APENDICE B:

Tabla 1: Listado de Flora

Tabla 2: Listado de Fauna

TABLA 1

INVENTARIO DE FLORA

Familia	Nombre Científico	Nombre Común
AMARANTHACEAE	*Achyranthes aspera L. var. aspera*	Rabo de gato
ASCLEPIADACEAE	*Calotropis procera*	Algodón de seda
COMPOSITAE	*Emilia fosbergii* *Parthenium hysterophorus* *Vernonia sp.*	Clavelillo rojo Artemisa cimarrona Yerba socialista
CYPERACEAE	*Cyperus iria* *Cyperus rotundus* *Cyperus surinamensis*	-- Coquí --
EUPHORBIACEAE	*Jatropha gossypifolia*	Tautuba
LABIATAE	*Leonotis nepetifolia*	Botón de cadete
LEGUMINOSAE CAESALPINIODEAE MIMOSOIDEAE PAPILIONOIDEAE	 *Tamarindus indica* *Parkinsonia aculeata* *Leucaena leucocephala* *Mimosa ceratonia* *Mimosa pigra* *Mimosa pudica* *Pithecellobium dulce* *Prosopis pallida* *Samanea saman* *Andira inermis* *Sesbania sesban*	 Tamarindo Palo de rayo Zarcilla Zarza -- Moriviví Guamá americano Bayahonda Dormilón Moca Sesbania
MALPIGHIACEAE	*Stigmaphyllum tormentosum*	Bejuco de toro
MALVACEAE	*Sida acuta* *Urena lobata*	Escoba blanca Cadillo
NYCTAGINACEAE	*Guapira fragans*	Corcho
ONAGRADEAE	*Ludwigia sp*	--
POACEAE	*Andropogon annulatus* *Andropogon bicornis* *Eleusine indica* *Hymenachne amplexicaulis* *Sorghum halepense* *Urochloa maxima*	-- -- Pata de gallina Trompetilla Yerba Jonson Yerba de guinea
POLYPODIACEAE	*Acrostichum sp.*	--
RUBIACEAE	*Randia aculeata* *Spermacocce verticilata*	Tintillo Botón blanco
SOLANACEAE	*Solanum torvum*	Berenjena cimarrona
STERCULIACEAE	*Guazuma ulmifolia*	Guacima
VITACEAE	*Cissus sicyoides*	Bejuco de caro

TABLA 2

FAUNA

Familia	Nombre Científico	Nombre Común
AVES		
ACCIPITRIDAE	*Buteo jamaicensis*	Guaraguao
ARDEIDAE	*Casmerodius albus*	Garza real
COLUMBIDAE	*Columba livia* *Columbina passerina* *Zenaida asiatica* *Zenaida aurita*	Paloma casera Rolita Tórtola aliblanca Tórtola cardosantera
CUCULIDAE	Crotophaga ani	Judío
EMBERIZIDAE	*Coereba flaveola* *Tiaris bicolor* *Quiscalus niger*	Reinita común Gorrión negro Chango
ESTRILDIDAE	*Estrilda melpoda*	Veterano
MIMIDAE	*Margarops fuscatus* *Mimus poliglottos*	Zorzal pardo Ruiseñor
TODIDAE	*Todus mexicanus*[1]	San Pedrito de Puerto Rico
TYRANNIDAE	*Tyrannus dominicensis*	Pitirre
VIREONIDAE	*Vireo latimeri*[1]	Bien-te-veo de Puerto Rico
ANFIBIOS		
LEPTODACTYLAE	*Leptodactylus albilabris*	Sapito de labio blanco
REPTILES		
TEIIDAE IGUANIDAE	*Ameiva exul* *Anolis cristatellus* *Iguana iguana*	Siguana Lagartijo común Gallina de palo

[1] **Especie endémica**

APÉNDICE C:

Fotos del Área de Estudio

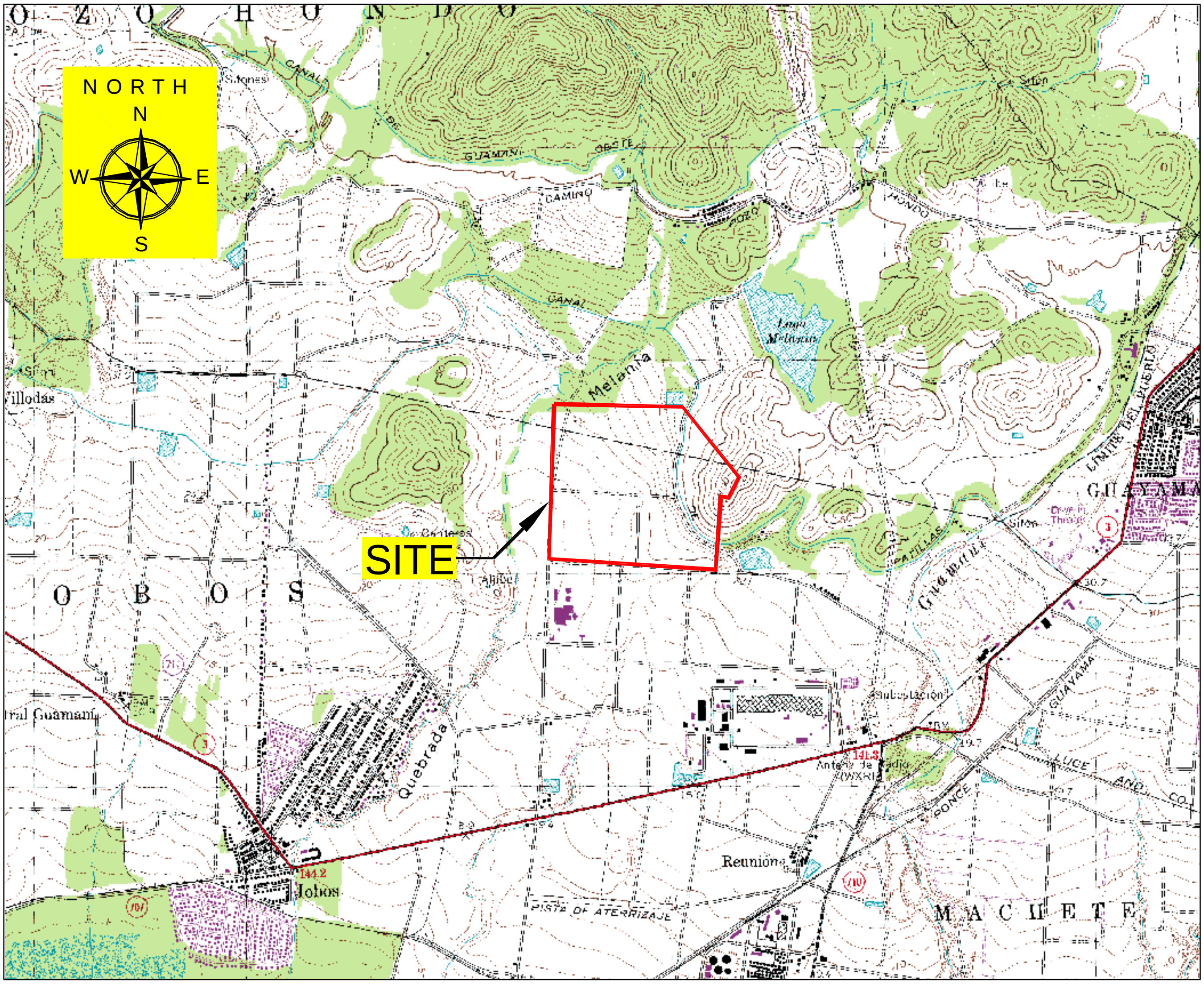

Area Humeda TN-4


Area Humeda TN-4


Area Humeda TN-4


Area del Proyecto TN-4

HYDROLOGIC-HYDRAULIC STUDY
SUPER-MAXIMUM SECURITY INSTITUTION
GUAYAMA, P.R.



APRIL 2006

PREPARED FOR:

CORRECTION ADMINISTRATION
P.O. BOX 71308
SAN JUAN, PR 00936
Phone: 787-273-6464

PREPARED BY:

OSVALDO RIVERA & ASSOC.
P.O. BOX 9042
SAN JUAN, PR 00908
Phone: 787-725-2729
Fax: 787-721-0467

TABLE OF CONTENTS

FIGURES

APPENDIX A adICPR Hydrologic-Hydraulic Input and Output Files for Existing Condition

APPENDIX B adICPR Hydrologic-Hydraulic Input and Output Files for Proposed Condition

APPENDIX C adICPR Hydrologic-Hydraulic Input and Output Files for Future Condition

LIST OF FIGURES

LIST OF TABLES

1. INTRODUCTION

1.1 Project Description and Location

The "Departamento de Corrección y Rehabilitación de Puerto Rico" proposes a Super Maximum Security Institution at state road PR-7707 in Jobos Ward, municipality of Guayama. **Figure 1** shows the project location on a partial reproduction of the USGS Central Aguirre topographic quadrangle. The project will consist of 500 cells in the first phase and 2,500 cells in the second one. **Figure 2** illustrates the proposed project layout for the first phase.

1.2 Scope and Purpose of Report

This report summarizes the results of the Hydrologic/Hydraulic (H/H) analysis for the proposed development site. The study evaluates the need for the sizing of a stormwater detention structure, in accordance with Puerto Rico Planning Board Regulation #3. The hydraulic structures to convey onsite and offsite runoff are also designed in this study.

1.3 Report Limitations and Warnings

It shall be the responsibility of the site engineer or the project's geotechnical consultant to adapt the hydraulic design recommendations included in this report, to the soil and other conditions at the site on any matters concerning slope stability, conflicts with other infrastructure, etc.

1.4 Parties Involved

Project Name:	Super Maximum Security Institution
Owner:	Departamento de Corrección y Rehabilitación
Civil Engineer:	Osvaldo Rivera & Associates
H-H Consultant:	Osvaldo Rivera & Associates

1.5 Authorization

Preparation of this report was authorized by means of written contract with Architect Jaime Gaztambide in representation of “Departamento de Corrección y Rehabilitación de P.R.”

2. STUDY AREA DESCRIPTION

2.1 Topography and Water Bodies

The topography of the project area presents elevations ranging from 116m to 16m. The area of the project site encompasses a total of 142 acres. The first phase of the project will develop approximately 40 acres and the rest of the area will be developed during the second phase construction as illustrates **Figure 3**.

The drainage pattern at the project site is mainly to the southwest where stormwater reaches the PR-7707 embankment and crosses the road by means of various hydraulic structures. **Figure 4** and **Figure 5** present the project site topography and existing hydraulic structures, respectively.

Runoff from three offsite areas located to the north of the site discharge into the project site following the southwestern drainage pattern mentioned and discharging through the hydraulic structures beneath PR-7707. These correspond to an area located directly to the north of the site, an overflow of Melanía Creek, and the overflow of the Melanía Reservoir.

Runoff from the area located directly to the north of the site travels from northeast to southwest prior discharging to the project site. This area encompasses a total of 102 acres.

Melanía Creek discharges from north to south and crosses the PR-7707 embankment by means of 4-60in pipes and a 40-in pipe which are located to the northwest of the site. Since these pipes do not have sufficient hydraulic capacity to convey Melanía Creek, the overflow of this stream travels south entering the project site and discharging through the hydraulic structures that cross road PR-7707 which entrances are located onsite.

The Melanía Reservoir is located to the northeast of the site. Stormwater from 600 acres reach this reservoir which includes two spillways for discharging its overflow. This overflow discharges to the offsite area located directly to the north of the site and thus eventually reaches the pipes located at the west limit of the site.

Runoff from a fourth offsite area enters the project site at its southeast limit. **Figure 6** presents the flow pattern at the study area.

2.2 Prior Studies and Floodplain Mapping

Neither FEMA nor the Puerto Rico Planning Board has performed a study of the area before. **Figure 7** illustrates a partial reproduction of the FEMA Flood Insurance Rate Map showing the project site location.

2.3 Field Observations

The field visit to the project area was made on February 2006 and it revealed the following:

- The drainage pattern at the project site is to the southwest.
- Stormwater from the project site crosses PR-7707 by means of four sets of pipes as illustrated in **Figure 5**.
- Melanía Creek watershed is located to the north of the site. The stream travels from north to south and crosses road PR-7707 by means of 4-60inch pipes and a 40-in one (see **Figure 5**). The topography of the area causes the overflow of the stream at the set of pipes to travel south, join the project site runoff and discharge through the rest of the hydraulic structures that cross PR-7707.
- The overflow from Melanía Reservoir also reaches the project site area. The lake discharges its overflow by means of two spillways.
- Offsite areas located to the north and the east also discharge to the project site area.

2.4 Field Data

A topographic survey of the site, referenced to mean sea level was provided by the project owner. Land surveyor René Guerra Menéndez prepared the topographic plans for the project site. A copy of the certified topographic survey is included in the back pocket of this report, and is also reproduced as

Figure 4. All elevations in this report are referenced to mean sea level vertical datum unless otherwise specified.

3. STUDY APPROACH

Three models were prepared to determine the effect of the proposed project: an existing condition model, a proposed condition model and a future condition model.

The existing condition model represents the pre-development conditions in the study area. Peak discharges are determined for the project site area and for the rest of the offsite areas that contribute in runoff to the hydraulic structures that cross PR-7707. The upstream ends of these structures are located within the project site. This model determines the existing water surface levels in the area and the hydraulic behavior of the existing structures.

The proposed condition model accounts for the change of hydrologic parameters at the proposed site to determine the hydrologic-hydraulic effects in the area. This model represents the first phase of the development. Since the proposed development will produce an increase in site runoff as compared to the existing undeveloped condition, a detention structure is provided in order to comply with Puerto Rico Planning Board Regulation # 3. The hydraulic structures to convey onsite and offsite runoff are designed. Water surface levels are determined and compared with the existing ones in order to demonstrate the compliance of the project site with Planning Board Regulation No.13.

The future condition model represents the second phase of the development. Compliance with Regulations No. 3 and No. 13 are also determined for this phase of the project.

The hydraulic analysis of the study area was performed using the one-dimensional unsteady-flow adICPR model (Streamline Technologies, 2000). This model dynamically routes storm water through open channels, closed conduits, and detention ponds. Each node in adICPR represents a control volume. Change in storage for each node is calculated based on the difference between inflows and outflows at each time step during the simulation period, and the change in storage is used to determine elevations at each node at the end of each time step. Flow through each link is calculated from the known elevations at each end of the link and the hydraulic properties of the link.

4. HYDROLOGIC ANALYSIS

4.1 Methodology

The hydrologic modeling of the project site and the offsite watersheds was performed using the Soil Conservation Service Unit Hydrograph method, incorporated in the adICPR model version 2.02. Parameters used in this method include basin area, curve number and time of concentration.

4.2 Watershed Delimitation

The existing condition model identified four watersheds for the analysis. These are Melanía Creek basin, Melanía Reservoir basin, the offsite basin that discharges to the site at the east (includes a section of the site), and the basin tributary to the hydraulic structures that cross PR-7707. The latter includes the majority of the project site area and the area located directly to the north of the site. **Figure 8** presents watershed limits for the existing condition.

In the proposed condition, the project site area was sub-divided in four basins as illustrated in **Figure 9**. This division was made according to the proposed drainage pattern in the project site and the layout of proposed hydraulic structures. The offsite basins remain unchanged except the north offsite basin which is modeled separately from the project site.

The future condition model presents the area of the project site sub-divided in five basins according to the future drainage pattern. The rest of the basins also remain unchanged in this condition. **Figure 10** presents watershed limits for the future condition.

Table 1 presents the areas of the identified basins for the three conditions.

Table 1: Watershed Areas (acres) for Existing, Proposed and Future Conditions

Basin	Area
Existing	
BasinPipes (includes project site and Offsite north)	227
Melanía Creek	414
Melanía Lake	602
BasinEast	122
Proposed	
Site 1	44.2
Site 2a	19.7
Site 2b	11.1
Site 3	50.2
Offsite North	102
Future	
Site 1	44.2
Site 2a	19.7
Site 2b	11.1
Site 3a	11.9
Site 3b	38.3
Offsite North	102

4.3 Soils and Curve Number

The curve number represents the runoff potential within the watershed and is estimated based on soil type (hydrologic soil group), land use, and antecedent moisture condition II. Soil types within the watershed areas were obtained using the Soil Survey of Humacao Area, Puerto Rico (**Figure 11**), published by the Natural Resources Conservation Service (Boccheciamp, 1977). Table 2 presents the soils and hydrologic soil groups within the identified basins. Table 3 presents curve numbers for the existing, proposed and future conditions.

Table 3: Hydrologic Soil Groups of Identified Soils within Basins

Soil	HSG
AmB	B
AmC2	B
VvB	B
PlB	D
DrF	D
Rs	D
DgF2	D
Vc	D
Gm	B
Vs	B

Table 3: Curve Number for Watersheds

Basin	HSG	Cover Type	**CN**
Existing			
BasinPipes (includes Project Site +Offsite North)	B	Pasture (Area=114.7ac)	76
	D	Pasture (Area=89.96ac)	
Melanía Creek	B	Pasture (Area=117.4ac)	80
	D	Pasture (Area=291.96ac)	
Melanía Lake	B	Pasture (Area=185.2ac)	79
	D	Pasture (Area=417ac)	
BasinEast	B	Pasture (Area=68.2ac)	75
	D	Pasture (Area=47.6ac)	
Proposed			
Site (all sub-basins)	B, D	Developed	92
BasinEast	B	Pasture(53.6ac), Dev.(17.75ac)	80
	D	Pasture(41.6ac), Dev.(9.2ac)	

Table 3: Curve Number for Watersheds

Basin	HSG	Cover Type	**CN**
Future			
Site (all sub-basins)	B, D	Developed	92
BasinEast	B	Pasture(53.6ac) Dev.(17.75ac)	80
	D	Pasture(41.6ac) Dev.(9.2ac)	

4.4 Time of Concentration

Time of concentration was computed using the Soil Conservation Service method (TR-55).

Time of concentration was estimated with the following equation:

$$t_c = L / (3600 * V)$$

where,

t_c = time of concentration (hr)

L = flow length (ft)

V = average flow velocity (ft / s) from figure 3 of TR-55, and

3600 = conversion factor from seconds to hours.

Table 4 presents time of concentration computed for existing, proposed and future conditions.

Table 4: Time of Concentration (minutes) for Existing, Proposed and Future Conditions

Basin	Tc
Existing	
BasinPipes (includes project site and Offsite north)	33
Melanía Creek	48
Melanía Lake	47
BasinEast	34
Proposed	
Site 1	8
Site 2a	8
Site 2b	5
Site 3	9
Offsite North	10
Future	
Site 1	8
Site 2a	8
Site 2b	5
Site 3a	5
Site 3b	6
Offsite North	10

4.4 Rainfall

The 24-hour precipitation depths for return periods of 2- and 100-years were obtained from Technical Paper #42 (U.S. Department of Commerce, 1961). Values of 4.9” and 12” were obtained for 2- and 100-yr return periods respectively.

4.5 Results of Hydrologic Analysis

Table 5 summarizes peak discharges for existing, proposed and future conditions of the project site and the offsite watersheds. These results were obtained using the adICPR model for 2- and 100-yr Table 5 return periods. Input and output files for hydrologic modeling of project site are included in Appendix A and B, respectively.

Table 5: Peak Discharges (cfs) for Watersheds

Basin	2-yr	100yr
Existing		
BasinPipes (includes project site and Offsite north)	438	1,556
Melanía Creek	717	2,357
Melanía Lake	1,022	3,456
BasinEast	224	818
Proposed		
Site 1	229	592
Site 2a	102	264
Site 2b	59	153
Site 3	256	661
Offsite North	355	1,190
Future		
Site 1	229	592
Site 2a	102	264
Site 2b	59	153
Site 3a	63	163
Site 3b	203	523
Offsite North	355	1,190

4.6 Verification of Peak Discharge

Peak discharges determined by the ICPR model, were checked for reasonableness using the Rational Method and the USGS Regional Regression Equations developed by López et.al. (1979).

The Rational Method equation has the following form:

$$Q_{100} = CI_{100}A$$

where;

Q_{100} =100- year peak discharge (cfs)

C = runoff coefficient.

I_{100} = Mean precipitation, (in. / hr.)

A = watershed area, (acres)

The López equation has the following form:

$$Q_{100} = 286*A^{0.832}*P^{0.531}$$

where;

Q_{100} = 100- year peak discharge (cfs)

A = watershed area (mi^2)

P = Mean annual precipitation (in. / yr.)

Table 6 and Table 7 summarize the parameters used in the verification methods for the watersheds and compares results with those obtained with the adICPR model.

Table 6: Rational Method Parameters for 100-yr Event

Basin	C	I (in / hr)	Area (acres)	Rational Method (cfs)	adICPR (cfs)
Existing					
BasinPipes	0.75	7	227	1,192	1,556
Basin East	0.75	6.9	122	631	818
Proposed					
Site 1	0.70	15	44.2	464	592
Site 2a	0.70	15	19.7	207	264
Site 2b	0.70	18	11.1	140	153
Site 3	0.70	14	50.2	492	661
Offsite North	0.75	13	102	1,033	1,190
Future					
Site 1	0.70	15	44.2	464	592
Site 2a	0.70	15	19.7	207	264
Site 2b	0.70	18	11.1	140	153
Site 3a	0.75	18	11.9	163	163
Site 3b	0.75	17	38.3	523	523
Offsite North	0.75	13.5	102	1,190	1,190

Table 7: Regression Equation Parameters for 100-yr Event of Melanía Creek and Melanía Reservoir Watersheds

Basin	A (mi2)	P (in)	Regression Equation (cfs)	adICPR (cfs)
Melanía Creek	0.65	58	1,726	2,357
Melanía Reservoir	0.94	58	2,346	3,456

The hydrologic results from the verification method are similar, and the adICPR hydrologic modeling results are accepted as reasonable.

5. <u>HYDRAULIC ANALYSIS</u>

5.1 Models Prepared

The following hydrologic-hydraulic models were prepared for the hydraulic analysis of the proposed project site:

Existing Condition: This model represents the existing conditions at the project site. Peak discharges are determined for the watersheds. Cross sections were incorporated in the ICPR model in order to determine water surface elevations in the area. The existing hydraulic structures are also modeled. **Figure 12** presents the schematic link-node diagram for this model.

Proposed Condition: The existing condition model was modified to incorporate the proposed land use change and detention structure as illustrated in the link-node diagram of **Figure 13**. Hydraulic structures to convey onsite and offsite runoff are designed for the first phase of the development.

Future Condition: The proposed condition model is modified to account for the development in the second phase of the project. **Figure 14** presents schematic link-node diagram for this model.

5.2 Hydraulic Characteristics at Study Area

5.2.1 Cross Sections

Existing Condition

Eighteen (18) cross sections were used to model the hydraulic conditions in the area in the existing condition. Nine (9) of this cross sections were taken at Melanía Creek and the rest were taken at the stormwater path of onsite and offsite runoff within the project site. **Figure 15** shows cross sections location.

Proposed Condition

The cross sections used in this model correspond to the ones used in the existing condition with the difference that cross sections XS30 and XS40 are modified to account for the proposed fill of the first phase of the project.

Future Condition

In this condition, cross sections XS 40 and XS 30 are eliminated and substituted for a Stage / Area node which represents the storage of runoff upstream the 4-60 inch pipes that convey Melanía Creek (see **Figure 16**). This storage area is formed by the proposed fill of the second phase development. The overflow of runoff from this storage area will be discharge through a special structure to the detention pond.

5.2.2 Roughness Coefficients

Values of Manning's hydraulic roughness coefficient (n-values) were based on field inspection and with reference to Chow (1959) and Barnes (1967). N-values of 0.04 and 0.05 were used at the main channel and the overbanks, respectively.

For the concrete culverts the corresponding n-value is 0.013. The contraction and expansion coefficients for the natural channel sections are 0.1 and 0.3 respectively, and for culvert entrance and exit are 0.3 and 0.5, respectively.

5.2.3 Existing Hydraulic Structures

Figure 5 illustrates the existing hydraulic structures that cross road PR-7707 and convey stormwater from onsite and offsite watersheds. Table 8 presents the parameters for the various structures.

Table 8: Hydraulic Parameters of Existing Hydraulic Structures

Pipe	No. Barrels / Size (in)	Length (m)	$IE_{u/s}$ (m)	$IE_{d/s}$ (m)
Pipe 42B	4-42	37	15.075	14.71
Pipe 60	4-60	38	15.52	15.0
Pipe 42A	4-42	50	15.089	14.92
Pipe 36	6-36	40	15.19	15.18
Pipe 40*	1-40	62.9	18.74	17.87
Pipe Creek*	4-60	20.3	18.34	18.17

*Convey Melanía Creek

5.2.4 Starting Water Surface Elevation

Since the existing hydraulic structures discharge to an open area located to the west of PR-7707, the existing ground elevation at the points of discharge were used as boundary condition in the hydraulic models.

5.3 Existing Condition

5.3.1 Overflow Patterns

The existing condition model represents the pre-development conditions in the area. Cross sections taken at Melanía Creek and at the overland flow path of onsite and offsite runoff were used in the pre-development model to determine the existing hydraulic conditions in the area. Weirs were used to represent the different overflow conditions in the study area. These correspond to the following:

1. From overland flow area to Melanía Creek –flow that diverts to the stream because of the topography of the terrain.

2. Weir flow through PR-7707- weir flow caused by the lack of capacity of the existing pipes.

3. Overflow from Melanía Creek to site- the overflow of the creek at the 4-60in and 40-in pipes travels south and enters the project site.

Figure 17 presents these flow patterns.

5.3.2 Melanía Reservoir

Melanía Reservoir receives runoff from 600 acres as determined in the hydrologic analysis presented in Section 4. The reservoir has two concrete spillways with dimensions 9mL x 2.8mH and 6.0mL x 2.8mH. These structures discharge the reservoir's overflow to the southwest where it eventually reaches the project site. The stage-area relationship was obtained from the USGS topographic quadrangle of Central Aguirre and it is shown in Table 9.

Table 9: Stage-Area Relationship for Melanía Reservoir

Stage	Area (sqm)
40	92,000
42.8	92,000

5.3.3 Results

Table 10 presents the 100-yr water surface elevations at the project site and Melanía Creek for the existing condition.

Table 10: Water Surface Elevations (m) for 100-yr Event-Existing Condition

Cross Section	WSE (m)
Melanía Creek	
XS 8	26.7
XS 7	25.3
XS 6	23.9
XS 5	22.8
XS 4	21.9
XS 3	21.1
XS 2	20.2
Existing 4-60in Pipes and 40-in Pipe	
XS 1	19.3
Overland Flow Path	
XS 75	31.6
XS 70	29.6
XS 60	27.6
XS50	24.5
XS40	21.0
XS 30	18.6
XS 20	17.7
Existing Pipes at PR-7707	
XS 15	16.5
Basin East	
XS 310	21.6
XS 300	20.7
XS 250	19.9
XS 200	19.7
XS 100	17.7

Table 11 presents the overflow that occurs in the area.

Table 11: Overflow Condition at Study Area

Condition	Flow (cfs)
Weirflow through PR-7707	1,630
Overflow from Pipe Creek to Project Site	3,183
Overflow from North basins to Melanía Creek	1,600

5.4 Proposed Condition

The proposed condition model presents a detention pond to reduce the post-development peak discharge to not more than the existing condition discharge. The channeling of onsite and offsite runoff within the project site is also presented in this model which represents the first phase of the development. **Figure 18** presents the proposed hydraulic structures within the project site.

5.4.1 Detention Pond Configuration

All onsite and offsite runoff will discharge to the proposed detention pond which location is presented in **Figure 18**. This structure will discharge through the existing hydraulic structures that cross PR-7707 and through three new set of pipes. In order to control the 2-yr and 100-yr events some modifications are presented to the existing pipes. As illustrated in **Figure 19** only one set of 42-in pipes and two (2) 24-in orifices will control the 2-yr event. In order to achieve this, the rest of the upstream end of the pipes will be encased with a special concrete structure. These structures will then include a rectangular weir at the top at the 2-yr event water surface elevation to control the 100-yr event. Table 12 presents the detention pond configuration. **Figure 19** through **Figure 22** presents the schematic design of the pond. <u>The detention pond was designed to account for both phases of the development.</u>

Table 12: Design Parameters for Detention Pond

Parameter	Value
Pond invert elevation (m)	14.7
Minimum Top Area (m^2)	62,100
Minimum Bottom Area (m^2)	39,700
Depth (m)	3.8
100-yr water level within pond (m)	18.15
Pond bottom slope (m / m)	0
Free Board (m)	0.35

5.4.2 Proposed Storm Sewer System

Table 13 presents the design parameters for the proposed hydraulic structures which will convey offsite and onsite runoff. The final point of discharge of these structures corresponds to the detention pond. **Figure 18** presents their location. A special structure is also proposed to convey the overflow of Melanía Creek to the detention pond without increasing the existing flow through the 4-60-inch pipe that carries the stream beneath PR-7707. The existing 40-inch pipe is eliminated in proposed and future conditions.

Table 13: Design Parameters for Proposed Hydraulic Structures

Structure	Tributary Basin	Size	Length (m)	IE $_{U/S}$ (m)	IE $_{D/S}$ (m)	S (m / m)	Q_{acc} (cfs)
Crossing 1	Site 2a	2-1.52m x1.52m	68	20.2	19.5	0.01	264
Crossing 2	Site 2b	1.52m x 1.52m	47	24	23	0.02	152
Channel 2b	Site 2b	1.2m x 1.2m	280	28.5	24	0.016	152
PipeNew	-	3 Sets 4-1.52m x 1.52m	40	15.19	15.18	0.0025	-
Weir Creek	Creek Overflow	23mW x 2mH	-	18.5	-	-	-

5.4.3 Results

5.4.4 Detention Analysis

Table 14 presents the pre- and post- development peak discharges showing the effect of the detention pond for the 100-yr and 2-yr events. The comparison is made at the downstream end of the existing 60-in pipe that convey Melanía Creek and at the downstream end of the existing pipes within the project site.

Table 14: Peak Discharge (cfs) for Existing and Proposed Conditions

Return Interval	Existing Condition		Proposed Condition	
	2yr	100yr	2yr	100yr
D/S Melanía Creek Pipe (Bndry 1)	365	640	197	627
D/S Pipes within Site (Bndry 2)	536	3,753	503	3,600

Table 15 presents a comparison between inflows at outflows at the detention pond.

Table 15: Comparison between Inflows and Outflows at Detention Pond

Return Interval	Inflow	Outflow
2-yr	1,150	503
100-yr	4,200	3,600

As illustrated in the results, the proposed detention pond reduces the proposed project peak discharges below the existing ones in compliance with Planning Board Regulation No.3.

5.4.5 Proposed Storm Sewer System

Table 16 presents a comparison between existing and proposed water surface elevations at Melanía Creek and the project site. The proposed condition model includes all hydraulic structures and the detention pond.

Table 16: Existing and Proposed Water Surface Elevations (m) for 100-yr Event

Cross Section	Existing	Proposed	Difference (m)
Melanía Creek			
XS 8	26.7	26.7	0
XS 7	25.3	25.2	-0.1
XS 6	23.9	23.9	0
XS 5	22.8	22.8	0
XS 4	21.9	21.9	0
XS 3	21.1	21.1	0
XS 2	20.2	20.2	0
Existing 4-60in Pipes			
XS 1	19.3	19.2	-0.1
Overland Flow Path			
XS 75	31.6	31.5	-0.1
XS 70	29.6	29.5	-0.1
XS 60	27.6	27.6	0
XS50	24.5	24.5	0
XS40	21	21.1	0.1
XS 30	18.6	19.5	0.9
XS 20	17.7	-	-
Existing and Proposed Pipes at PR-7707			
XS 15	16.5	16.5	0
Basin East			
XS 310	21.6	21.7	0.1
XS 300	20.7	20.5	-0.2
XS 250	19.9	-	-
XS 200	19.7	-	-
XS 100	17.7	-	-

As illustrated in the results, the proposed hydraulic structures do not increase water surface elevations off property limits by more than 0.15m in compliance with Planning Board Regulation No.13. The increase of 0.9m that occurs at XS 30 occurs within property limits without affecting any neighbor.

5.5 Future Condition

The future condition model represents the second phase of the development. The detention pond has already been designed for both phases of the development. Besides the hydraulic structures proposed in the first phase of the development, a new channel is proposed in this model. Cross sections XS 40 and XS 30 are eliminated in this model. A stage / area node is modeled upstream the 4-60in pipes that convey Melania Creek to account for the storage caused by the fill of the second phase development (see **Figure 16**). This storage area will receive runoff from Melanía Creek and the northern basins. The special structure provided for the Melanía Creek overflow is modified in order to maintain the flow through the 4-60in pipes equal than in the existing condition. This weir will convey overflow from the natural storage area to the detention pond. Table 17 presents the design parameters for the new structures. Their location is presented in **Figure 24**.

Table 17 : Design Parameters for Proposed Hydraulic Structures-Future Condition

Hydraulic Structure	Tributary Basin	Size	Length (m)	IE $_{U/S}$ (m)	IE $_{D/S}$ (m)	S (m / m)	Q_{acc} (cfs)
Channel F1	Site 3a	1.4m x 1.4m	180	23	19.5	0.0194	163
Channel F2	Site 3b	2.9m x 2.9m	82	19.5	18.5	0.012	1,100
Weir Overflow Creek	-	29 mW x 2mH	-	-	-	-	-

Table 18 present a comparison between existing and future condition 100-yr water surface elevations.

Table 18: Existing and Future Water Surface Elevations (m) for 100-yr Event

Cross Section	Existing	Future	Difference (m)
<u>Melanía Creek</u>			
XS 8	26.7	26.6	-0.1
XS 7	25.3	25.2	-0.1
XS 6	23.9	23.9	0
XS 2-(Stage AreaN)	20.2	20.2	0
	Existing 4-60in Pipes		
XS 1	19.3	19.2	-0.1
<u>Overland Flow Path</u>			
XS 75	31.6	31.5	-0.1
XS 70	29.6	29.6	0
XS 60	27.6	27.5	-0.1
XS50	24.5	24.6	0.1
<u>Basin East</u>			
XS 310	21.6	21.7	0.1
XS 300	20.7	20.5	-0.2

As illustrated in the results, the hydraulic structures proposed for the future phase of the development do not increase water surface elevations by more than 0.15m in compliance with Planning Board Regulation No.13.

Table 19 presents the existing and future condition discharges showing the effect of the detention pond for the 100-yr and 2-yr events. As previously explained, the detention pond was design to account for both phases of the development.

Table 19: Peak Discharge (cfs) for Existing and Future Conditions

Return Interval	Existing Condition		Proposed Condition	
	2yr	100yr	2yr	100yr
D/S Melanía Creek Pipe (Bndry 1)	365	640	199	622
D/S Pipes within Site (Bndry 2)	536	3,753	517	3,594

Table 20 presents a comparison between inflows at outflows at the detention pond.

Table 20: Comparison between Inflows and Outflows at Detention Pond

Return Interval	Inflow	Outflow
2-yr	1,203	517
100-yr	4,080	3,594

As illustrated in the results, the proposed detention pond works adequately in the future condition in compliance with Planning Board Regulation No.3.

6. CONCLUSIONS AND RECOMMENDATIONS

1. Existing, proposed and future condition 100-yr peak discharges at onsite and offsite basins resulted in the following:

Basin	100yr
Existing	
BasinPipes (includes project site and Offsite north)	1,556
Melanía Creek	2,357
Melanía Lake	3,456
BasinEast	818
Proposed	
Site 1	592
Site 2a	264
Site 2b	153
Site 3	661
Offsite North	1,190
Future	
Site 1	592
Site 2a	264
Site 2b	153
Site 3a	163
Site 3b	523
Offsite North	1,190

2. The pre-development 100-yr peak discharges at the points of analysis correspond to 640 cfs and 3,753 cfs
3. The post-development (with detention) 100-yr peak discharges at the point of analysis correspond to 627cfs and 3,600cfs. The proposed detention pond reduces the proposed project peak discharge below the existing one

complying with Planning Board Regulation # 3.

4. Onsite and offsite runoff will discharge to the detention pond.
5. The proposed condition model represents the first phase of the development. The future condition represents the second one. The proposed detention pond has been designed to work for both phases of the development.
6. The 100-yr water level within the detention pond is 18.15 m. The minimum finished floor elevation for the proposed building in the surrounding of the structure should be set 1.0m higher than the 100-yr water level within it.
7. The site engineer shall design storm water systems for the proposed project site to discharge into the detention structure. The site grading must provide overland flow paths to direct stormwater to the detention system.
8. The proposed detention pond and outlet pipes should be inspected periodically to avoid obstruction with debris and to insure the removal of accumulated sediment.
9. The proposed hydraulic structures to convey offsite and onsite runoff are presented in Section 5.4.5. These structures do not increase water surface elevations by more than 0.15m in compliance with Planning Board Regulation No.3.

7. REFERENCES

Boccheciamp, Rafael A. 1978. “Soil Survey of Humacao Area” U.S. Soil Conservation Service. San Juan.

Streamline Technology Inc. 2000. “ICPR- Stormwater Management ”

Technical Paper #42 U.S. Department of Commerce, Washington D.C. 1961.

U.S. Geological Survey, Water Resources Division. “Water Resource Data for Puerto Rico and the Virgin Islands”,1960 – 1995. San Juan.