Computer Design of Lateral Weirs System for Irrigation on Vegetative Strips

Walter F. Silva-Araya[1] and Victor Vargas[2]
[1] Dept. of Civil and Surveying Engineering, University of Puerto Rico at Mayaguez
Mayaguez, Puerto Rico, 00681
Walter.silva2@upr.edu
[2] Graduate Student, Dept. of Civil and Surveying Engineering, University of Puerto Rico at Mayaguez, Mayaguez, Puerto Rico

ABSTRACT

Jobos Bay National Estuarine Research Reserve (JBNERR) is located at the south eastern coast of Puerto Rico. The reserve includes subtropical forests, mangrove forests, coral reefs, seagrass beds, salt and mud flats, lagoons and freshwater wetlands. Mar Negro wetland is the biggest mangrove forest in the area, located at the south boundary of JBNERR. The north side of the Reserve has been used for agricultural activities since the Spanish Colonial times. Since 1993 the mangrove population started to diminish. Its mortality was promoted by antropogenic activities such as deforestation, domestic sewage discharging into the mangrove lagoon, change of hydrologic patterns caused by urban developments and agricultural practices. During the decade of 1990 intensive agricultural activities north of the reserve caused a negative impact on mangroves. Pesticides, fertilizers and chicken manure applied in agricultural fields were being discharged directly into de mangrove forest, mainly through two drainage earth channels. The objectives of this project were: 1) To model the hydrologic conditions existing north of JBNERR, 2) Hydraulic design of side-weirs along channels to promote uniform distribution of irrigation water from agricultural activities and runoff. Water discharges into a vegetative strip before discharging into Mar Negro. The hydraulic analysis was performed using ArcGIS, in-house computer models and, EPA's Storm Water Management Model (SWMM). The hydraulic design resulted in an innovative system of lateral weirs which distribute water uniformly along the vegetative strip. This water distribution system controls surface runoff from irrigation and rainfall, reduces surface erosion and improves the quality of overland flows discharging into Jobos Bay.

Keywords: Hydraulic design, side-weirs, vegetative strips, mangrove protection, Jobos Bay Estuarine Research Reserve, irrigation, computer simulation, Puerto Rico.

1. INTRODUCTION

The Jobos Bay National Estuarine Research Reserve (JBNERR) is one of 26 estuarine areas under the National Estuarine Research System designated by the National Oceanographic and Atmospheric Administration (NOAA). JBNERR located between Salinas and Guayama at the south of Puerto Rico. Jobos Bay covers an area of 2,833 acres of mangrove forest and diverse habitats from the landward transition zone of coastal fan-delta and alluvial deposits to offshore cays in the Caribbean Sea (Kuniasky et al, 2010).

[1] Professor, Dept. of Civil and Surveying Engineering, University of Puerto Rico at Mayaguez
[2] Graduate Student, Dept. of Civil and Surveying Engineering, University of Puerto Rico at Mayaguez

Since Spanish colonial time until 1970's, the principal land use in the Jobos Bay Reserve watershed was agriculture. From coconuts plantation bordering the shoreline to sugarcane on the coastal plain, agriculture was predominant. In the 1960's, the industrialization started in Puerto Rico and the sugarcane cultivation went down until the Central Aguirre closed on 1990. After the sugar cane era, these lands continued with the agricultural activity but changed to the production of vegetables and fruits (Whitall et al., 2011; Kuniasky et al., 2010). Changes in water management in the Jobos area evolved parallel to agricultural activities. The drainage hydraulics and hydrology of the JBNERR area has been frequently modified by the construction of canals and ditches to capture water from the streams for irrigation purposes. From 1910 to 1935, sugarcane industry increased irrigation and a reservoir network was constructed to supply water to those uncultivated areas (Kuniasky et al., 2010). The sugar cane industry builds a series of canals to drain the water pumped from the wells. Excessive pumping started to lower the water table and the demand of water supply for agriculture increased.

In 1993, the Puerto Rico Land Authority (PRLA) selected Hacienda Aguirre (see Figure 1) to install a demonstration project on corn planting using an irrigation pivot system. The site was plowed and the top soil was placed near the northern boundary of Mar Negro, creating a dike which was used as roadway. This dike altered of the flow pattern in the zone. Due to this action, six of seven abandoned ditches were cleaned and excavated. These ditches (two of them are in the area of interest of this project) drains from north to south, directly into the mangrove forest (Gregory Morris and Associates, 2000).

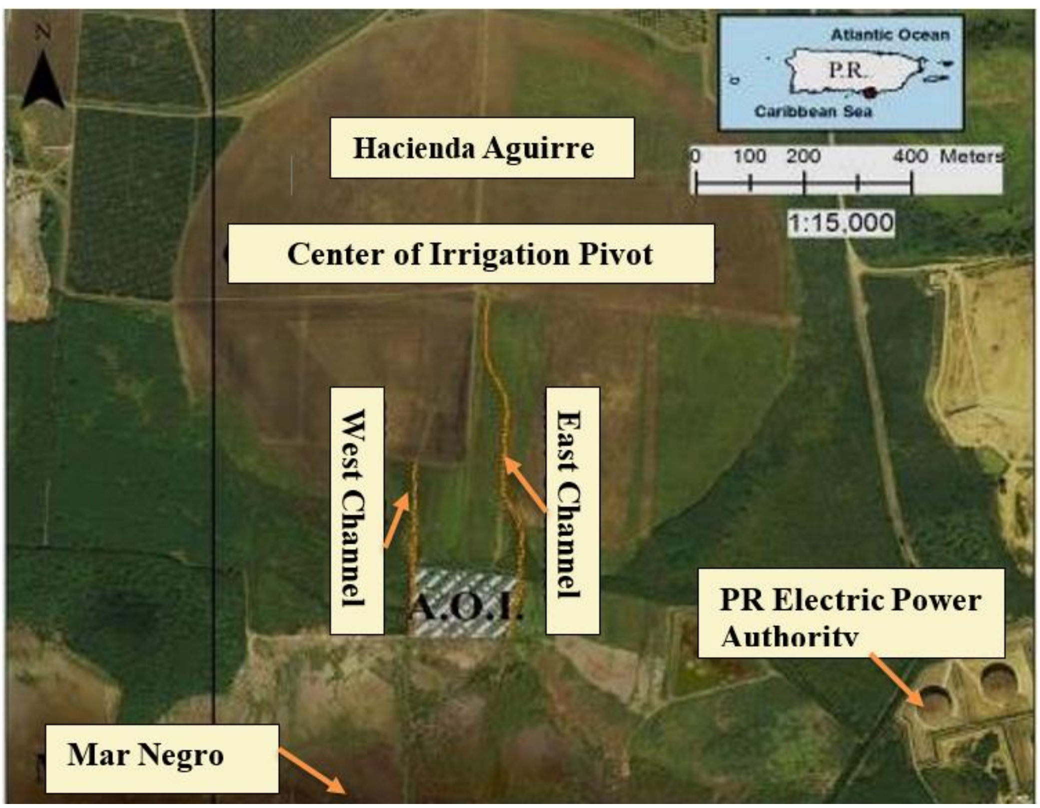

Figure 1. Location of the drainage channels. (Aereal image by 3001, Inc. 2007)

1.1 **Problem Statement**
The northern area of the JBRR has been used for agricultural activities since the Spanish colonial times. Figure 2 shows historic pictures of the study area since 1987 (Figure 2.A). At this time agriculture was present under the sugar mill Central Aguirre until 1990 when this industry closed operations. In 1993, the center pivot irrigation system was installed close to the mangrove forest. Figure2.B, from 1997, shows that the mangrove cover was residing. The pivot irrigation method was operational until late 2009 when the PRLA decided to stop operations to upgrade the system. However, at current time, it remains non-operational. In 1998, the Hurricane Georges affected the area causing great damage to the mangrove forest. The overall impact of hurricane Georges on Jobos Bay mangroves is not known with precision; however, hurricanes generally set back succession and reduce mangrove areas (Demopoulos, 2004). Figure 2.C shows the situation of the mangroves forest in 2007 which is similar to current conditions.

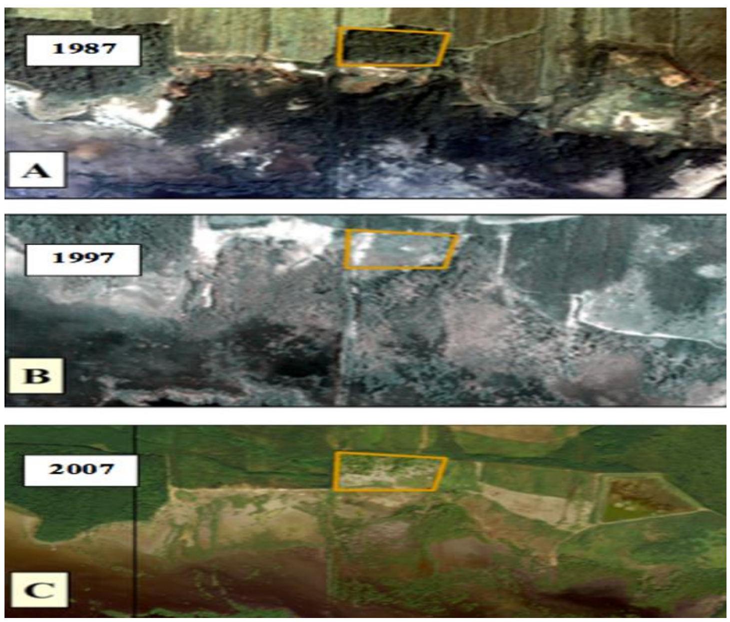

Figure 2. Photos showing the evolution of the project area during the last decade. Project area shown in the orange box. Aereal Images by 3001 Inc., 2007.

1.2 Scope of this Study
The purpose of this project was the design of an innovative overland flow water distribution system using lateral weirs discharging into a vegetative strip to improve the quality of waters discharging into the Jobos Bay mangrove area, control surface runoff, and reduce surface erosion. This paper focuses on hydrologic and hydraulic modeling and simulations used for design of the lateral-weir system.

2. MODEL DESCRIPTION

This project could be divided in two differents modeling parts, the hydrology, which determines the surfaces runoff for the given subwatershed and the design of open channels with different features and lateral weirs. EPA SWMM model was used in both parts. (James et al., 2011).

2.1 Hydrologic Modeling

The surface runoff in SWMM is computed using the water balance equation which takes in consideration the precipitation, storage, evaporation, infiltration, and runoff. The modeling software treats the subcatchments as a nonlinear reservoir and the water input comes from precipitation or any upstream subcatchments. The storage capacity in the subcatchments will depends on the ponding, surface wetting and interception information provided by the user. The surface runoff depends on the storage information and the Manning`s equation. The outflows are integrated in the system as evaporation, infiltration, and runoff. The water depth, which is continually changing depending of time and amount of inflows, will be computed with the runoff per unit area over the subcatchments (Rossman, 2010). SWMM provides three alternatives of infiltration models; however, the Curve Number method developed by Natural Resources Conservation Service (NRCS) was followed in this project (James and James, 2000).

2.2 Hydraulic Modeling

The hydraulics computations will depend on the flows generated by the hydrologic model. In general, SWMM represents the hydraulic system by different elements such as conduits, nodes (junction, outfall, and flow divider nodes), storage units, pumps, flow regulators and weirs. Some of these elements are used to model the hydraulic system proposed in this project. A conduit in the current project represents the open channel that moves the water from SWMM node to another. Nodes are elements which links conduits. These elements are able to represent confluence of natural surface channels and they can receive external inflows into the channel system.

The conduits, in SWMM, can be represented in all geometric shapes. The user has the option to draw transects of the channel or cross section. For this project, the trapezoidal geometry was used for open channel simulation. Side or lateral weirs are considered in SWMM are considered "flow regulators". These are structures used to control or divert flows within a conveyance system. Side weirs are represented in SWMM as a link connecting two nodes, where the weir is placed upstream of the node. Dynamic wave routing was selected as the mathematical model for hydraulic design. The algorithm solves the complete one-dimensional Saint Venant flow equations consisting of the unsteady momentum and continuity equations for each channel reach. Dynamic wave method computes backwater, entrance/exits losses, and flow reversal. The requirement of this method is to use smaller time step, usually minutes or less. The equation used for side weirs in SWMM are represented by the general formula (Chaudhry, 2008).

$$Q_w = C_w L h^{3/2}$$

where C_w = weir discharge coefficient, L = length of weir crest, H= head above weir creast and, Q_w = discharge across the weir. A total of eight equations for side-weirs design were tested using in-house computations. Table 1 was adapted from Emiroglu et al., 2011 and summarizes the equations used in this research. See references for details. The variables in Table 1 are: C_u =

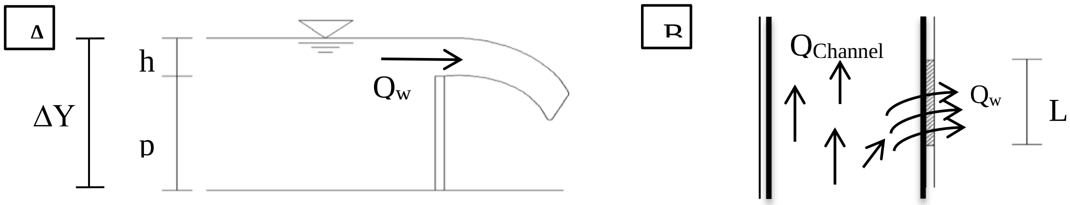

Figure 4. Weir schematics: A: Lateral view of the weir; B: Top view of the weir.

discharge coefficient, N = 1 for sharp weir crest, g = acceleration of gravity, L*w* = effective length of creste, P = crest level from channel bottom, *Hl* = height of water surface above weir crest in parent channel at downstresm end (y-P), J = coefficient as a function of the ration h_0/L and h_0/P (see reference), K = coefficient function of the ratio h_0/P_0 (see reference), B = channel width, b = top of water, F_a = Froude number upstream of weir (see reference).

3. HYDROLOGIC MODELING RESULTS

3.1 Watershed Characteristics

The hydrological analysis began with the delineation of basins and sub-basins in ArcGIS, using a USGS topographic map, aerial photography (USACE) and field work. The roughness coefficients for overland flow were obtained from the NRCW website by entering the watershed as the area of interest. Other variables determined for overland flow computation were the precipitation for design and the abstractions. The abstraction method selected was the Curve Number method (CN). The CN values were obtained using high resolution aerial photos, field work and a GIS shapefile of Puerto Rico CN prepared by the NRCS.

Table 1. Lateral-weir equations (Refer to Emiroglu et. al, 2011 for details)

Author	Discharge coefficient (C_d) equation*
Nadesamoorthy, T. and Thomson, A.	$C_d = 0.432\left(\frac{2-F_A^2}{1+2F_A^2}\right)^{0.5}$
Subramayan, K. and Awasthy, S. C.	$C_d = 0.864\left(\frac{1-F_A^2}{2+F_A^2}\right)^{0.5}$
Hager, W.H.	$C_d = 0.485\left(\frac{2-F_A^2}{2+3F_A^2}\right)^{0.5}$
Sing R, Manivanna, T and Satynarayana, T.	$C_d = 0.33 - 0.18F_A + 0.49\left(\frac{P}{h_1}\right)$
Jalili, M.R. and Borghei, M.R.	$C_d = 0.71 - 0.41F_A + 0.22\left(\frac{P}{h_1}\right)$
Borghei M, Jalili M.R. and Ghodsian M.	$C_d = 0.7 - 0.48F_A + 0.3\left(\frac{P}{h_1}\right) + 0.06\left(\frac{L_w}{B}\right)$
Ranga Raju et al.	$C_d = 0.54 - 0.40F_A$
May. R.W.P., Bromwich, B.C., Gasowski, Y. and Rickard, C.E.	$C_e = n\sqrt{g}\left(J - K\left(\frac{L_w}{b}\right)F_A\right)$

$C_e = \frac{2}{3}C_d\sqrt{2g},$ ** (May et al., 2003)

3.3 Design Event

The irrigation circle located north of the study site is the main area of irrigation, able to produce runoff that reach the AOI. This circle has an area of 101.2 hectares (1.012 km^2) which was divided in 4 subareas of 25.3 hectares each. According to the NRCS (NRCS, 2009) the farmer only uses half of circle area for cropping and leaves the other half at rest, ready for the next sow season. During the operation of the center pivot unit, the farmers only irrigate one quarter of the circle at a time. This method of cropping allows the farmers to choose any two quarters of the circle in which they desire to sow. The operation of the irrigation system depends on the crop type. Irrigation time varied between 13 and 22 hours. Also the irrigation time was chosen depending on the size of the sowing in the quarter of the circle. Corn crops required 22 hours of irrigation twice a week (NRCS, 2009).

During the years of 2008 and 2009 crops of corn, cowpea and sorghum were sowed and irrigated using the pivot unit system (Williams et al., 2012). The sorghum crops required an irrigation of 9.14 millimeters per day, while the corn crops needed 9.89 millimeters per day (NRCS, 2009). The irrigation data correspond to the peak season during the months of July and August. The annual irrigation amount for 2008 and 2009 were 553 mm and 1270 mm respectively (William C. O. et al., 2012). Also the amounts of rain for the same years were 1059 mm and 670 mm, respectively.

Irrigation discharges are very small compared to rainfall generated runoff. The design criteria for the lateral weirs was the maximum accumulation of rainfall during a period of 24 hours. The operation was verified by using irrigation flow after the design was completed. Historical rainfall data was obtained from NOAA National Estuarine Research Reserve website for the meteorological Station Jobos Bay weather (SJB) located in latitude 17°57'23.25"N and longitude 66°13'22.69"W (National Estuarine Research Reserve System, 2012). The SJB is situated approximately 2.5 kilometers to the east of the study area. This station provides information from 2002 to present in periods of 15 minutes. Only data from 2002 to 2011 was analyzed and processed to obtain the maximum accumulation of rain in a period of 24 hours every year. The rainfall chosen for hydraulic design was the 2011 maximum 24-hr duration storm event with a cumulative rainfall of 48.7 mm. This rainfall event produced a peak runoff of 0.36 m^3/s for the West Channel and 1.09 m^3/s for the East Channel. This event produced the minimum flow among the 10 rainfall events analyzed in SWMM. Larger events flooded the area completely making unnecessary a design for peak flows exceeding one-year frequency. This project does not represent any risk for human beings and the design is for agricultural irrigations runoff management, which are lower than this event. Irrigation discharge data available is scarce.

4. HYDRAULIC MODELING RESULTS

4.1 Steady-state computations

Step by step procedure was followed during the computations using in-house MS Excel-based computations. The new design involves two separate channels with 90 degrees bend toward the center of the AOI. These channels will have a bottom width of 3 meters as the current channels and they decrease until 0.5 meters at the end of the channel downstream. Each channel distributes the runoff evenly through three (3) side weirs. The channel depth is 0.40 m for West Channel and 0.75 m for the East Channel. The side slope is 1.5 and longitudinal slope is 0.001 for both channels. The channel lining analysis compared different types of lining: earth, concrete, riprap, and grouted riprap. The selected lining was grouted riprap. The reinforce concrete was rejected due the high cost. The riprap was grouted to eliminate the possibility that an increase of flow can move the layer of rocks and decrease the operational efficiency of the channel. Also, riprap without grout allows vegetation grow between rocks decreasing the flow velocities and increasing the maintenance cost. The lining design was prepared for a riprap size of 90 mm. This size of riprap can produce a roughness of 0.032 but, the grout cement will increase approximately 13% the roughness of the

design liner. The critical shear stress, allowable for the channel bed and walls, are 56.55 MPa and 31.36 MPa, respectively. The maximum shear stress of the West Channel bed and wall applied by the water flow are 3.38 MPa and 2.70 MPa, respectively. The water flow in the East Channel bottom bed and side wall produced a 5.63 MPa and 4.50 MPa, respectively. The channel liner will support approximately 6% of critical shear stress in the bottom bed and 18% in the side wall.
The channels bottom width begins three (3) meters upstream and reduces to 0.5 meters downstream. These reductions occur by three transitions. The transition was straight line as recommended by FHWA (2012). All transitions have a flare angle less than 12.5 degrees to keep subcritical flow along the transition. Each transition is located before the weirs. The inlet transitions reduce the width of the channel downstream, increasing the flow velocity and decreasing the water depth. The selection of the weir equation for design purposes required the discharge coefficient for a side weir condition. The most important variables for the discharge coefficient computation are the crest height and the water head. All discharge coefficient equations include these two variables accompanied with other constants which depend on the experimental conditions from which the equation was derived. Side weir provides the most reliable operation for Froude numbers less than one (May, et al., 2003). The discharge coefficient proposed by Subramayan, K. and Awasthy, S. C. (Emiroglu, et. al, 2011) was selected based on Chow, (1959), and French, (1987) recommendations. Also this equation produces the higher discharge coefficient and similar values for the three side weirs. The side weir equation requires similar discharge coefficient to adjust the condition of length and crest height, in order of obtain similar spill flow in each one. The water profile of the side weirs was computed using the Euler improved method. The transition of water along the side weir in subcritical flow produces an M1 profile characterized by an increase of water depth downstream of the side weir and a reduction of average velocity.

4.2. SWMM computations

After modeling the side weirs using SWMM, the maximum weir discharge obtained fluctuates between 0.100 m^3/s to 0.128 m^3/s in the West Channel and between 0.32 m^3/s to 0.395 m^3/s in the East Channel (see Figure 5). As a pattern, the first two side weirs (from upstream to downstream) have the same amount of flow but the last one has a slighly higher spill flow value. The SWMM model shows a spill flow 15% to 16% lower than the target spill flow for design and aproximately 6% to 10% above the design value in the third weir. Percentage error was computed using the steady state design value versus the values obtained in SWMM. These differences in spill flow could be due to the difference in computational methods.

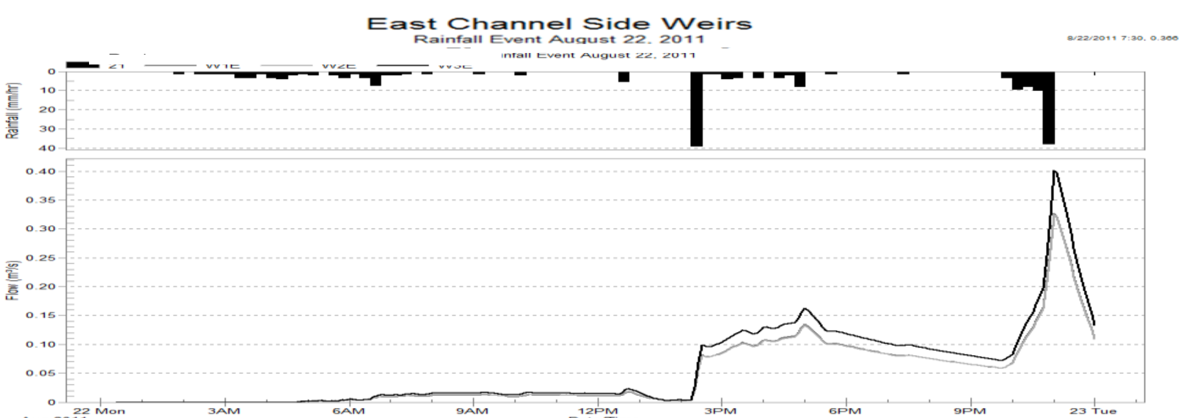

Figure 5. East channel flow magnitudes over lateral weirs obtained with SWMM

5. REFERENCES

1. Chow, V. T. (1959). *Open-Channel Hydraulics.* Caldwell, NJ: McGraw Hill Book Company, Inc.
2. Demopoulos, A. W. (2004). Black Mangrove Benthic Community Structure, Seedling Growth and Survival, and Sediment Characteristics in Anthropogenically Disturbed and Pristine Habitats. Jobos Bay National Estuarine Research Reserve. Jobos Bay National Estuarine Research Reserve.
3. Emiroglu, M., Agaccioglu, H., and Kayaa, N. (2011). *Discharging capacity of rectangular side weirs in straight open channels,* Elsevier Publishers.
4. Federal Highway Administration. (2012). *Hydraulics Engineering*. (V. Ghelardi, Editor, U.S. Department of Transportation) Retrieved 2 12, 2013, from Introduction to Highway Hydraulics - Chapter 4 - Open-Channel Flow.http://www.fhwa.dot.gov/engineering/hydraulic s/pubs/08090/04.cfm
5. French, R. H. (1987). *Open-Channel Hydraulics.* (J. Z. Margolies, Ed.) New York: MacGraw-Hill Book Co.
6. Gregory L. Morris and Associates. (2000). Hydrologic-Hydraulic and Biological Analysis of Jobos Estuary Mangrove Mortality Jobos, Puerto Rico. Santurce, P.R.: Puerto Rico Land Authority.
7. Hager, W. H., & ASCE, M. (1986). Lateral Outflow Over Side Weirs. Journal of Hydraulic Engineering, 113.
8. James, W., Rossman , L. E., & James, W. C. (2011). *Water System Models, User`Guise to SWMM5* (13th Edition ed.), Computational Hydraulics International, Ontario, Canada.
9. James, W. and James, R. C. (2000). Water System Models Hydrology, A guide to the Rain, Temperature and Runoff Models of the USEPA Swmm4. Canada: Computational Hydraulics International.
10. Kuniansky, E. L. and Rodríguez, J. M. (2010). Effects of Changes in Irrigation Practices and Aquifer Development on Groundwater Discharge to the Jobos Bay National Estuarine Research Reserve near Salinas, Puerto Rico,. U.S. Geological Survey.
11. May, R., Bromwich, B., Gasowski, Y. and Rickard, C. (2003). *Hydraulic Design of Side Weirs.* Reston, VA: Thomas Telford Publishing.
12. NRCS. (2009). Salinas Forage Farm Location; Puerto Rico, USA. (p. 41). Natural Resources Conservation Service.
13. Rossman, L. A. (2010). Storm Water Management Model User`s Manual version 5.0. Cincinnati, Ohio, U.S.: Water Supply and Water Resources Division.
14. Whitall, D. R., Costa, B. M., Bauer, L. J., Dieppa, A. and Hile, S. D. (2011). *A Baseline Assessment of the Ecological Resources of Jobos Bay, Puerto Rico.* National Oceanic and Atmospheric Administration, U.S. Department of Commerce. Silver Spring, MD.
15. Williams, C. O., Lowrance, R., Bosch, D. D., Williams, J. R., Benham, E., Dieppa, A., et al. (2012). Hydrology and Water Quality of a Field and Riparian Buffer Adjacent to a Mangrove Wetland in Jobos Bay Watershed, Puerto Rico. *Ecological Engineering* (ECOENG-2313).