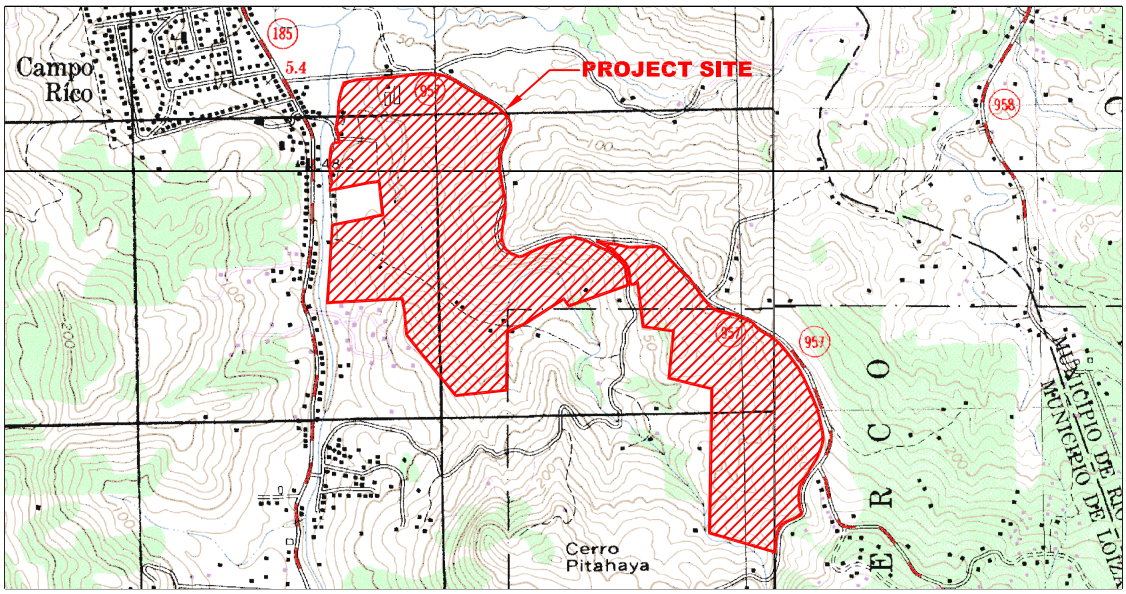
DESARROLLO ALTAMIRA II, INC.
San Juan, Puerto Rico

HYDROLOGIC-HYDRAULIC STUDY FOR CAMPO RICO NEW CASTLE CANOVANAS, PUERTO RICO



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HYDROLOGIC-HYDRAULIC STUDY
FOR CAMPO RICO NEW CASTLE DEVELOPMENT
CANÓVANAS, PUERTO RICO

Casiano Ancalle, P.E.

June, 2007

I. INTRODUCTION

A housing development project named Campo Rico New Castle is to be constructed in the Municipality of Canóvanas. The site access is at state road PR-957 that is connected to state road PR-185. Río Canóvanas bounds the northern west side of the project site. The project consists on housing units, recreational facilities, and green areas.

According to the regulatory flood maps, a small part of the site is located in the floodway of Río Canóvanas, and an unnamed watercourse crosses a portion of the property. However, the development of the project will not affect these areas. The development of the site is going to increase runoff. This increase has to be mitigated according to the prescriptions of the Puerto Rico Planning Board Regulation No. 3.

Purpose of Study

The purpose of the study is to determine the flood levels and footprint for the watercourse crossing the site, and to size the watercourse crossing structures. In addition, as the development of the site will increase the peak runoff discharge, the study will quantify the increase and provide means for mitigating it according to the Puerto Rico's Planning Board Regulation No. 3.

Approach

The following steps have been undertaken throughout the study:

Hydrologic Analysis: The following parameters were determined for the hydrologic analysis: drainage areas, average soil curve number and runoff lag time. Based on these parameters, discharge for 100-years frequency storm was determined for existing and proposed condition. HEC-1 model was used. Discharges for more recurrent storm events such as: 2-, 10-, 25-, 50- and 100-years, were also determined.

Hydraulic Analysis: A hydraulic analysis was performed in order to estimate the water surface elevations for the three storm watercourses. Existing condition's water levels were determined first. Water levels considering the crossing structures were determined. Comparison of water elevations for both conditions would fall within the limits established by Regulation No. 13. The US Army Corps of Engineer's HECRAS computer model was used.

Runoff Discharge Mitigation Analysis: A mitigation analysis was made in order to counteract the impact of the proposed development. A mitigation pond was employed as a detention structure. HEC-1 model was used for the mitigation analysis. 2-, 10-, 25-, 50- and 100 frequency discharges were analyzed for mitigation.

Conclusions and recommendations were elaborated.

Authorization

Eng. Francisco Charles on behalf of Desarollos Altamira, Inc. authorized this study, under a contract signed with Eng. Casiano Ancalle, principal of CA Engineering.

II. BACKGROUND INFORMATION

Location

The project is located in the Municipality of Canóvanas. The site bounds north with state road PR-957, west with Río Canóvanas, south with private properties, and east with the housing project Altamira I. The property access is through road PR-957 branching out from state road PR-185. The approximate total area of the project is 209.83 acres. Figure 1 illustrates the project site location on the USGS Quadrangle. Figure 2 shows the development layout.

Topography

The topography of the site slopes down to the northwest, towards Río Canóvanas. Ground elevations range from 220.00 to 45.00 meters (m.s.l.). See Figure 3.

Water Bodies

A storm watercourse crosses the site from south east to northwest to discharge into Río Canóvanas. The majority of the project's site runoff discharges to this watercourse. A very small part discharges directly into Río Canóvanas.

Flooding

A very small part of the project site is located in the Río Canóvanas floodway. But the project will not affect this area. Also, the channel of the storm watercourse crossing the project site, located at the northern portion of the project site is zoned floodable without elevations, which is to say that FEMA did not perform a detailed study at this segment. The present study includes a hydraulic analysis of this watercourse. A partial copy of sheets 860 and 780 of FEMA's flood map dated April 19, 2005 is included in Figure 7.

Former Studies

Though no specific studies have been made for the project site in the past, two studies are very much related to it, FEMA's study for Río Canóvanas and CA Engineering's study for Altamira. The segment of the river bounding the project site falls within the FEMA's study, its levels and flood footprint is used in this study. On February 2006, CA Engineering elaborated a HH study for Altamira development. This study has been endorsed by the Department of Environment and Natural Resources on October 16, 2006. Copy of this letter of endorsement is included in Appendix I.

Field Information

Field data used in this study was taken from the topographic information provided by Eng. Francisco Charles. Results obtained in this study are strictly based on this information. The topography is tied to mean sea level reference. This fieldwork is included in the pocket attached to the back cover of this study as Appendix H.

Study Level

This study is intended as an aid to the design engineer in the preparation of the construction drawings for the recommended structures. Figures, schematics and drawings must not be used as construction drawings. The design engineer must elaborate the construction drawings in agreements with the recommendations of this study.

III. HYDROLOGIC ANALYSIS

The computer program entitled Flood Hydrograph Package (HEC-1), developed by the U.S. Army Corps of Engineers [1990], was used for the performance of the hydrologic analysis. Using this program, the Unit Hydrograph method and the Runoff Curve Number (CN) method, both developed by the Soil Conservation Service (SCS), were applied to determine the design hydrograph. This was computed by a process of translating the rainfall excess into a runoff hydrograph known as convolution.

Peak discharges for 2-, 10-, 25-, 50- and 100-year storm frequency were estimated for the existing condition and proposed condition.

Drainage Areas

Two drainage areas have been identified for the existing condition at the project site: P1 and P2. Area P1 was subdivided in P1A, P1B, P1C, P1D, P1E and P1F; and drainage area P2 was subdivided in P2A, P2B and P2C. Offsite areas considered in the hydrologic analysis were: E0, E1, E2, E3 and E4. (Figure 4) At proposed condition, the project site drainage areas were slightly modified to fit the development need: Area P1 was subdivided in nine areas; while Area P2 kept its three subareas as shown in Figure 5. The total area of the project site is approximately 209.83 acres.

Table 1 shows the drainage areas taken into account for the hydrologic analysis, for both existing and proposed condition.

Table 1
Drainage Areas

	Name	**Area,** acres	
		Existing	**Proposed**
Project Areas	P1A	30.15	35.21
	P1B	8.95	--
	P1B1	--	9.70
	P1B2	--	6.01
	P1C	11.60	
	P1C1	--	10.61
	P1C2	--	12.73
	P1D	21.06	22.80
	P1E	45.23	18.69
	P1F	19.46	--
	P1F1	--	11.20
	P1F2	--	9.50
	P2A	13.39	22.20
	P2B	40.00	39.68
	P2C	19.99	11.50

Curve Numbers

Curve numbers were established. According to the SCS's Soil Maps, the project area is classified in ten types of soil: Mabi clay, Caguabo clay loam, Mucara silty clay loam, Toa silty, Aceituna

Silty, Rio Arriba, Reilly land, Rock land, Humatas stony and los Guineos silty. Weighted curve numbers were estimated for each drainage area A CN value of 93 was adopted for the developed site.

Appendix A includes the Curve Number computations for existing and proposed condition. Figure 6 shows the soil map corresponding to the project site and offsite areas.

Lag Time

The Lag Time (T_{Lag}) was estimated using the SCS method defined as:

$$T_{Lag} = \frac{L^{0.8}(S+1)^{0.7}}{1900\ Y^{0.5}}$$

where :	L	=	channel length (ft)
	S	=	1000/CN - 10
	Y	=	average watershed slope

Detailed Lag Time calculations are shown in Appendix A.

Rainfall Data

The variation of rainfall volume with time was required as part of the storm input for the SCS Curve Number method. Therefore, the development of a design storm with a rainfall frequency and duration was necessary to compute the design hydrograph for the watershed.

Rainfall data used in this study was obtained from the Technical Paper No. 42 (TP-42) [National Weather Service, 1961]. The 24-hour duration rainfall depths for 100, 50, 25, 10 and 2 years frequency are shown in Table 1.

Table 2
Rainfall for 2, 10, 25, 50 y 100 years

Duration *Hrs.*	**Precipitation** *Inches*				
	2-yr	***10-yr***	***25-yr***	***50-yr***	***100-yr***
0.083	0.61	0.70	0.89	1.00	1.19
0.25	1.32	1.62	2.00	2.23	2.59
1	2.70	3.65	4.20	4.60	5.15
2	3.20	4.60	5.25	5.75	6.50
3	3.70	5.15	5.80	6.50	7.10
6	4.60	6.25	7.15	8.10	9.00
12	5.40	7.50	8.80	10.00	11.00
24	6.40	9.00	10.25	11.75	13.00

Depth-Area Adjustment

Point rainfall estimates obtained from the TP-42 represent values for areas up to 10 mi^2; therefore, a depth-area adjustment should be applied to the rainfall data when the watershed area is greater. In this case, the largest watershed consists of 0.3880 mi^2. Hence, this adjustment was not applied.

Time Distribution of Rainfall

The SCS Type II Storm Distribution methodology was used to distribute the rainfall depth in time. This method is considered acceptable for small areas.

Rainfall Extraction

Rainfall extraction such as vegetative interception, depressional storage, and infiltration were estimated using the SCS's Runoff Curve Number method. Though this method is used to predict runoff volume directly, the rainfall extraction is incorporated in the model as function of the curve number of the watershed.

Hydrologic Analysis Results

Hydrologic analysis was made for all the drainage areas. Individual and composite runoff

hydrographs were obtained using HEC-1 methodology. Table 3 shows the summary of the results for this analysis. Input and output data for the HEC-1 model for existing and proposed condition are included in Appendix B and C, respectively.

Table 3
Peak Discharges

AREA		Peak Discharge, cfs				
		2-yr	**10-yr**	**25-yr**	**50-yr**	**100-yr**
Existing	P1B	36	49	62	71	84
	P1C	61	79	100	115	136
	COMB1	96	127	162	185	219
	P2C	81	109	134	151	175
	CHANN	80	107	133	149	174
	P2B	178	233	288	324	377
	CHANN	173	224	275	307	356
	P2A	80	99	126	142	169
	CHANN	72	92	117	131	155
	COMB2	297	390	478	538	626
	E0	871	1184	1436	1610	1856
	CHANN	869	1181	1437	1612	1853
	E1	458	608	744	835	967
	COMB3	1287	1743	2116	2370	2725
	COMB4	1552	2113	2570	2883	3333
	CHANN	1538	2099	2554	2866	3314
	E3	100	125	157	177	209

	CHANN	92	118	148	166	196
	P1F	102	130	162	183	214
	COMB5	194	248	310	349	410
	COMB6	1664	2272	2758	3092	3570
	CHANN	1653	2262	2745	3079	3555
	E2	218	277	346	389	456
	CHANN	201	262	325	365	428
	P1E	158	215	261	293	338
	COMB7	329	439	537	602	696
	COMB8	1960	2677	3244	3637	4194
	CHANN	1928	2647	3207	3597	4151
	P1D	91	124	156	178	209
	COMB9	1994	2739	3319	3722	4294
	CHANN	1959	2702	3278	3677	4245
	E4	79	98	124	140	165
	CHANN	72	88	111	125	143
	P1A	144	184	229	258	302
	COMB10	216	272	340	383	445
	COMB11	2323	2920	3543	3973	4584
Proposed	P1B1	44	57	69	77	89
	P1C1	51	65	79	89	103
	COMB1	95	121	148	165	191
	P2C	58	72	89	100	116
	CHANN	56	69	83	94	109
	P2B	191	241	296	331	383
	CHANN	183	232	281	314	361

P2A	109	138	169	189	220
CHANN	104	133	163	182	211
COMB2	338	428	518	580	668
E0	871	1184	1436	1610	1856
CHANN	869	1181	1437	1612	1853
E1	458	608	744	835	967
COMB3	1287	1743	2116	2370	2725
COMB4	1593	2150	2611	2930	3384
CHANN	1579	2136	2595	2913	3363
E3	100	125	157	177	209
PIPE	95	121	151	170	199
P1F1	56	70	87	97	113
COMB5	143	184	226	253	295
COMB6	1686	2279	2766	3102	3578
CHANN	1664	2259	2741	3073	3547
P1F2	48	61	74	83	97
COMB7	1700	2306	2797	3135	3618
CHANN	1687	2294	2782	3119	3601
E2	218	277	346	389	456
CHANN	201	262	325	365	428
P1E	92	116	142	159	185
COMB8	291	369	457	512	596
COMB9	1919	2599	3148	3525	4065
CHANN	1887	2570	3113	3489	4025
P1D	121	152	187	209	243
COMB10	1970	2680	3243	3633	4189
CHANN	1955	2667	3227	3617	4170
P1C2	63	79	97	108	126
COMB11	2010	2737	3313	3713	4280
CHANN	1987	2715	3286	3685	4249

	E4	79	98	124	140	165
	PIPE	69	86	109	120	140
	P1A	139	182	219	243	279
	COMB12	189	247	300	331	380
	COMB13	2175	2962	3583	4014	4626
	P1B2	32	40	49	55	64
	COMB14	2197	2991	3617	4052	4669

The overall summation of the discharges at proposed condition is higher than that of the existing condition. This means that the development of the project area produces an increase in runoff that has to be mitigated as required by Puerto Rico Planning Board Regulation No. 3.

IV. RUNOFF DISCHARGE MITIGATION ANALYSIS

The development of the site will increase the runoff discharge. The Puerto Rico Planning Board Regulation No. 3 requires a flow mitigation structure wherever an increase in discharge is produced. Therefore, several flow detention structures will be included in the project.

Methodology

The computer program HEC-1 provides means for routing a hydrograph through detention structures. The purpose of detention is that the proposed condition peak discharge does not exceed the existing condition peak discharge.

Runoff Mitigation Structures

Runoff discharge mitigation for the whole project will be made by detention ponds located at the following areas: Area P2B, P2A, P1F1, P1D and P1A as shown in Figure 8.

Depth-Volume Relations

Volume-depth relations developed for the mitigation system is based on ponds with a 595-square-meter base area for Pond P2B, 1570-square-meter base area for Pond P2A, 392-square-meter base area for Pond P1F, 594-square-meter base area for Pond P1D, and 1,338-square-meter base area for Pond P1A. Bank slope for all ponds will be of 2H:1V. Volume-Depth curve computations are included in Appendix D.

Flow Rating Curve

Flow–Depth relation for all ponds was estimated according to the control structures assigned. The control structure for detention Pond P2B is composed by three 2.5’-diameter orifices located at the bottom of the pond, three 2.3’-diameter orifices located at the 1.5 m above el bottom of the pond and a 7.31-meter long weir at 2.70 meter over the pond bottom. The control for Pond P2A is composed by a 2.4’-diameter orifice located at the bottom of the pond and a 9.14-meter long

weir at 2.8 meter above the pond bottom. For the Pond P1F, the control consists of two 3.3'-diameter orifices located at the bottom of the pond, two 1.5'-diameter orifices located at the 1.8 m above the bottom of the pond and a 3.04-meter long weir at 2.7 meter above the pond bottom, For Pond P1D, the control consists of two 2.4'-diameter orifices located at the bottom of the pond, two 2.0'-diameter orifices located 1.7 m above the bottom and a 5.48-meter long weir at 2.8 meter above the pond bottom, And for Pond P1A: three 2.50'-diameter orifices located at the bottom of the pond, three 2.5'-diameter orifices located 1.5 m above the bottom of the pond, and a 7.31-meter long weir at 2.8 meter above the pond bottom. Flow through the orifices was computed using Torrecelli's formula. Appendix D shows the computations for the flow-depth relationship and the curves.

Results

The results of the detention analysis show that the proposed detention pond provides appropriate runoff mitigation for the 100-year frequency discharges. By routing the combined discharge through the detention pond, the 100-year discharge was reduced from 4,669 cfs to 4,478 cfs, which is less than the discharge computed for existing condition, 4,584 cfs; thus complying with Regulation No.3. Mitigation for more recurrent discharges was also verified.

Input and output data for the HEC-1 mitigation model are included in Appendix E. Table 4 shows the comparison of the discharges for existing, and proposed condition.

Table 4
Peak Discharges results for Existing, Proposed and Mitigation

AREA		Peak Discharge, cfs				
		2-yr	**10-yr**	**25-yr**	**50-yr**	**100-yr**
Existing	P1B	36	49	62	71	84
	P1C	61	79	100	115	136
	COMB1	96	127	162	185	219

	P2C	81	109	134	151	175
	CHANN	80	107	133	149	174
	P2B	178	233	288	324	377
	CHANN	173	224	275	307	356
	P2A	80	99	126	142	169
	CHANN	72	92	117	131	155
	COMB2	297	390	478	538	626
	E0	871	1184	1436	1610	1856
	CHANN	869	1181	1437	1612	1853
	E1	458	608	744	835	967
	COMB3	1287	1743	2116	2370	2725
	COMB4	1552	2113	2570	2883	3333
	CHANN	1538	2099	2554	2866	3314
	E3	100	125	157	177	209
	CHANN	92	118	148	166	196
	P1F	102	130	162	183	214
	COMB5	194	248	310	349	410
	COMB6	1664	2272	2758	3092	3570
	CHANN	1653	2262	2745	3079	3555
	E2	218	277	346	389	456
	CHANN	201	262	325	365	428
	P1E	158	215	261	293	338

	COMB7	329	439	537	602	696
	COMB8	1960	2677	3244	3637	4194
	CHANN	1928	2647	3207	3597	4151
	P1D	91	124	156	178	209
	COMB9	1994	2739	3319	3722	4294
	CHANN	1959	2702	3278	3677	4245
	E4	79	98	124	140	165
	CHANN	72	88	111	125	143
	P1A	144	184	229	258	302
	COMB10	216	272	340	383	445
	COMB11	2323	2920	3543	3973	4584
Proposed	P1B1	44	57	69	77	89
	P1C1	51	65	79	89	103
	COMB1	95	121	148	165	191
	P2C	58	72	89	100	116
	CHANN	56	69	83	94	109
	P2B	191	241	296	331	383
	CHANN	183	232	281	314	361
	P2A	109	138	169	189	220
	CHANN	104	133	163	182	211
	COMB2	338	428	518	580	668
	E0	871	1184	1436	1610	1856
	CHANN	869	1181	1437	1612	1853
	E1	458	608	744	835	967
	COMB3	1287	1743	2116	2370	2725
	COMB4	1593	2150	2611	2930	3384
	CHANN	1579	2136	2595	2913	3363
	E3	100	125	157	177	209

	PIPE	95	121	151	170	199
	P1F1	56	70	87	97	113
	COMB5	143	184	226	253	295
	COMB6	1686	2279	2766	3102	3578
	CHANN	1664	2259	2741	3073	3547
	P1F2	48	61	74	83	97
	COMB7	1700	2306	2797	3135	3618
	CHANN	1687	2294	2782	3119	3601
	E2	218	277	346	389	456
	CHANN	201	262	325	365	428
	P1E	92	116	142	159	185
	COMB8	291	369	457	512	596
	COMB9	1919	2599	3148	3525	4065
	CHANN	1887	2570	3113	3489	4025
	P1D	121	152	187	209	243
	COMB10	1970	2680	3243	3633	4189
	CHANN	1955	2667	3227	3617	4170
	P1C2	63	79	97	108	126
	COMB11	2010	2737	3313	3713	4280
	CHANN	1987	2715	3286	3685	4249
	E4	79	98	124	140	165
	PIPE	69	86	109	120	140
	P1A	139	182	219	243	279
	COMB12	189	247	300	331	380
	COMB13	2175	2962	3583	4014	4626
	P1B2	32	40	49	55	64
	COMB14	2197	2991	3617	4052	4669
Proposed w/ Mitigatio	P1B1	44	57	69	77	89
	P1C1	51	65	79	89	103

	COMB1	95	121	148	165	191
	P2C	58	72	89	100	116
	CHANN	56	69	83	94	109
	P2B	191	241	296	331	383
	POND	157	230	273	305	359
	CHANN	157	217	267	293	340
	P2A	109	138	169	189	220
	POND	48	57	69	90	136
	CHANN	48	57	68	86	129
	COMB2	250	337	406	446	541
	E0	871	1184	1436	1610	1856
	CHANN	869	1181	1437	1612	1853
	E1	458	608	744	835	967
	COMB3	1287	1743	2116	2370	2725
	COMB4	1537	2074	2509	2806	3266
	CHANN	1535	2072	2508	2805	3264
	E3	100	125	157	177	209
	PIPE	95	121	151	170	199
	P1F1	56	70	87	97	113
	COMB5	143	184	226	253	295
	POND	135	167	204	225	257
	COMB6	1631	2211	2676	2996	3476
	CHANN	1628	2210	2671	2989	3474
	P1F2	48	61	74	83	97
	COMB7	1654	2246	2712	3034	3524
	CHANN	1652	2245	2711	3033	3528
	E2	218	277	346	389	456
	CHANN	201	262	325	365	428

	P1E	92	116	142	159	185
	COMB8	291	369	457	512	596
	COMB9	1838	2513	3045	3409	3927
	CHANN	1823	2482	2997	3356	3867
	P1D	121	152	187	209	243
	POND	91	122	148	164	190
	COMB10	1908	2595	3145	3520	4057
	CHANN	1906	2589	3120	3493	4029
	P1C2	63	79	97	108	126
	COMB11	1948	2646	3205	3589	4138
	CHANN	1944	2644	3185	3559	4113
	E4	79	98	124	140	165
	PIPE	69	86	109	120	140
	P1A	139	247	300	331	380
	COMB12	189	247	300	331	380
	POND	151	230	270	297	335
	COMB13	2094	2874	3456	3856	4448
	P1B2	32	40	49	55	64
	COMB14	2109	2896	3480	3883	4478

Mitigation Structure Dimensions and Accessories

Final dimensions for the detention pond will include a minimum free board of 0.6 meters. The detention pond will have the characteristics shown in Tables 5 through 9

Table 5
Detention Pond P2B Characteristics

Pond P2B	Dimensions	Bottom Area	595 m^2
		Height	3.47 m
	Outlet Control	Orifice @ bottom	Three-2.5'Diameter
		Orifice @ 1.50 m bottom	Three-2.3'Diameter
		Weir Length @ 2.70m	7.31 m

Table 6
Retention Pond P2A Characteristics

Pond P2A	Dimensions	Bottom Area	1570 m^2
		Height	3.63 m
	Outlet Control	Orifice @ bottom	One-2.4'Diameter
		Weir Length @ 2.80m	9.14 m

Table 7
Retention Pond P1F Characteristics

Pond P1F	Dimensions	Bottom Area	392 m^2
		Height	3.30 m
	Outlet Control	Orifice @ bottom	Two-3.3'Diameter
		Orifice @ 1.80 m bottom	Two-1.5'Diameter
		Weir Length @ 2.70m	3.04 m

Table 8
Retention Pond P1D Characteristics

Pond P1D	Dimensions	Bottom Area	594 m^2
		Height	3.49 m
	Outlet Control	Orifice @ bottom	Two-2.4'Diameter
		Orifice @ 1.70 m bottom	Two-2.0'Diameter
		Weir Length @ 2.80m	5.48 m

Table 9
Retention Pond P1A Characteristics

Pond P1A	Dimensions	Bottom Area	1338 m^2
		Height	3.40 m
	Outlet Control	Orifice @ bottom	Three-2.5'Diameter
		Orifice @ 1.50 m bottom	Three-2.5'Diameter
		Weir Length @ 2.80m	7.31 m

Schematic layout of the detention system is presented in Figure 9, 10, 11, 12 and 13.

Pond Discharge

The Pond P2B will discharge through a 7'x 7' box culvert into a storm watercourse that ends up in the storm watercourse crossing the project site, Pond P2A through a 60" diameter pipe, Pond P1F1 through a 72" diameter pipe, Pond P1D through a 72" diameter pipe and Pond P1A through a 7'x7' box culvert. All pond discharges must be provided with a headwall and energy dissipater.

V. HYDRAULIC ANALYSIS

A hydraulic analysis of the storm watercourse was made in order to determine the water surface elevations and the extent of the floodplain. Also, the analysis will allow sizing of the crossing structures needed for the project roads.

The hydraulic analysis was made using the mathematical model HEC-RAS developed by the US Corps of Engineers. For this model, the hydraulic regime is steady, uniform, and one-dimensional. The model accepts changes in the geometry of the watercourse, bank-bed-overbank friction coefficient and shapes of hydraulic structures.

The friction coefficient used in the modeling was obtained from visual inspection of the watercourses bed and banks; and crosschecked with the typical values provided by Barnes (1967) and Chow (1959).

Cross sections for the creek were taken from the fieldwork provided. This fieldwork is shown in Appendix H. Figure 14 depicts the location of the cross sections.

Roughness

Manning coefficients estimated for the model are 0.013 for concrete, 0.045 for the channel and 0.045 for the over banks. These values reflect the changes in direction and the irregular shape of the channel bed, and the vegetation present in the courses.

Contraction and Expansion Coefficients

Coefficients of contraction and expansion used are those recommended by the HEC-RAS user's manual. Thus, coefficients of 0.1 and 0.3 respectively were used for gradual transitions.

Hydraulics for Existing Condition

Existing condition consists in the storm watercourse as it is at the present. The analysis was based on the cross sections of the watercourse and the discharges obtained from the hydrologic analysis. Rainfall events of 100, 50, 25, 10 and 2-years were considered in the analysis.

The hydraulic analysis was made starting at the point where the storm watercourse joins Río Canóvanas. The starting water elevation for the subcritical run was taken from the FEMA study, 47.5 m, msl. The discharge used for analysis was 4,584 cfs, which was obtained from hydrologic analysis. Flow regime for the existing condition was mainly sub-critical, and super-critical for the upstream sections.

The computer output for the existing condition is included in Appendix F. The following Table 10 includes the summary of the results.

Table 10

Existing Condition Hydraulics 100-yr

River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
		(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
12.5	100Y	12.91	71.00	72.22	72.09	72.31	0.011273	1.38	9.34	20.51	0.65
12	100Y	12.91	71.00	72.09	72.09	72.27	0.029024	1.90	6.80	18.90	1.01
11	100Y	117.55	64.41	65.74	65.92	66.38	0.043234	3.56	33.05	48.55	1.38
10	100Y	117.55	60.00	61.70	61.91	62.53	0.033473	4.05	29.04	28.96	1.29
9	100Y	117.55	57.00	59.01	59.17	59.72	0.030697	3.75	31.35	32.79	1.22
8	100Y	120.21	55.00	56.32	56.45	56.85	0.034992	3.24	37.07	53.71	1.25
7	100Y	120.21	52.99	54.58	54.58	55.06	0.020531	3.07	39.14	40.97	1.00
6	100Y	120.21	51.00	52.58	52.65	52.94	0.035841	2.68	44.89	87.88	1.20
5	100Y	120.21	48.00	49.79	49.45	50.08	0.007826	2.43	51.43	41.71	0.64
4	100Y	120.21	48.00	49.42		49.60	0.006613	2.02	65.62	64.79	0.59
3	100Y	120.21	47.00	48.53	48.53	48.77	0.014138	2.44	61.16	113.03	0.83
2	100Y	129.81	45.00	47.53	46.50	47.53	0.000142	0.40	363.42	231.94	0.09
1.6	100Y	129.81	45.00	47.50		47.52	0.000618	0.63	210.67	192.68	0.18
1.3	100Y	129.81	45.00	47.49		47.51	0.000632	0.63	209.06	192.08	0.18
1	100Y	129.81	44.00	47.50	42.58	47.50	0.000019	0.18	600.15	190.92	0.04
15	100Y	93.85	77.00	78.90	78.90	79.63	0.018952	3.77	24.88	17.38	1.01
14	100Y	100.67	74.00	74.74	75.17	76.17	0.091971	5.28	19.05	27.05	2.01
13	100Y	100.67	72.00	73.63	73.94	74.63	0.045422	4.44	22.68	24.58	1.47

Hydraulics for Proposed Condition

Proposed condition includes the project site developed. The discharges used were the ones resulting from the offsite discharges plus the runoff discharge from the site as mitigated by the ponds. Also, four (4) bridges are included. Geometry of these bridges is shown in Figures 16, 17, 18 and 19.

The computer output for the hydraulics analysis is included in Appendix G. The following Table 11 includes the summary of the results for the proposed condition.

Table 11
Proposed Condition Hydraulics 100-yr

River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
		(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
15	100Y	92.43	77.00	78.89	78.89	79.61	0.018976	3.76	24.61	17.32	1.01
14	100Y	98.38	74.00	74.73	75.15	76.14	0.093150	5.26	18.70	27.00	2.02
13.5		Bridge									
13	100Y	98.38	72.00	73.44	73.92	74.90	0.076270	5.36	18.35	22.09	1.88
12.5	100Y	12.12	71.00	72.20	72.07	72.29	0.011264	1.35	8.95	20.27	0.65
12	100Y	12.12	71.00	72.07	72.07	72.25	0.028805	1.85	6.54	18.73	1.00
11	100Y	109.51	64.41	65.78	65.87	66.27	0.031151	3.10	35.30	49.76	1.18
10.5	100Y	109.51	60.00	63.12	61.85	63.21	0.001658	1.37	80.02	42.36	0.32
10.3		Bridge									
10	100Y	109.51	60.00	61.32	61.85	63.03	0.097280	5.79	18.91	24.56	2.11
9	100Y	109.51	57.00	59.11	59.11	59.61	0.020173	3.15	34.76	34.46	1.00
8	100Y	114.10	55.00	56.17	56.42	56.92	0.052659	3.83	29.83	45.80	1.51
7.5	100Y	114.10	52.99	56.09	54.54	56.14	0.000834	0.94	121.69	68.14	0.22
7.3		Bridge									
7	100Y	114.10	52.99	54.02	54.54	55.93	0.163983	6.12	18.63	33.00	2.60
6	100Y	116.48	51.00	52.64	52.64	52.91	0.024543	2.31	50.52	93.20	1.00
5	100Y	116.48	48.00	49.77	49.43	50.06	0.007711	2.39	50.60	41.50	0.64
4	100Y	116.48	48.00	49.40		49.58	0.006501	1.99	64.67	64.74	0.58
3	100Y	116.48	47.00	48.51	48.51	48.75	0.014232	2.44	59.59	112.73	0.83
2	100Y	126.81	45.00	47.60	46.48	47.60	0.000117	0.37	380.12	232.18	0.08
1.6	100Y	126.81	45.00	47.57	46.47	47.59	0.000455	0.58	225.80	198.23	0.16
1.5		Bridge									
.3	100Y	126.81	45.00	47.49		47.51	0.000571	0.62	209.12	192.10	0.18
1	100Y	126.81	44.00	47.50	42.56	47.50	0.000017	0.18	600.15	190.92	0.03

Results

The maximum increase in floodwater levels due to a hydraulic structure allowed by the Puerto Rico Planning Board Regulation No. 13 for rural areas is 0.30 meters. There is no increase in water level at the cross section located in the immediate upstream of the project site. Though within the project site, just upstream of the bridges, minor increases are seen; but none of them over the allowed by the PR Planning Board. However and since this increase is constrained within the project site, and no increase is reflected outside, the level increase requirement called by Regulation No. 3 is fully fulfilled.

Flood Plain Encroachment

In some areas, the floodplain of the storm watercourse crossing the site is too wide. Then, encroachment of the floodplain in some spot areas is needed in order to rescue some land for development. The results of the encroachment analysis indicate that no increment in water surface elevation above 0.30 meters is produced. The computer output for the encroachment analysis in included in Appendix G. And the watercourse floodplain/floodway is shown in Figure 15. No construction will take place in the floodway.

VI. CONCLUSIONS AND RECOMMENDATIONS

The following are the conclusions of this study:

1. According to the FEMA's Flood Insurance Rate Map, a very small part of the project site is located in the floodway of Río Canóvanas. This part will not be developed at this stage.

2. Proposed condition discharge is higher than that of the existing condition. Runoff mitigation is needed.

3. The proposed detention ponds reduce the increment in discharge due to the development from 4,669 cfs to 4,478 cfs which is lower than 4,584 cfs that belongs to existing condition.

4. The 100-year floodway footprint determined for the watercourse crossing the project site will not be affected by the development.

The following are the recommendations of this study:

1. The project site's roadway elevation must be at least 0.90 meters above the adjacent 100-year flood water elevation observed from the FEMA flood maps and from the HECRAS results on this study. And the minimum floor elevation will be 0.45 meters above the adjacent roadway elevation.

2. The detention ponds will have the dimensions indicated in Table 5 through 9 of this study. The discharge headwall from the ponds will be followed by rip-rap or gabions to prevent erosion of the downstream areas

3. The engineer will make sure that the proposed drainage areas be graded in a way that the drainage area for this basin conform the areas indicated in this study.

4. Maintenance of the road crossing structures and the runoff mitigation pond outlets is necessary during operation. Maintenance shall consist on keeping the entrance free from clogging debris or vegetation.

Study Limits

All the recommendations specified in this study must be considered to assure the optimum performance of the proposed structures operation. The design engineer will be responsible for elaborating the drawings in conformance with the recommendations of this study.

The results of this study are based on free flow conditions through the hydraulic structures. Proper maintenance must be developed to assure this condition. On the event of the occurrence of any severe obstruction to the flow, the results and recommendations may be impaired. Finally, the design engineer must use results and recommendations included in this report only and exclusively for the intended purposes as indicated in this study.

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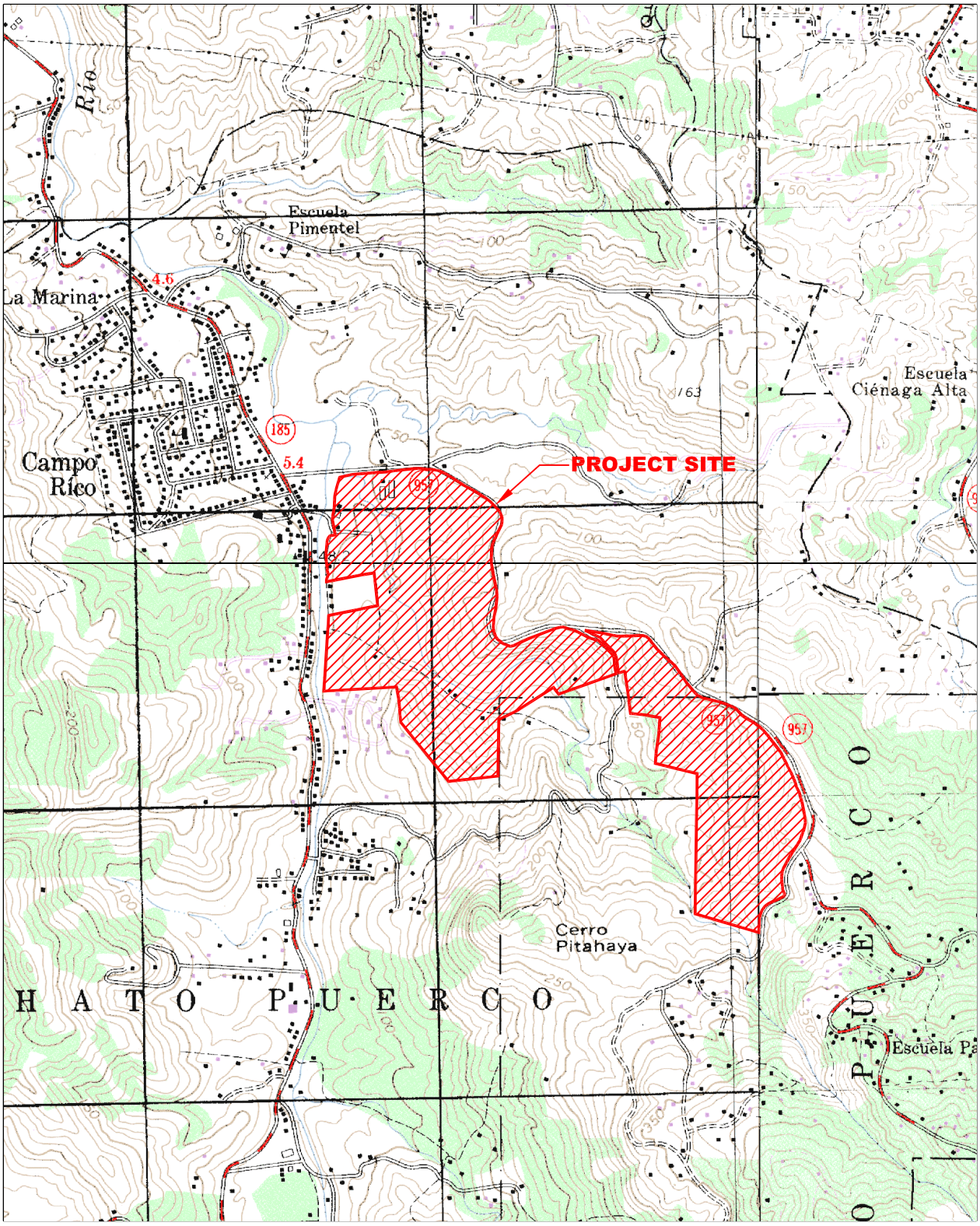
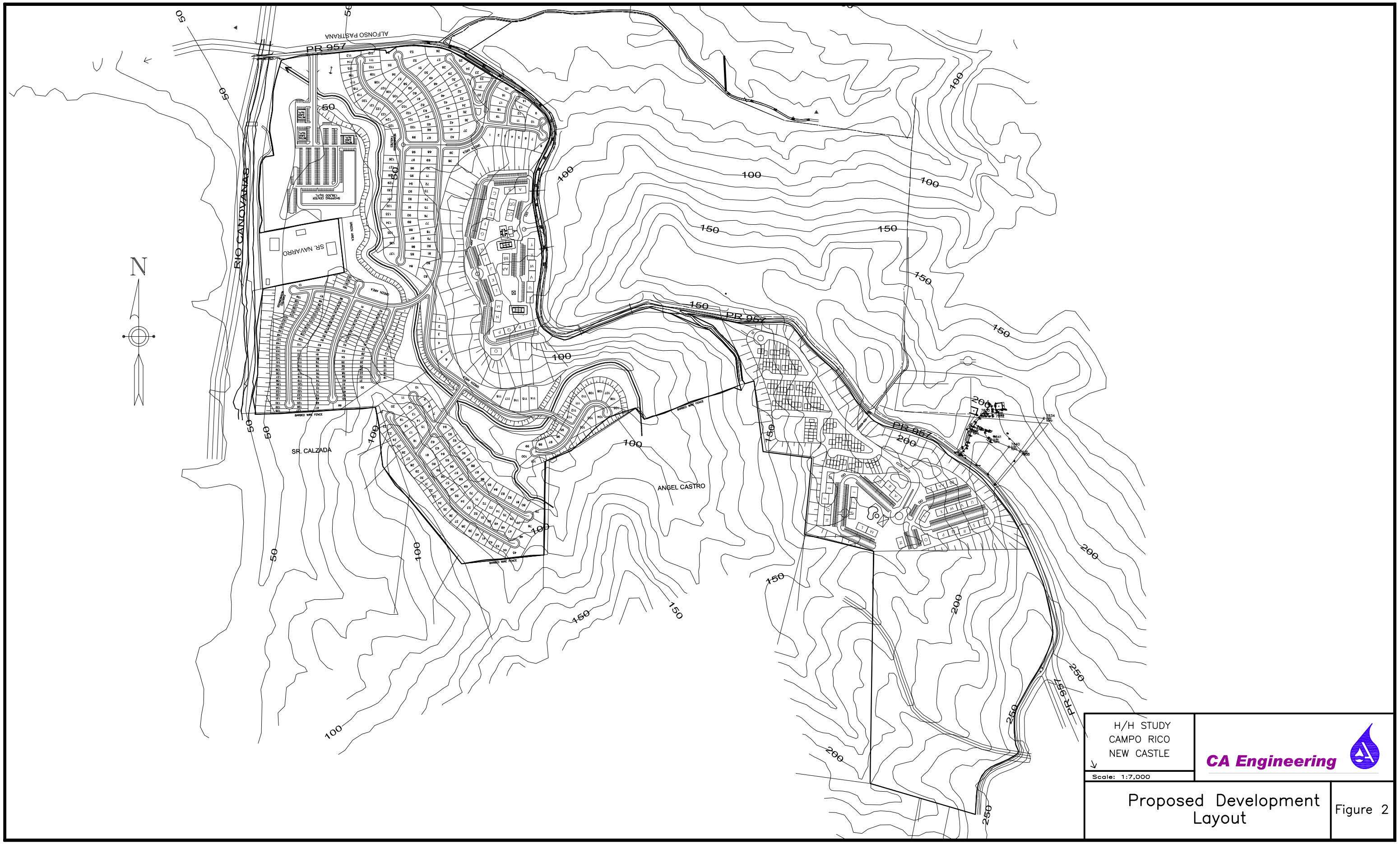
FIGURES



Partial Copy of USGS Quadrangle of Canovanas, P.R.

CA Engineering	H/H STUDY – CAMPO RICO NEW CASTLE, CANOVANAS, P.R.	Scale: 1:20,000
	Figure 1 Location Map	


N
PR 957
ALFONSO PASTRANA
RIO CANOVANAS
SR. NAVARRO
SR. CALZADA
ANGEL CASTRO
BARBED WIRE FENCE
H/H STUDY
CAMPO RICO
NEW CASTLE
Scale: 1:7,000
CA Engineering
Proposed Development Layout
Figure 2