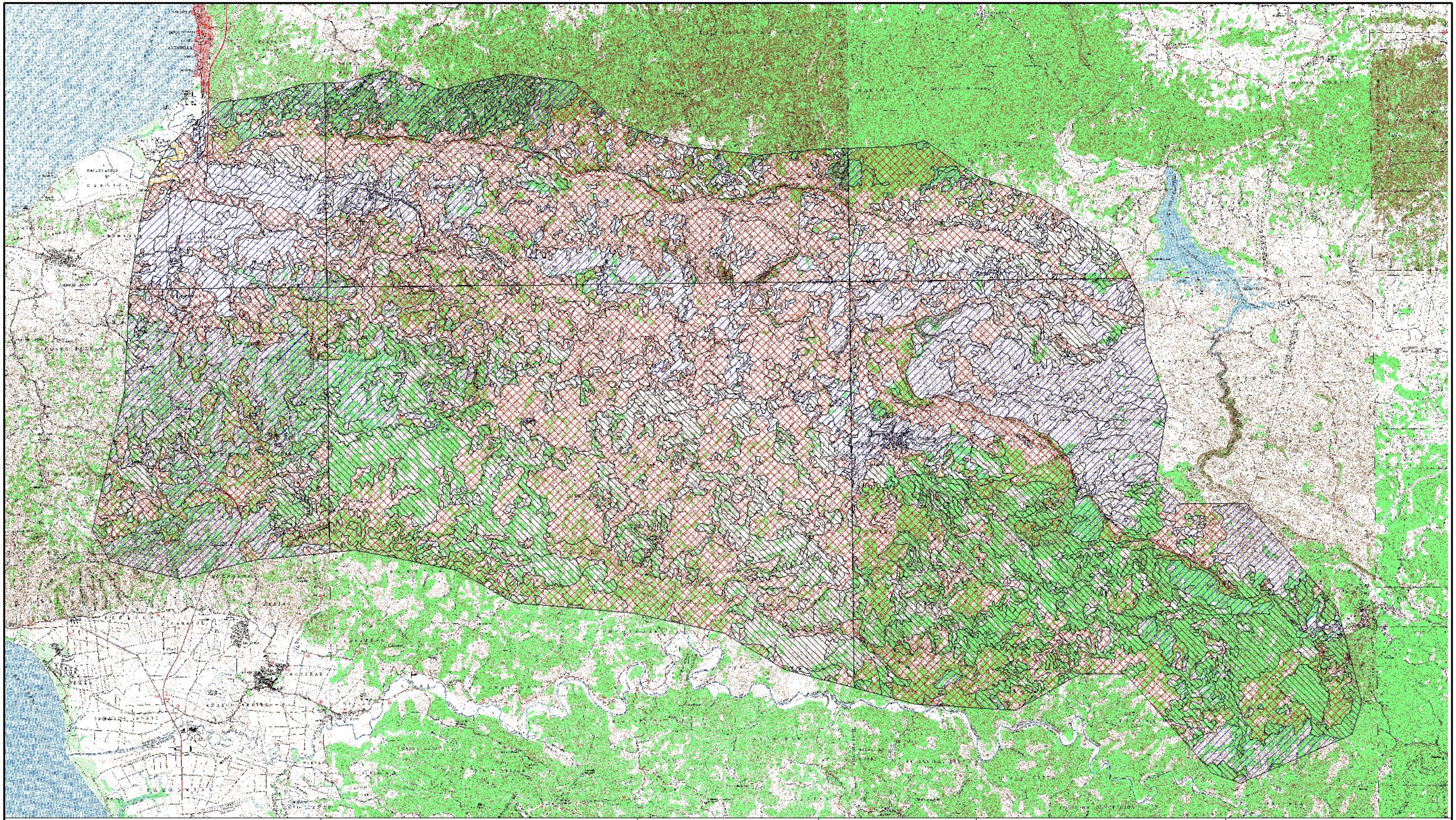


PLATE 3. DISCOVERY BAY RESORT AND MARINA
AGUADA, PUERTO RICO
RIO CULEBRINAS WATERSHED STUDY AREA
SOIL ASSOCIATION WITH LOCATION MAP

N

LEGEND

Soil Type

A
B
C
D

SCALE 1:90,000

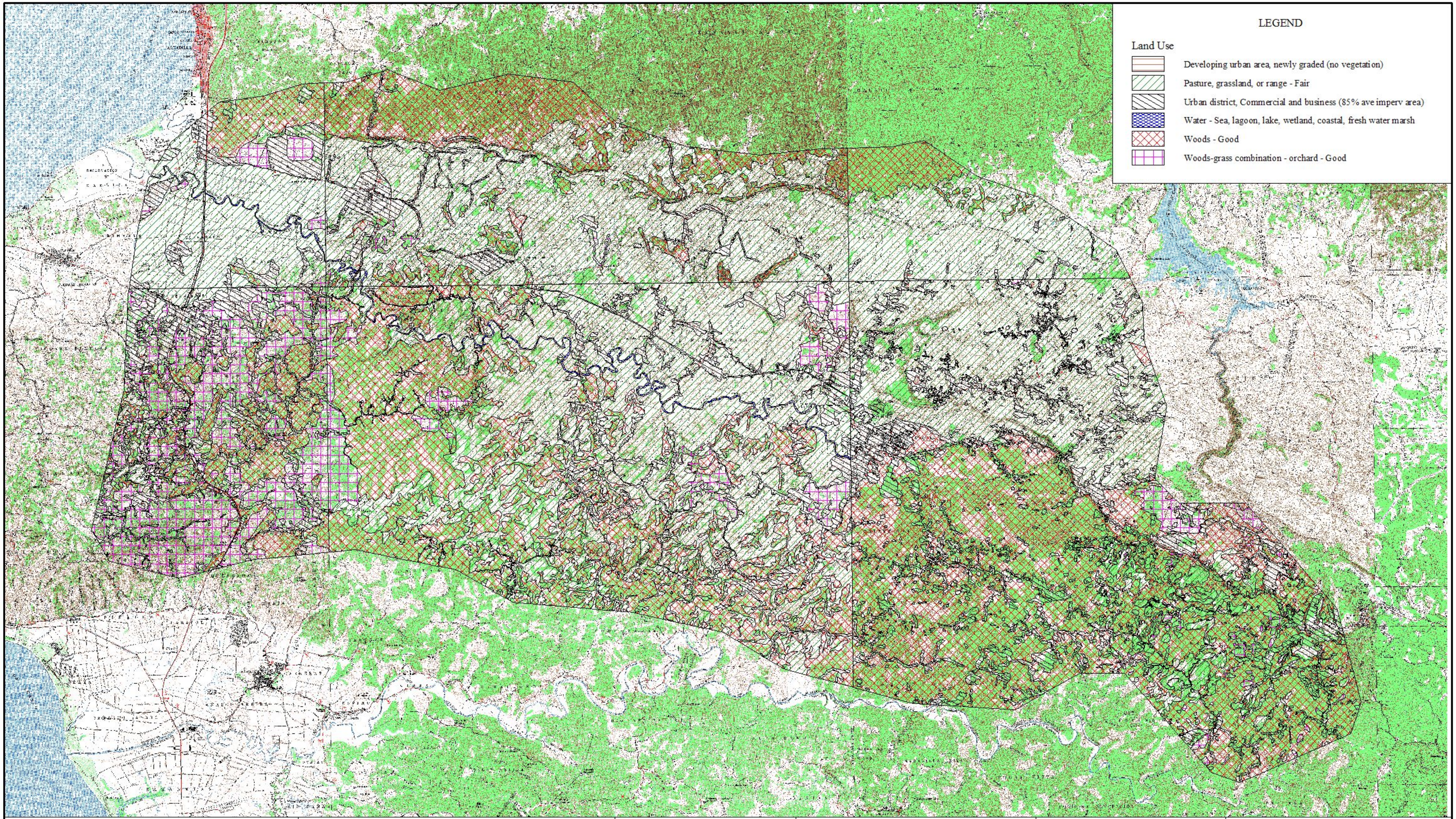


PLATE 4. DISCOVERY BAY RESORT AND MARINA
AGUADA, PUERTO RICO
RIO CULEBRINAS WATERSHED STUDY AREA
LAND USE WITH LOCATION MAP

N

SCALE 1:90,000