Hydrologic -Hydraulic Study for an Urban Project Adjacent to a River with no Base Flood

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Abstract — *The massive land development increases the impervious areas of watershed, which induces high runoff and downstream flooding problems especially in areas adjacent to a river. It is extremely important to determinate a federal regulatory floodway in these areas in order to regulate the urban development without control in this zone to ensure that there are no increases in upstream flood elevations neither downstream. This paper intends to show the studies and the procedures required to conduct a large-scale urban project adjacent to the river where there is no established base flood in order to comply with existing regulations for this area.*

Key Terms — *hydrology and hydraulic studies, mitigation, detention pond.*

INTRODUCTION

It proposes the development of a planned community on a land with total area of 1.96 square kilometers. The proposed development includes the formation of 1000 housing units, two multifamily complexes, recreational facilities and the development of a private school, 9-hole golf course and linear public walkway.

The land to be developed, in the past was used for growing sugar. At the present this land is vacant and is occasionally used for livestock grazing. The alternative of agricultural use for these areas is not viable either economically and because of the lack of manpower for implementing it. The Municipality of San Lorenzo intends to supplement the loss of jobs in the traditional agricultural sector, with positive impact on the generation of jobs in the construction area.

The land for the proposed development is located on the west side of State Road PR-203 and North of the Río Grande de Loíza in the Navarro ward of the Municipality of San Lorenzo.

As on the FIRMs map (Rate Insurance Flood Maps) identified as 72000C1230H, prepared by the Federal Emergency Management Agency, the area to be developed is affected by floodway area that consists of an area variably wide along the length of the South boundary of the terrains along the Río Grande de Loíza. This fringe is identified in the map as Zone A. As mentioned in the Regulation of Planning Board No.13, Section 7.02, where areas classified as Zone A, if there is no detailed study available, then a hydrologic hydraulic study must be performed in order to determine the level of the base flood. In order to be viable the impact of the project in the existing levels of water must not be higher than 0.30 meters in a rural zone.

For this reason a hydrologic hydraulic study was performed in the unstudied area of the Rio Grande de Loíza. Once the base levels have been established we proceed to determine the flows generated by the existing condition and proposed condition to the urbanized area. The difference of both conditions is the flow that will be mitigated according to Regulation of the Planning Board No. 3, Section 14.04.

The mitigation will consist in the design of a detention pond located south of the project site. Once mitigation is performed, we need to verify that the proposed development does not change the existing base flood elevations.

Purpose of the Study

The purpose of this study is to determine the "Regulatory Floodway" as defined by the Federal

Emergency Management Agency (FEMA), the channel of a river or other watercourse and the adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation more than a designated height. Surface water levels will be determined for a frequency of 100-year flood.

Approach

The following steps have been undertaken throughout the study:

- Hydrological analysis of existing condition in the unstudied area of Río Grande de Loíza. This includes the determination of the hydrological parameters and based on these, the determination of the discharge for 100-yr frequency storm. HEC-HMS model [1] was used for this purpose.
- Hydraulic analysis of existing condition to determine the base flood levels. HEC-RAS model [2] was used in this case.
- Hydrological analysis of existing and proposed condition of the site drainage basin. Discharge will be determined for 2, 25 and 100 year storm event. HEC-HMS model was used.
- Runoff Mitigation analysis was made in order to counteract the impact of the proposed development. A mitigation pond was employed as a detention structure. HEC-HMS model was used for the mitigation analysis. Was analyzed for 2, 25 and 100 year storm event.
- Confirm that the mitigated proposed project with mitigation does not alter the existing base flood levels. HEC-RAS model was used for this purposed.

Description of the Study Area

This section provides a brief description of the basin area including its location, topography, water bodies and flooding problems.

Location

The Río Grande de Loíza has the largest watershed in Puerto Rico with an area of 803 square kilometers at its mouth**.** The Munucipalities of Juncos, San Lorenzo, Las Piedras, Gurabo, Canóvanas, Carolina and Loíza Aldea lie within the Río Grande de Loíza watershed. Figure 1 shows the drainage area (practically it is in the Municipality of San Lorenzo) contributing to the study reach (indicated in the figure1 as A-B) of the Río Grande de Loíza located in Navarro Ward at San Lorenzo Municipality.

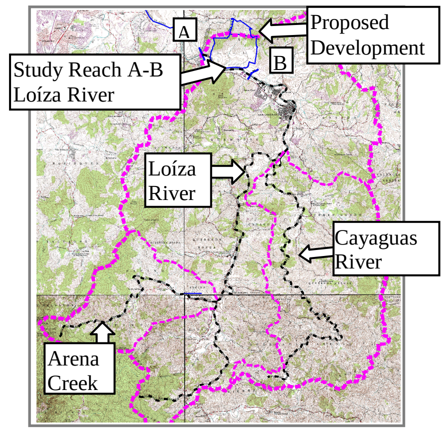

Figure 1
Total Watershed of the Study Area

Also shown, is the proposed development area which is located just north of the study reach of the Río Grande de Loíza and on the west side of State Road PR-203.

Topography

The drainage area of the studied reach consists of 134 square kilometers and its location is mostly in the Municipality of San Lorenzo. The terrains are predominantly of rugged topography. These cover approximately 75% of the total territory. Their area is characteristically steep with pronounced slopes and elevations that reach up to 640 meters over mean sea level in the West portion of the drainage area. These areas are different from the urbanized areas and the valleys nearby the river, which have lighter slopes. The control point of the basin is shown in Figure 1 with letter A.

Water Bodies

The main water body in this area is the Río Grande de Loíza which has two tributaries; these are the Cayaguas River and the Arena Creek.

Flooding

In the study reach A-B there are no elevations shown on the map because this reach has not been the subject of a hydrology hydraulic study.

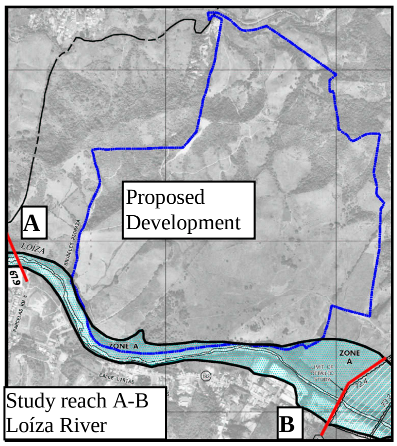

Figure 2
Flood Map

However, "FEMA" Federal Agency Emergency Management delimited the flood zone with a fringe as shown in figure 2. This area is known as Zone A, which indicates that the risk of flooding is 100 year storm event. There are two control points; upstream of the project identified as point B in Figure 2, where "FEMA" determined the base flood level of 72.4 meters and downstream of the project identified as point A, where a private firm determinate a base flood of 67.9 meters.

The flood area is to the South of the proposed development and affects only a small portion of it, practically it is unseen because the proposed urbanization areas do not contemplate any other development inside this zone. Despite this, a study is realized to delimit the water levels that correspond to a 100 year event. These levels will be used as part of the design of the proposed project.

PRELIMINARY EVALUATION

A visit to the project site was made on December 6, 2009. At this time the river bed, banks, and general area were assessed. The river channel seemed to be stable, varying in width from about 90 to 120 feet. The river bottom and banks consisted of a coarse sand, gravel, and stones at various places. At the time of the visit the river was flowing very shallow (less than two feet deep at most places observed). The banks were profuse in vegetation, which consisted of grasses of various types, bamboo, shrubs and trees.

Former Studies

The Río Grande de Loíza has been subject to various flood studies; here the mentioned reports will only be the ones that involve of the Municipality of San Lorenzo and areas that are next to the study.

This study was prepared by the FEMA to revise and update a previous “Flood Insurance Study for Río Grande de Loíza Basin, Puerto Rico” that was published on July 25, 1980. Current FEMA flood maps are based on this study. Peak 100-year flow at the USGS Gaging Station was estimated in this report at 3,280 cubic meters, while at San Lorenzo, it was estimated at 2,260 cubic meters per seconds. The proposed development lies between these two points but closer to San Lorenzo station.

Historic Events

Major floods of the Río Grande de Loíza are known to have occurred since 1899. The 1899 flood is believed to have been a very large flood. However, reliable data on discharge and stage corresponding to this flood has not been available. The same is true of floods that occurred in 1932 and 1943.

Flow data at USGS Gaging Station 0550, near PR-30, at Caguas, downstream of the study reach is available for the 1960-1994 period, meaning 35 consecutive years. Additionally, it is known that in

1945, a peak discharge of 2,407 cubic meters was estimated at this station.

HYDROLOGY ANALYSIS

This study was performed for a recurrence of 100 years. Peak flows determined in this model will be used in a hydraulic model to determine water levels in the study reach of the Río Grande de Loíza.

Methodology

The Hydrologic Modeling System (HEC-HMS) created by USA Army Corps was used for this study, which is designed to simulate the precipitation runoff processes of dendritic watershed systems. It is designed to be applicable in a wide range of geographic areas for solving the widest possible range of problems. This includes large river basin and flood hydrology, and small urban or natural watershed runoff.

Watershed

Basin study area was delineated using the topographical quadrangles of USGS of Caguas, Juncos, Yabucoa and Patillas as shown in figure 1. It practically includes the Municipality of San Lorenzo and its drainage area is of 133 square kilometers approximately. It consists of three sub-basins; Quebrada Arenas, Rio Cayaguas and Río Grande de Loíza at the height of Carraízo, its areas are of 25.9, 15.5 y 91.6 square kilometers respectively. The sum of these sub-basins adds up to the total capacity of the primary basin.

Land Cover

Land Cover determination was made by using PR Online GAP Data Explorer Tool (www.gapserve.ncsu.edu) which is an Open Source Freeware solution to online GIS applications. It was created by the Biodiversity and Spatial Information Center at North Carolina State University. This data is based 2001 datasets. The summary of the land cover report indicate the watershed consists of 39 % of forest areas, 54% grassland and pastures and 6% urban areas.

Soils and Curve Number

Most of the watershed is comprised of soils with high runoff characteristics as shown in Table 1. About 70% consists of Pandura, Múcara, Caguabo, and Juncos soils on steep to very steep slopes. These are classified as Hydrologic Soil Group D. The remaining 30% of the watershed consists of soils type B which is about 18%, and C about 13%. Type B soils include Lirios, Limones, and Jagüeyes. Type C soils include Los Guineos, Naranjito, Cayagua and Candelero. Soils were identified using the Soil Conservation Soil Survey of the San Juan Area [3].

Table 1
Curve Number and Soil Type for Existing Condition

Soil Type	Hydrologic Group	Area Km^2	Cover Description	CN
LrE2 LeE2 JgE2	B	5,911	Pasture, grassland fair cond.	69
LsD,LsE2 NaE2,NaF2 CdC2,CgC2	C	4,218	Woods fair cond.	73
PaE2 MuE2 CbF2	D	22,794	Pasture, grassland fair cond.	84

A weighted Runoff Curve Number of 80 (Antecedent Moisture Condition II) was determined based on soil type and cover. Runoff Curve Numbers were taken from Table 2-2c of Technical Release 55, TR-55 [4].

Rainfall

The rainfall duration used for this study was 24 hours. The rainfall distribution used was Soil Conservation Services Type II – 24 hours. There are four rain gage stations within the study watershed with point precipitation Frequency estimates. An average of them was used to obtain the precipitation frequency for this area. These precipitations were obtained from the NOAA National Weather Service, ATLAS 14. The Point rainfall estimates obtained from NOAA represent values for areas up to 25.8 square kilometers; therefore, a depth-area adjustment should be applied to the rainfall data when watershed area is greater. In this case it was applied because the watershed consists of 134 square kilometers. Table 2 show the precipitation of different design interval with its corresponding factors (from Figure 4-5, Technical Paper No. 42) to obtain average rainfall over the watershed [5]. Rainfall losses such as vegetative interception, depression 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 losses are incorporated in the model a function of the curve number of the watershed.

Table 2
Precipitation Frequency Estimates
(Annual Maximum)
24 Hour Rainfall Duration

Recurrence Interval	Precipitation (mm)	Adjust Factor	Precipitation (mm)
2	136	0.95	129
25	350	0.95	333
100	493	0.95	468

Time of Concentration

Time of concentration is a fundamental watershed parameter. The peak discharge is a function of the rainfall intensity, which is based in time of concentration. Time of concentration is time required for a drop of water to travel from the most hydrologically remote point in the sub catchment to the point of collection. In this study were used tree equations for determinate the time of concentration of the different stage of flow. These equations are Manning Kinematic, Ratio of flow length to flow velocity and Manning's. These are shown in Table3.

Results of Hydrologic Analysis

The summarize results of the hydrologic analysis for different recurrence interval are shown in Table 4. The results of the hydrologic model at the outlet point were compared with a study of FEMA mentioned in this document.

Peak 100-year flows at the project site determined by the hydrologic model (2,639m^3) are compatible with those estimated by FEMA. This agency determined peak flows of 2,260mt^{3}and 3,280mt^3 at San Lorenzo and Caguas, respectively. The study area lies almost at the middle of these two points. This tends to justify a peak flow at the project site, in the range of these two values.

Table 3
Time of Concentration

Equation	Stage of flow	Time Conc. (min)	Lag Time (min)
$T = \dfrac{\underline{.007(nL)^{0.8}}}{(P_2)^{0.5} s^{0.4}}$	Sheet flow	39.4	23.6
$Tt= L/3600V$; $V = S/.0039)^{0.5}$	Shallow flow	0.79	0.47
$V=\dfrac{\underline{1.49r.^{67} s^{0.5}}}{N}$	Channel Flow	281	169
Total		362	193

Table 4
Peak Discharges

Recurrence Interval (Years)	Peak Flows (M^3/s)
25	1,899
100	2,639
500	4,016

Verify this, it was used an interpolation was made to obtain the estimation of flow at the ungaged site (which is the project site). This can be done because there are records at two gaging station, one upstream (USGS 50051800 at Hwy 183 San Lorenzo with drainage area of 41 square kilometers) and downstream (50055000 at Caguas which drainage area of 232.6 square kilometers). The drainage area of the ungaged site is 133 square kilometers. With these areas and using the flows determined by FEMA at the San Lorenzo and Caguas station, it was used "1" to get an estimate of the flow in the study reach where;

$$\frac{Qy}{Ay} = \frac{Qx}{Ax} + \left(\frac{Qz}{Az} - \frac{Qx}{Ax} \right)\left(\frac{Ay - Ax}{Az - Ax} \right) \quad (1)$$

Qx is the flow at gaged site X of drainage area Ax; Qy is the flow at ungaged site Y of drainage area Ay; Qz is the flow at gaged site Z of drainage area Az and ($\tilde{Q}x$, $\tilde{Q}z$) is the average of entire record at X and Z.

The estimation of peak flow between these two gage stations was 2,638 mt^3which is almost the same the peak value obtained by the hydrological model. In this way it was established that the peak flow obtained by the model are correct and as a consequence their results can be used by hydraulic model to obtain the water surface of the study river reach.

HYDRAULIC ANALYSIS

This section presents information of a 100 yr simulated flood performed by the hydraulic model program (HEC-RAS) for the study reach.

Simulated Flood

The study area has no base flood, but downstream of the Project site (identified in figure 2 with letter A) a Private Firm made a hydraulic study where a base flood of 69.7 meters was determined just at the beginning of the reach study. At the end of the study reach (identified in figure 2 with letter B), FEMA determinate an elevation of 72.42 meters for this point. The project area lies between these two points. The elevation of 69.7 meter will be used in the model like a boundary condition downstream while FEMA elevation will be used to calibrate the hydraulic model.

Methodology

Water surface profiles presented in this report have been determined using the computer program HEC-RAS. This program is intended for calculating water surface profiles for steady, gradually varied flow in natural or man-made channels, and is capable of considering the effect of obstructions such as bridges, culverts and weirs in the determination of flood elevations. Also has the ability to perform subcritical, supercritical or mixed flow regime calculations all in a single execution of the program.

Layout of Hydraulic Analysis

The locations of cross-sections used in this study are shown in Figure 3.

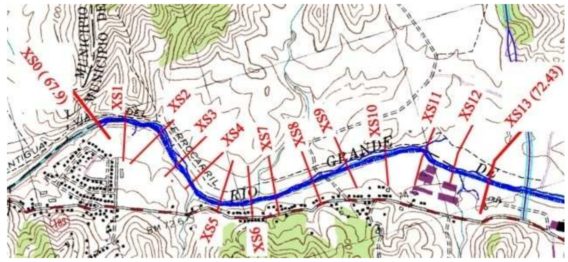

Figure 3
Cross Sections

A total of twelve cross-sections were taken from topographic maps made from aerial photographs. This could be possible because the map has details of river channel. Also field inspection helps support the information acquired from the map. Two cross sections were taken in the field at various locations to confirm the river channel elevations and geometry.

Hydraulic Roughness

Roughness coefficients (Manning's "n" values) were assigned based on field observations and review of aerial photographs [6]. A coefficient of 0.035 was used for the river channel and a coefficient of 0.06 was used for overbank flow. This value was used to calibrate the model to FEMA base flood elevation upstream the study reach.

Peak Discharge

The peak discharge for the 100 year storm design used in the Hec-Ras model runs was t determined in the hydrologic part of this report. These values are shown in Table 4.

Results of Hydraulic Analysis

Results of the Hec-Ras program show the following water surface elevations for the existing condition. The elevation of the cross section (0) was the one determined by the Private Firm and the elevation of the cross section (13) was determined by FEMA.

Table 5
Water Surface Elevations
for 100 year Design Storm

Cross Sections	Water surface Elevations	Cross Sections	Water surface Elevations
0	67.9	7	69.65
1	68.49	8	70.55
2	69.39	9	70.65
3	69.37	10	70.53
4	69.31	11	71.16
5	69.26	12	72.42
6	69.02	13	72.43

These are validated because they were calibrated with FEMA base flood elevation upstream. These elevations are important because the grading design of the urbanized area must be above these elevations.

ANALYSIS OF PROPOSED AND EXISTING CONDITION

The purpose of analyzing both conditions is to determinate and compare the peak discharges because the difference must be mitigated. This analysis would be done for 2, 25 and 100 year storm events.

Methodology

For this analysis it was also used HEC-HMS model to determinate the peak discharge flow for both conditions.

Existing Condition Description

The land that will be developed are located just north of the study river reach and west of road PR-203 in the Municipality of San Lorenzo. This property consists of an area of 1.96 square kilometers. As shown in Figure 4. The watershed was delimited using USGS topographic quadrangle. The drainage area was divided in two sub-basins, “Ba” and “Bb” with areas of 0.85 and 1.11 square kilometers respectively. These areas are two existing intermittent creeks unnamed which end up at watercourse discharging into Río Grande de Loíza.

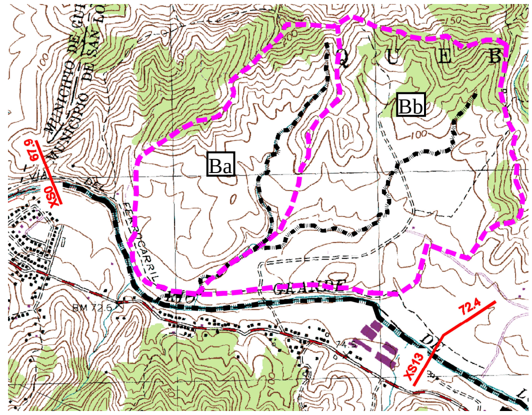

Figure 4
Existing Condition Basin Areas

The whole land of this property discharges into these two creeks. The topography is around one third mountainous with steep slope and the remaining have gentle slopes.

The land cover for this area is 25% forest and 75% pasture and grassland.

These two sub-basins comprised soils with high runoff like the main watershed. About 96% consist of Mucara, Mabi, Caguabo and Juncos. These are classified as Hydrologic Soil Group D. The remaining 4 % consist of soil type B which is Vivi. A weighted Runoff Curve Number of 79 (Antecedent Moisture Condition II) was determined based on soil type and cover for both drainage area

Proposed Condition Description

The proposed residential development also consists of 1.96 square kilometers. For the simulation of the proposed condition the site was subdivided into four drainage area “B1”, B2, B3 and “B4” with areas are 0.64, 0.21, 0.58 and 0.53 square kilometers respectively. The areas B1 and B2 are located into existing drainage area “Ba” while B3 and B4 are in drainage area “Bb” as shown in Figure 5.

The general drainage pattern of the property will not be altered because the creeks areas remain like their existing condition draining south of the site to the river. Their channel will not be altered and will retain green stripes in their natural state for conservation of all existing vegetation along them.

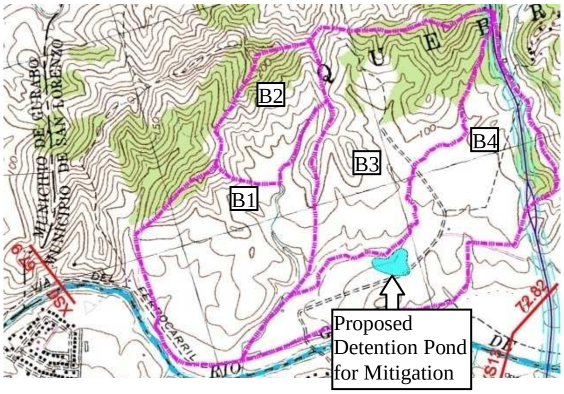

Figure 5
Proposed Drainage Areas

The storm water system of the project will consist of pipes and catch basins which collect the flow runoff prior to discharge it in the creeks. The condition development contemplates the development approximately 50 % of the land; the rest will remain like green areas. The percent of impermeability of the proposed basins due to these development areas are shown in the Table 6 and also the Weighted Runoff Curve Number. For the development area CN of 90 was estimated, while for green areas including forest and pasture it was 80 based on soil type (hydrology soil group D) and land use. The increase of flow caused by the development will be mitigated in a detention pond.

Table 6
Existing and Proposed Weighted Runoff Curve Number

Exist. Basin	Area Km2	CN	Prop. Basin	Area Km2	CN	%Imp
Ba	0.85	79	B1	0.64	85	50
			B2	0.21	82	25
Bb	1.11	79	B3	0.58	86	50
			B4	0.53	82	25

Lag Time

In this study SCS Lag Time method was used to obtain the lag time for both conditions: existing and proposed, this one is defined by "(2)"; where T_{lag} is lag time in hours; L is hydraulic length of watershed in feet; S is maximum retention in the watershed in inches and Y is watershed slope in percent.

$$Tlag = \frac{(L^{0.8})(S+1)^{0.7}}{1900\sqrt{Y}} \qquad (2)$$

Table 7 shows the lag time for the analyzed watersheds under existing and proposed conditions. Watershed was routed downstream to the watershed exit.

Table 7
Lag Time Estimates - Proposed and Existing Condition

Existing Condition			Proposed Condition		
Basin	Length (mts)	Lag time time(min)	Basin	Length (mts)	Lag time time(min)
Ba	1,850	1.25	B1	1,723	1.4
Bb	2,646	1.97	B2	1,655	1.17
			B3	2,426	1.8
			B4	2,140	1.68

Is important to note that the difference in lag time of the existing and proposed condition is low because only the runoff caused by the development is controlled by pipes to the discharge point which is nearest to the creek. The creek channel is kept in existing condition so its lag time just increases in sections where it receives flow from urbanized areas. Also the routed length of the urbanized area is less compare to the existing condition.

Rainfall

The rainfall duration used for this study was also 24 hours. The rainfall distribution used was Soils Conservation Services Type II – 24 hours. The rainfall data for this area was obtained by NOAA and are shown in Table 8. The depth-adjustment factor for these basins was not necessary because their areas are less than 10 square miles.

Table 8
Precipitation Frequency Estimates (Annual Maximum) 24 Hour Duration Rainfall

Recurrence Interval, yr	Precipitation mm
2	130
25	336
100	475

Results of Hydrology Analysis

Table 9 presents the peak discharges at different recurrence intervals generated by the analyzed watersheds under existing and proposed conditions obtained using HEC-HMS model.

Table 9
Peak Discharges

Area	Peak Discharge, Mt^3/s		
Existing Condition	2yr	25yr	100yr
Ba	6	19.4	29
Bb	6.3	21.0	31
Total	11	37.5	56
Proposed Condition			
B1	5.4	15.7	22.5
B2	1.8	5.6	8.1
B3	129	10.3	14.7
B4	3.7	12.2	17.7
Total	14.3	42.2	60.9
Difference of both Conditions	**3.17**	**4.7**	**5.5**

It is important to note that the increase of discharge due to post-development is not so high compared to pre-development; this is because the soils in their existing condition generate high runoff in itself. However, the increase in runoff should be mitigated as required by the Planning Board of Puerto Rico Regulation No. 3.

RUNOFF MITIGATION ANALYSIS

The development of the site will increase the runoff discharge. Therefore, a flow detention structure will be design for this project in order to hold storm runoff back but release it continuously at an acceptable rate through a flow limiting outlet structure, thus the downstream peak flow will be the same or less than the flow in its existing condition.

Methodology

The computer program HEC-HMS provides means for routing a hydrograph through detention structures. The purpose of this procedure is to insured that new development, with detention, will not cause any adverse impacts on existing flooding conditions downstream. Mitigation at area "B4" as show in Figure 5, will contribute in reducing the combined peak discharge of the overall project drainage areas below the discharge at existing condition. This mitigation will be analyzed for 2, 25 and 100 year storm event of 24 hours duration.

Runoff Mitigation Pond

The detention pond evaluated in this study will mitigate the increment in discharge from the site. This pond will be located south of the project site. Figure 5 shows its location.

Depth-Storage Relationship

Volume-depth relations developed for mitigation are based on a trapezoidal pond with an area of 16,670 square meters and a depth of 3.05 meters [7]. Its side slope ratio will be 2H: 1V. The bottom elevation of the pond is 72.5 meters. Free board of 0.45 meter shall be maintained during the 100yr storm event.

Flow Rating Curve

The routing hydrograph for the detention pond was estimated considering the discharge through one 0.45 meter diameter orifice located at the bottom, three 0.91 meter diameter orifices located at 0.80 meter above the bottom and a 3.96 meter long weir with 0.91 meter depth at 2.1 meter above the bottom.

Mitigation Results and its Corroboration

The results of the detention analysis show that the proposed detention pond provides appropriate runoff mitigation for 2, 25 and 100 year frequency discharges.

Table 10
Peak Discharge of Different Conditions

Condition	Peak Discharge, M^3/s		
Area	2yr	25yr	100yr
Existing	11.1	37.6	55.6
Proposed	14.3	42.3	61.0
Mitigation	11.0	36.8	55.5

Table 10 shows that the total discharges of the project in different recurring intervals do not exceed the peak discharge in its existing condition;

thus complying with Regulation No.3, even though it is necessary to prove that the proposed project will not alter the existing water levels in the river. It was used the existing run of the river made with HEC-RAS model to ensure that water levels do not alter with the mitigated discharge. The only difference is that now the flow of the mitigated project (56 m^3/s) will be added in the cross section number 6, which is the discharge point of the proposed area.

The results obtained adding this flow can be seen in Table 11.

Table 11
Water Surface Elevation including Proposed Development with Mitigation (100 yr Design Storm)

Cross Sect.	W.S.E Ext.	W.S.E Prop.	Cross Sect.	W.S.E Ext.	W.S.E Prop.
0	67.90	67.90	7	69.65	69.82
1	68.49	68.53	8	70.55	70.63
2	69.39	69.45	9	70.65	70.72
3	69.37	69.43	10	70.53	70.61
4	69.31	69.37	11	71.16	71.19
5	69.26	69.32	12	72.42	72.44
6	69.02	69.07	13	72.43	72.45

As observed there was a small increment in the water levels in the different sections. Only in section 7 we can observe that the flow of the proposed project is increased by 0.17 meters. In the rest of the sections the increase was between 0.02 y 0.08 meters. The Regulation allows 0.30 meter of increase in rural area. This way it is established that the proposed development complies with the Regulation whenever it does not exceed more than 0.30 meter above the existing levels of the river.

Conclusion

Once the base flood is established all elements of the project shall be constructed at a minimum of 0.30 meters above the water surface elevations for the 100-year flood shown on Table 10 to avoid flooding problems.

It is important to verify that the proposed mitigated project does not alter the existing water levels. And not only justify that the increasing flow generated by the development being mitigated will not change the base levels of the existing flood.

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