Extended Hydrologic & Hydraulic Study for the Recreational Pool Area of the Toro Negro Forest Reserve, Municipality of Orocovis, Puerto Rico

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Abstract — *An extended analysis of a recent study in the recreational pool area of the Toro Negro Forest Reserve, optimizing previously proposed geometric changes with additional components in order to increase public safety. Low Duration, High Intensity Rainfall Events are considered in addition to regulatory 24 hour duration events with the purpose of providing evacuation times for unpredictable flash floods using hydrodynamic routing and storage modeling as a tool to establish operational rules for the proposed system.*

Key Terms — *Flash Flood Reaction Time, High Intensity Rainfall, Hydrodynamic Routing, Short Duration Storm Events.*

INTRODUCTION

This Hydrologic-Hydraulic study is an expansion to an existing study [1] for a proposed repair and renovation of a once landmark site that was subject to closure as a consequence of heavy rains causing landslides into the recreational facilities, this evaluation was then necessary to determine the existing hydraulic conditions for the reach under analysis and the impact the proposed improvements could have on the Toro Negro Recreational Area and colliding properties.

Location

The reach under study is tributary to the Rio Inabón River which drains into the Rio Grande de Manatí River and is located around latitude: 18°10'15.78"N and longitude: 66°29'15.70"W in Toro Negro National Forest in the Municipality of Orocovis. At present time, the creek passes through the community of Barrio Ala de la Piedra crossing PR-143 at km 32.4.

Previous Study

The original analysis [1] performed by SALO Engineering PSC was approached in a regulatory set of mind taking emphasis on the impact the proposed changes by the architectural firm POLIS Group LLC would have on the 100-year return period 24-hour duration event in order to comply with Regulation 13 of the Puerto Rico Planning Board [12]. Additional 1-year and 2-year events were also performed.

Issues and Shortcomings

The following list enumerates some shortcomings of such an approach to the analysis of a recreational area located in such a region:

- The proposed changes raise the water surface elevation in the pool area by 0.14 meters.
- The proposed changes raise the water surface elevation by 0.19 meters just before the pool area in the water intake and sedimentation box.
- There are no measures present to provide warning for sudden short duration events with high intensity rainfall periods since the study only considers 24 hour durations.
- The analysis doesn't optimize the use of the proposed geometric changes. This includes the downstream pool formed from existing recreational pool's split-up into two smaller pools (One shallow upstream pool and a deeper downstream pool just before the existing spillway and sluice gate). This downstream pool could be used as part of a peak discharge damping mechanism.

Scope of Work (SOW)

The following tasks will be performed as part of this study:

- Verify hydrologic parameters in the previous study making sure they are up to date.

- Evaluate a feasible alternative to the one already proposed with the help CAD, HEC HMS [4] & Storm and Sanitary Analysis (SSA) [10], [11] routing software.
- Compare proposed alternative to original proposal.
- Verify if necessary the proposed alternative complies with Regulation #13 of the Puerto Rico Planning Board [12] by not experimenting rise in water levels by more than 0.15 meters in both upstream and downstream parcels.

PREVIOUS HYDROLOGIC STUDY

Hydrologic parameters of the studied area should not have changed drastically since September 2013 when this previous study was performed. Nonetheless a revision of the following data was performed to ensure accurate representation of the current conditions.

Rainfall Distribution & Precipitation Data

The Frequency Storm Method [2] used in this study, creates a balanced synthetic storm of a known excedance probability and was an appropriate decision since the often used SCS Type II Storm distribution [2] accumulates most of the rainfall on its peak time resulting in values that often exceed the Rational Methods [3] peak discharges for the same area under study. No other studies in the area were found. HEC-HMS [4] was used to interpolate the distribution from NOAA Atlas 14 [5] values for June 7, 2013. Precipitation depth values were again taken on August 25, 2014 to ensure validity of the previous study to the current condition. Since values did not change, previous study values are valid.

Watershed Delineation

The watershed for the existing and proposed conditions was prepared using the USGS Topographic Quadrangle of Orocovis. Drainage paths were established and along with site visits and satellite photography the watershed was validated then divided into its main causeways. The three resulting sub-basins adequately represent the current condition in present time and so are valid for this extended study. The main basin has a base elevation of approximately 875-meters and an approximate top elevation of 1055-meters mean sea level (MSL).

Land Use, Soil Type & NRCS Curve Number

Since the site location is situated in the middle of a National Forest Reserve, all sub-basins have a Forested Area Land Use exceeding 97% of their coverage area. The remaining coverage area is mostly recreational followed by agricultural and rural residential. Puerto Rico Planning Board GIS layers [13] indicate 100% of the soil in the watershed is LOS GUINEOS CLAY which consists of very deep, well drained soils on side slopes with very fine grain size and a hydrologic soil group of C. The resulting weighted NRCS CN [6] for the Toro Negro Recreational Area basin was calculated at 73 (same for all sub-basins).

Time of Concentration

The time of concentration was estimated using the SCS TR-55 Method [7] which applies to small urban watersheds and can be used with other type of watersheds if certain parameters are met. Sheet flow and shallow concentrated segments of the method contributed most of the Tc for all three sub-basins with the exception of sub-basin 1 where the first segment of open channel flow dominated by heavy brush and timber contributed 13.41 minutes almost 35% of its total 38.61 minutes. Because of the main watershed’s steep slopes along its talweg, well defined open channels with clear beds close to the recreational area don’t contribute any mayor increments in the total time of concentration for all three basins.

A simple use of the SCS Lag time equation [8] was used to validate the values of the previous study. Since this method was developed with agriculture in mind and does not contemplate specifics on the farthest and longest water drop path, values were expected to be lower than the ones obtained on the previous study using the SCS TR-55 method. Table 1 shows a brief summary of previous TR-55 values and SCS Lag time values. A site visit validated the parameters used in the TR-55 computations for Tc and so previous values are representative of the current condition.

Table 1
Summary of Time of Concentration Values

Basin ID	Area (ac)	C	i(10)	i(100)	Q_{10} (cms)	Q_{100} (cms)
1	66	1	4.75	6.70	8.89	12.54
2	20	1	4.75	6.70	2.65	3.74
3	12	1	4.75	6.70	1.62	2.29
Total	98	1	4.75	6.70	13.16	18.57

Peak Discharges

Hydrograph simulations were performed using HEC-HMS [4] for 1-, 2-, 5-, 10-, 25- and 100-year return periods with durations of 1, and 24 hours. For this extended study special emphasis will be given to storm durations of 6 hours or less. Table 2 summarizes the simulation results for all return periods and corresponding durations.

Table 2
Peak Discharges for the Toro Negro Recreational Pool Area

Duration (hours)	Return Period (years)	Sub basin 1 (cms)	Sub basins 2 & 3 (cms)	Total (cms)
1	1	1.6	0.9	2.4
	10	3.6	2.0	5.4
	100	6.6	3.9	9.9
	1	3.2	1.7	4.8
24	10	7.4	4.2	11.1
	100	12.0	6.8	18.0

To validate these results the rational method [3] was used to compute the 10yr-24hr and 100yr-24hr events which are summarized in Table 3. Discharge values are slightly higher but well below the acceptable range of error and so previous values are representative of the current condition.

Table 3
Rational Equation (10yr-24hr & 100yr-24hr Events)

Basin ID	Area (ac)	C	i(10)	i(100)	Q_{10} (cms)	Q_{100} (cms)
1	66	1	4.75	6.70	8.89	12.54
2	20	1	4.75	6.70	2.65	3.74
3	12	1	4.75	6.70	1.62	2.29
Total	98	1	4.75	6.70	13.16	18.57

HYDRAULIC STUDY

The previous study [1] showed how peak discharges in the order of 4.8 cubic meters per second (169.5 cfs), 6.8 cubic meters per second (240.1 cfs) and 18 cubic meters per second (636 cfs) affected the water surface elevation profile, flow regime and velocities along the pool area and boundaries for the respective 1yr-24hr, 2yr-24hr and 100yr-24hr rainfall events.

For this extended study events with durations of less than 6 hours will be modeled and evaluated with additional changes to the ones previously proposed. This additional changes will tackle sudden flash flood discharges from endangering future users of this facility. The model simulates the routing of surface and system waters considering hydraulic parameters such as surveyed cross sectional data, inverts, diameters and roughness coefficients.

In addition the proposed changes from this extended study must be proven to not adversely affect the water surface elevation profile of the previously studied event of 100yr-24hr. This final analysis will be performed in HEC-RAS [9] in order to compare to previous proposed changes.

The site survey provided by the Municipality of Orocovis was complemented with a site visit to validate previous observed conditions. *The survey was not geo-referenced and was performed with an arbitrary coordinate system. Being so, any elevations mentioned or illustrated in the following sections do not represent mean sea level values.*

Reach & Hydraulic Structure Characteristics

The site consists of two streams high up on the top mountains of the Toro Negro Forest Reserve which define and reach a confluence just upstream of the pool area. This pool area is fed by a 4-inch pipe intake on a headwall with a water table 3-inches above the top of orifice. An existing bypass channel is fed by a #-inch wide weir with a depth of water of 1-inch and an invert 6-inches above the existing water intake's invert. The pool area has a downstream dam with 2-feet wide by 2-feet high sluice gate in the bottom of the pool, two 6.56-feet wide by 2.33-feet high lateral weirs and two 2.75-feet wide by 2.33-feet high center weirs all of whom have inverts 12-feet above the sluice gate's invert. From there on downstream the stream resumes its natural path in what appears as a supercritical flow regime.

Modeling Methodology

The computer model Autodesk Storm and Sanitary Analysis [10] from now on known as SSA was used to perform a hydrodynamic routing analysis of the proposed conditions. Stage-volume relation were calculated and input in the model nodes representing the distribution box, swimming pool and storage dam areas. All other components including existing bypass channels, overflow weirs and orifices were input into the model accordingly. Corresponding modifications including proposed glory hole, open channel and weir modifications were added to the proposed alternative.

Modeling Methodology Data

Hydrodynamic Routing [11] is the most sophisticated routing method in SSA solving the complete one-dimensional Saint Venant flow Equations to produce the most theoretically accurate result. Consisting of continuity and momentum equations for conduits and volume continuity equations at nodes, which makes it capable of being applied in virtually any general network layout, even those containing multiple downstream diversions and loops. This generality comes at a price of having to use time steps in the order of a minute or less to achieve numerical stability. This link routing method can account for channel storage, backwater, entrance losses, exit losses, and pressurized flow. The basic geometric data consist of establishing the connectivity of the drainage system, conduit length, corresponding inverts at both ends along with nodes, inlets and storage curves. Hydraulic structure data (Rim, sag, weirs, orifices, cross sections, etc.) is also needed. In addition, hydraulic properties such as Manning’s roughness coefficient and ineffective flow or interest areas are assigned to each link.

The unsteady flow data and geometrical properties of the network studied are described in detail on the following sections.

Existing Hydraulic Structures

In order to maintain constant normal operation of the swimming pool, base flow conditions must be established. Since the main recreational pool area is fed by the natural stream of water through a 4-inch pipe orifice with a water table of 3-inches above its top crest in both previous site visits and a recent one. The base flow into the swimming pool was estimated at 0.03 cubic meters per second (1.06 cfs) using the Orifice Equation (1). The bypass weir was also estimated at 0.03 cubic meters per second (1.06 cfs) using the Weir Equation (2).

Proposed Geometry

As part of this extended study previously proposed changes will be optimized with newly proposed hydraulic structures in order to maximize the safety of those who use the recreational pool area.

Previous Study

The previous hydraulic modifications in the pool area consisted of raising the intake headwall by 6-inches, replacing the intake pipe with a 6-inch diameter orifice, the construction of a concrete sediment box and the construction of a 56.4-feet pool dividing dam with a 2.5-feet wide 6-inches deep base flow box weir that would separate the swimming pool from an ornamental deeper pool just upstream of the existing dam.

Extended Study

In order to not stray much from what was already proposed, previous proposed geometry is used with slight modifications as an extended proposed alternative.

Glory Hole Discharge Weir at Headwall & Sediment Trap Box

As a solution to the frequent overtopping of the previously proposed intake headwall changes; a box shaped glory hole with a crest elevation 1.8-feet above the bottom elevation of the previously proposed sedimentation box, discharging into a nearly 3-feet drop which feeds into a 3-feet by 5.58-

feet open channel. Figure 1 and Figure 2 illustrate the proposed changes in this area.

Proposed Surcharge Bypass Open Channel

The proposed channel is divided into two prismatic segments with varying slopes. The first and steeper 10.67% slope segment has an approximate length of 19-feet and is situated just downstream of the 3-feet drop from the glory hole. The longer segment with a 1.16% slope has a total length of 98.42-feet and discharges into the previously proposed ornamental pond which from here on out will be referred to as the storage dam.

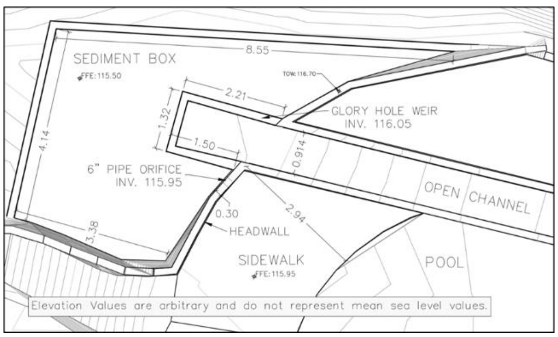

Figure 1
Proposed Glory Hole and Open Channel Plan View

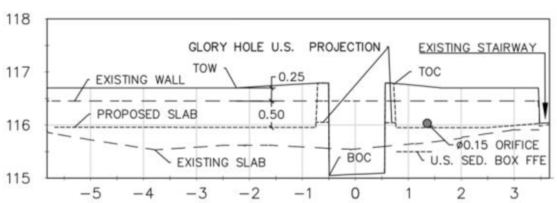

Figure 2
Downstream facing Proposed Headwall, Orifice & Glory Hole Weir Cross Section

It is imperative to mention that for these hydraulic structures to serve their purpose, proper maintenance must be part of a rigorously enforced clean-up routine of the sedimentation box, bypass open channel and storage dam proposed as part of this extended study.

Proposed Storage Dam Retention Pool

The proposed bypass channel discharges into the previously proposed 135,000 gallon pool with 40,225 gallons filled during normal operation at a water depth of 7.55 feet. The resulting 94,775 gallons of volume with a remaining water depth of 6.56 feet are available as storage during extraordinary rainfall events. The stage relation curves for area and volume in the storage dam are shown in Figure 3.

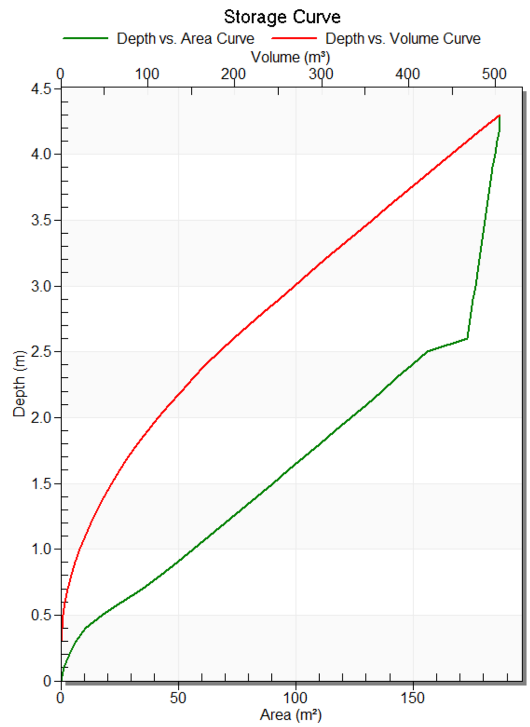

Figure 3
Storage Dam Stage-Area & Stage-Volume Curves

The existing weir structure of the original pool dam remains unchanged but the 2-feet by 2-feet square sluice gate is closed for normal operation while an additional 8-inch orifice is added at an invert depth of 4.59-feet of water. This orifice will account for the base-flow discharge out of the storage dam during normal operations and add discharge capacity when combined with an open sluice gate.

Figure 4 shows a plan view of the area and its components.

Proposed Swimming Pool

The recreational area's 239,000 gallon swimming pool is filled up during normal operation providing no storage and so no stage relation curves were provided for this report.

The pool's intake feeds the base-flow through the 6-inch orifice located in the sedimentation box headwall and its outflow is discharged into the dam storage through the 2.5-feet wide 6-inches deep base flow box weir located in the previously proposed pool dividing dam. Figure 4 illustrates in plan view the components described above.

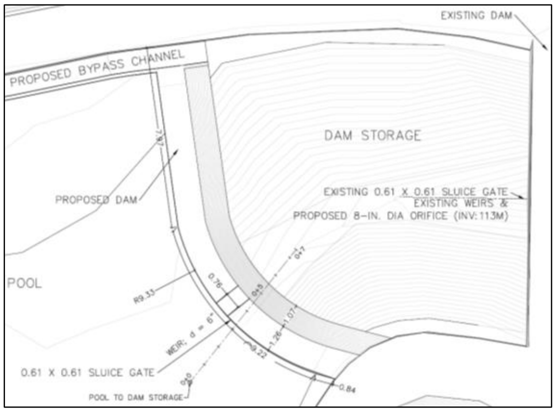

Figure 4
Dam Storage, Proposed Dam, Existing Dam Plan View

Model Schematic

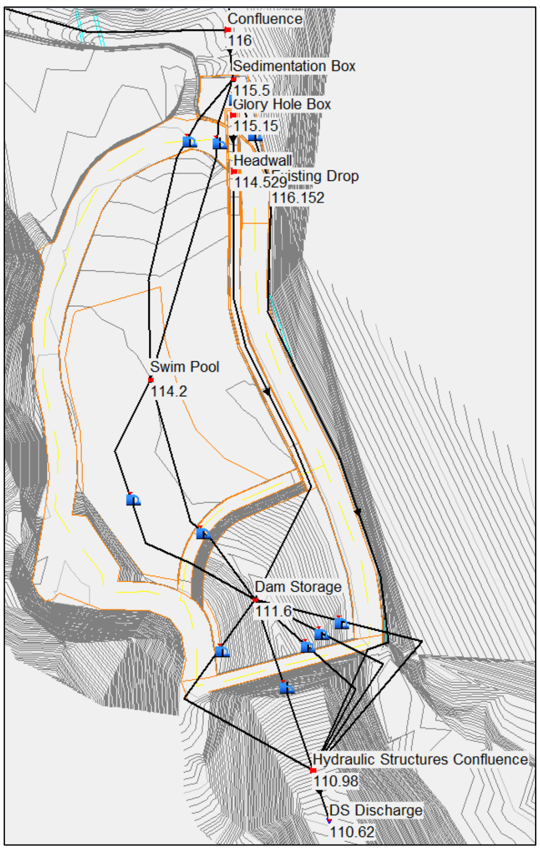

Figure 5
Schematic Model Plan View with Node Inverts

The Toro Negro Recreational Pool Area is composed of a complex system of Links (weirs, orifices, channels) and nodes (pools, drops, conveyance changes). The proposed changes in this extended study further complicate the system and its analysis. A General Area Plan View of the models nodes and links is shown in Figure 5. Further detail of the schematic will be described by node as follows.

Stream Confluence

The most upstream node in the system is a confluence junction between both main streams that feed the pool area. Both channels were input their respective cross sections taken from the survey used for the previous study. A third cross section was input into the model just downstream of the confluence and upstream of the sedimentation box. Figure 5 shows the confluence node just upstream of the sedimentation box along with all three cross section links.

Sedimentation & Water Distribution Box

The next node in the system is the Sedimentation Box which serves as the distribution storage node in the system. In this structure, water surface elevations interact with the different components described in previous. The glory hole, existing weir, overflow weir and proposed base flow orifice along with the existing drop and bypass channel and headwall node which acts as a slope change for the proposed channel, are shown in Figure 6.

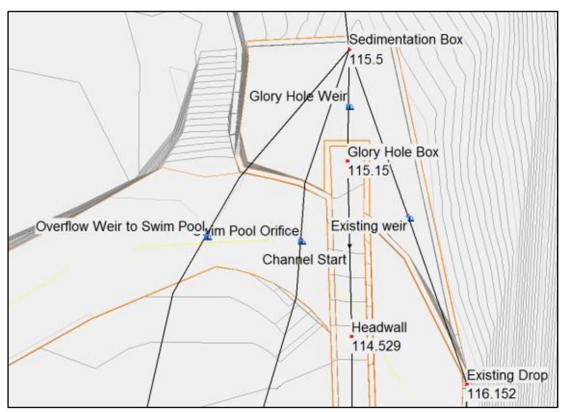

Figure 6
Sedimentation Box Schematic Detail

Swimming Pool & Bypass Open Channel

The main swimming pool area which draws water from either the sedimentation box's orifice or overflow weir is a storage node that will be modeled full to its base flow weir invert. This base flow weir along with an overtop weir are its two means of discharging into the storage dam as shown in Figure 7.

Storage Dam & Existing Dam Structure

The last storage node is the storage dam which receives water from the swimming pool's base flow and overflow weirs in addition to the proposed bypass channel. This storage node discharges through different stages by means of a base flow orifice, three weirs and its overflow weir out of the recreational pool area. The existing bypass channel also discharges at this point where a downstream cross section represents the last link before the systems outfall. The described components are shown in Figure 7.

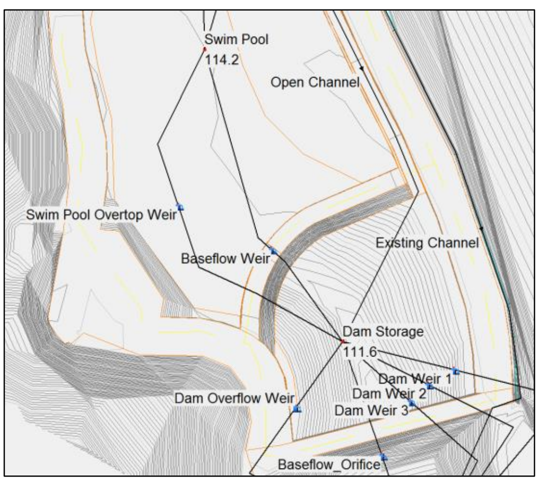

Figure 7
Dam Storage Node & Link Schematic Detail

ANALYSIS AND RESULTS

Flash Flood Analysis (High Intensity Events)

In order to simplify the analysis, emphasis will be given to the 100yr-1hr event since it represents in its range the most improbable of storm events with a short duration of time and exceedingly high discharge intensity even when using a more spread-out distribution method like the frequency method.

The unsteady hydrodynamic simulation for this analysis used the output hydrograph from HEC-HMS [4] for the extraordinary rainfall event of interest. Figure 8 shows the unsteady state rainfall data for the 100yr-1hr rainfall event starting at the 10th hour of the simulation. The simulation is given a 10 hour stabilization buffer in order to account for storage and conveyance equilibrium and better detect the sudden increase in water surface elevation on the proposed storage dam. A routing time step of 5 seconds was used to run the simulation with reporting of time steps on a minute basis.

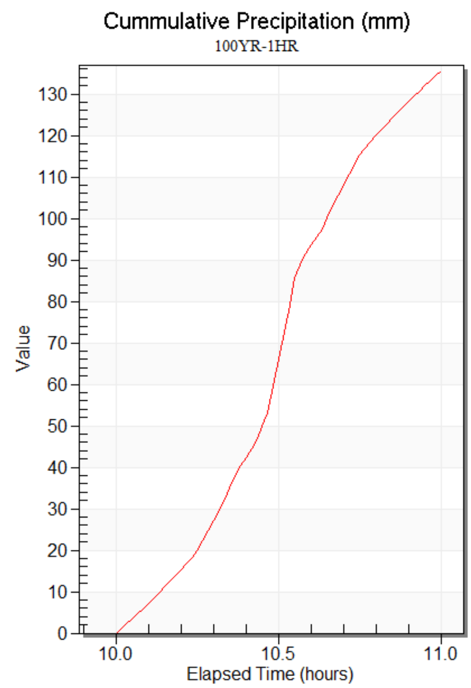

Figure 8
Cumulative Precipitation from HEC-HMS

Two high interest markers are identified in the system; the first is the stage elevation on the storage dam which stabilize around 113.97m before the rainfall event. At 26 minutes past the 10 hour mark, water surface elevations rise to 114.07m for an increase of 4-inches. The other marker of interest is the overflow weir on the intake that discharges into the swimming pool. Once this structure spills water, life and property are at risk inside the recreational pool area. This structure spills water 41 minutes after the 10 hour mark. As a result, a minimum of 15 minutes of reaction time are available between markers. Table 4 shows markers and resulting reaction times for 100yr-1hr event. Figure 9 illustrates the time series and markers for the 100yr-1hr rainfall event.

Table 4
Reaction time for the 100yr-1hr event

Return Period	100 Years
Duration	1 hour
Intake Overtopping	41 min
WSE Alarm Dam Storage	26 min
Reaction Time	15 min

*Events start 10 hours into the simulation.

Regulatory Flood (Long Duration Events)

In addition to the focus of the study in short duration high intensity rainfalls, the proposed changes should not adversely affect the water surface elevations in the regulatory statistical rainfall event of 100yr-24hr. Since the site is located in the middle of a national forest, property limits are far away from the impacted area. Regulation 13 of the Puerto Rico Planning Board [12] does not apply for the reach under study because of these facts. Still because of the nature and use of the pool area, water surface levels were evaluated.

Events with durations exceeding 12 hours flood the lower levels of the recreational pool area overtopping the intakes weir conveying the creeks discharge like a river rather than a complicated set of hydraulic structures. HEC-RAS [9] was used to evaluate the changes and see how they compare to the previously proposed changes for these type of storm events.

Out of the two contributing creeks, the one that runs from sub-basin 1 to the east has a small bridge connecting to a set of bathrooms making it a point of interest in addition to the pool area. Figure 10 shows the schematic for the reach under study.

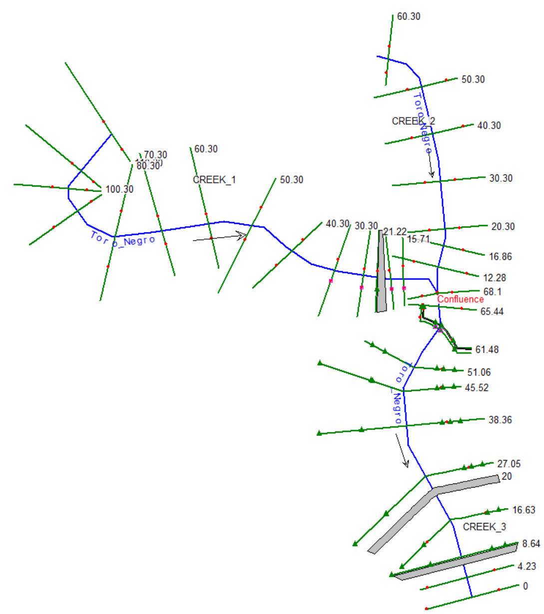

Figure 10
HEC-RAS Reach Schematic

The changes proposed were input into the model for the intake culvert structure, the proposed dam

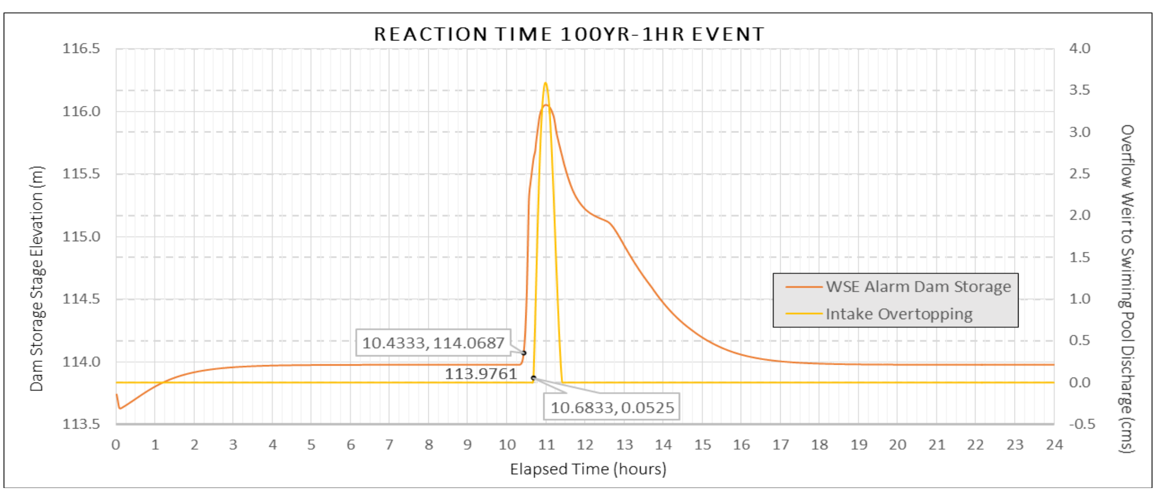

Figure 9
Dam Storage Stage Elevation & Intake Overflow Time Series Results for the 100yr-1hr Event

dividing inline structure and the existing dam's base-flow orifice. All cross-sections in the pool area were updated to reflect the reduction in pool depth provoked by the proposed bypass channel. Upstream cross-sections for both culvert and inline structures are shown in Figure 11 and Figure 12.

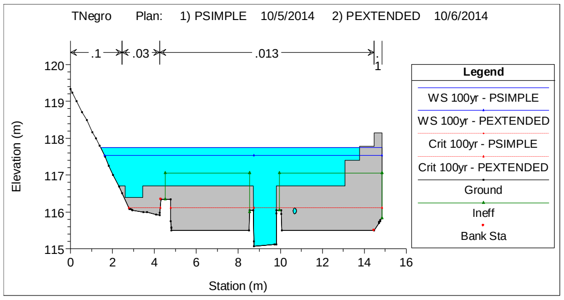

Figure 11
Intake Culvert Structure with 100yr-24hr Water Surface Elevations for Previous & Extended Alternatives

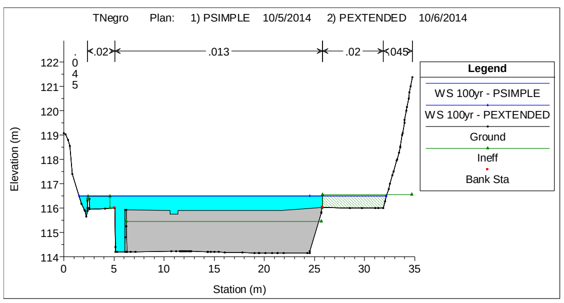

Figure 12
Dam Dividing Inline Structure with 100yr-24hr Water Surface Elevations for Previous & Extended Alternatives

Just upstream of the bridge before the pool area a reduction in water surface elevation of 0.21m just was observed in the simulation. Another significant drop in WSE was observed in the intake wall area with a 0.29m drop in the confluence between creeks and a 0.21m drop in just upstream of the intake headwall. Results for both previous and extended proposals are shown in Table 5. Figure 13 illustrates the 100yr-24hr water surface profiles for both previous and extended proposals.

In addition, Table 5 shows the comparison between water surface elevations in both alternatives as it's worth noting a 0.35m rise just downstream of the intake wall. This rise is representative of the discharge inside the bypass weir and should not be interpreted as happening downstream of its overflow weir as both finish floor elevations after the wall are the same at 115.95m.

Table 5
Water Surface Elevation Comparison

Reach	River Sta	Profile	WSE Previous (m)	WSE Extended (m)	ΔWSE (m)
CREEK_1	110.3	100yr	121.09	121.09	0
CREEK_1	100.3	100yr	120.38	120.38	0
CREEK_1	95.71	100yr	120.04	120.04	0
CREEK_1	90.3	100yr	119.76	119.76	0
CREEK_1	80.3	100yr	119.25	119.25	0
CREEK_1	70.3	100yr	119.22	119.22	0
CREEK_1	60.3	100yr	118.91	118.91	0
CREEK_1	50.3	100yr	117.81	117.81	0
CREEK_1	40.3	100yr	118.51	118.29	-0.22
CREEK_1	30.3	100yr	118.5	118.28	-0.22
CREEK_1	24.14	100yr	118.48	118.24	-0.24
CREEK_1	21.22	100yr	118.46	118.25	-0.21
CREEK_1	20	100yr	Bridge		
CREEK_1	18.43	100yr	118.29	118.16	-0.13
CREEK_1	15.71	100yr	118.28	118.16	-0.12
CREEK_2	60.3	100yr	121.02	121.02	0
CREEK_2	50.3	100yr	119.34	119.34	0
CREEK_2	40.3	100yr	118.79	118.79	0
CREEK_2	30.3	100yr	118.16	118.16	0
CREEK_2	20.3	100yr	118.25	118.13	-0.12
CREEK_2	16.86	100yr	118.26	118.14	-0.12
CREEK_2	12.28	100yr	118.28	118.16	-0.12
CREEK_3	68.1	100yr	117.65	117.36	-0.29
CREEK_3	65.44	100yr	117.71	117.49	-0.22
CREEK_3	61.48	100yr	117.75	117.54	-0.21
CREEK_3	60.7	100yr	Culvert		
CREEK_3	60.62	100yr	116.65	117	0.35
CREEK_3	51.06	100yr	116.49	116.48	-0.01
CREEK_3	45.52	100yr	116.49	116.49	0
CREEK_3	38.36	100yr	116.5	116.49	-0.01
CREEK_3	27.05	100yr	116.5	116.49	-0.01
CREEK_3	20	100yr	Inl Struct		
CREEK_3	16.63	100yr	116.38	116.37	-0.01
CREEK_3	8.64	100yr	116.38	116.37	-0.01
CREEK_3	6.5	100yr	Culvert		
CREEK_3	4.23	100yr	112.79	112.79	0
CREEK_3	0	100yr	111.21	111.18	-0.03

CONCLUSION & RECOMMENDATIONS

After revising the literature and performing the hydraulic calculations and simulations for the extended proposed condition, the following is concluded:

- Hydrological conditions for the examined reach have not significantly changes since the previous study.
- Using previously proposed changes with some enhancements and operational rules, the capacity and safety of the facilities can be greatly increased.
- The construction of the proposed bypass weir/channel and a stage-alarm system in the

previously proposed storage dam can give a 15-minute evacuation window for rainfall events with return periods of 100-years and 1-hour durations.

- With the extended proposed conditions, no changes in water levels outside the property for the 100-year event were observed. The project complies with Regulation #13 of the Puerto Rico Planning Board [12] by not experimenting a rise in water levels by more than 0.15 meters for colliding properties.
- Water surface levels in the 100yr-24hr event slightly decreased in the recreational pool area, while significantly decreasing by 0.21m just upstream of the bridge before the pool area, 0.29m in the intake wall area and 0.21m in the confluence between creeks just upstream of the intake headwall.

All previous recommendations and the following are necessary to maintain the system working properly:

- The previously proposed sediment trap area and proposed bypass channel require a rigorous cleaning & maintenance program.
- The structural design shall consider forces provoked by the water velocity and floating debris that flow through the proposed bypass channel. The designer should also consider the fact that the previously proposed pool dividing dam is no longer fixed at both ends to the pools walls and should provide with the additional reinforcements in order to prevent dam breaking.
- The bypass weir for the proposed channel should be protected by a 4 sided cage from large debris.
- The alarm system should be periodically tested to ensure proper operation.

$$Q = C_d * A * \sqrt{2gh} \qquad (1)$$

Where:

$h =$ Head acting on the centerline (m)

C_d= discharge coefficient,

$A =$ Area (m^2),

$g = 9.81\, {}^{m}/{}_{s^2}$,

$$Q = C * L * H^n \qquad (2)$$

Where:

$C =$ Constant for structure

$L =$ Width of the crest (m)

$H =$ Head of water of crest (m)

$n =$ 1.5 for rectangular weir structure

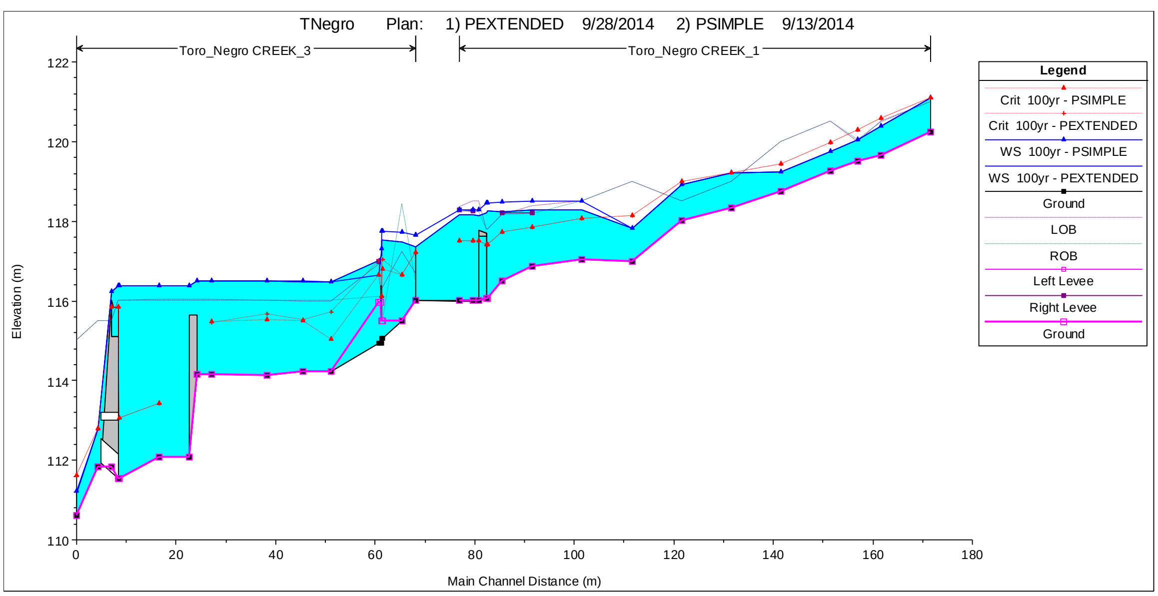

Figure 13

Previous and Extended Water Surface Profiles for the 100yr-24hr Rainfall Event

REFERENCES

[1] López, F. M., "Toro Negro Recreational Area Hydrologic and Hydraulic Study", SALO Engineering PSC, San Juan, PR, September 2013.

[2] Chow, V. T., et al., "Chapter 14: Design Storms", *Applied Hydrology*, McGraw-Hill, 1988, pp. 444-492.

[3] Chow, V. T., et al., "Chapter 15: Design Flows", *Applied Hydrology*, McGraw-Hill, 1988, pp. 496-502.

[4] Hydrologic Engineering Center (HEC) Institute for Water Resources, US Army Corps of Engineers (USACE) , "HEC-HMS 4.0", Retrieved from: http://hec.usace.army.mil/software/hec-hms/, on December 31, 2013.

[5] National Weather Service (HEC) Hydro-meteorological Design Center Studies, National Oceanic and Atmospheric Administration (NOAA), "Precipitation Frequency Data Server (PFDS)", Retrieved from: http://hdsc.nws.noaa.gov/, on August 25, 2014.

[6] Soil Conservation Service (SCS), United States Department of Agriculture (USDA), "Chapter 2: Estimating Runoff", *Technical Release 55 (TR-55)*, 2nd Edition, June 1986.

[7] Soil Conservation Service (SCS), United States Department of Agriculture (USDA), "Chapter 3: Time of Concentration & Travel Time", Technical Release 55 (TR-55), 2nd Edition, June 1986.

[8] Woodward, D. E., *et al.*, "Chapter 1502: Methods of Estimating Time of Concentration – Watershed Lag Method", *Part 630 Hydrology National Engineering Handbook*, May 2010.

[9] Hydrologic Engineering Center (HEC) Institute for Water Resources, US Army Corps of Engineers (USACE) , "HEC-RAS 4.1", Retrieved from: http://hec.usace.army.mil/software/hec-ras/, on January 2010.

[10] Autodesk, "Storm & Sanitary Analysis 2012 Technical Capabilities and Functionalities", 2011. Retrieved from: http://download.autodesk.com/us/systemdocs/pdf/AutoCAD_Civil3D_2012_Storm_Sanitary_Analysis_TechnicalCapabilities.pdf.

[11] Autodesk, "Storm & Sanitary Analysis 2015 User Guide", 2013. Retrieved from: http://images.autodesk.com/adsk/files/Autodesk _Storm_and_Sanitary_Analysis_2014.pdf.

[12] Morales Vargas, H., et al. "Reglamento de Planificación Número 13", *Reglamento sobre Áreas Especiales de Riesgo a Inundación*, Séptima Revisión, September 30, 2009.

[13] Junta de Planificación de Puerto Rico (JP), "Portal de datos Geográficos Gubernamentales - Descarga de Geodatos - Ambientales", 2010. Retrieved from: http://gis.pr.gov.