HYDROLOGIC-HYDRAULIC AND SEDIMENTATION ANALYSIS FOR YAUCO LANDFILL YAUCO, PUERTO RICO

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I. INTRODUCTION

General

The Yauco landfill is located on the State Road PR-389 Km. 3.8, Barinas ward, within the Municipality of Yauco. Figure 1 shows its location on the USGS topographic maps of Yauco and Punta Verraco, published by the US Geological Survey [1982]. Since the storage capacity of the facility is near the limit it is necessary to be increased. Therefore, additional areas within the property limits will be required for this purpose. An Environmental Impact Statement (EIS) is required to determine the impact on the environment within the project area and vicinity due to the proposed landfill expansion.

For this case, one of the required technical studies which include the EIS is a Hydrologic-Hydraulic and Sedimentation Analysis. The main purpose of this study is the determination of the superficial runoff produced during ordinary and extraordinary rainfall events and the soil erosion potential during those events.

Five retention ponds currently exist within the existing natural drainages and/or creeks within or adjacent to the project property area. The purpose of these ponds is to control the sediments leaving the landfill area.

Flood levels along the creeks or natural drainages will be computed for the 2- and 100-year frequency events since, representative of the ordinary and extraordinary events, respectively. In addition, the sediment yield for the existing and proposed conditions for the 2- and 10-yr events will be determined. Minimum required control structures will be designed for these events. The sediment yield for the 100-year event will be computed for informative purposes only since the resulting minimum required control structures might be too big for their implementation.

The property is not located within flooding zone as indicated in the FEMA's Flood Insurance Rate Maps (FIRM) No. 1615H and 1980H. In spite of that, the project will be treated as a flooding zone and the requirements of the Planning Regulation No. 13 of the PR Planning Board [PRPB, 2005a] will be applied. Figure 2 shows a partial copy of these maps published by the Federal Emergency Management Agency (FEMA).

Authorization

Mr. Ricardo Flores Ortega, P.E., representative of RF Engineering, P.S.C., authorized this study through an agreement with Mr. Miguel Menar Figueroa, P.E., representative of Menar Hydrosystems Engineering, P.S.C.

Scope of Work

The scope of work for the hydrologic-hydraulic study includes the following tasks.

- Site visit to determine the existing hydrologic and hydraulic conditions.
- Hydrologic and hydraulic modeling of the existing and proposed conditions.
- Determination of sediment yields for the existing and proposed conditions for the 2-, 10-, and 100-yr frequency events.
- Determination of the hydraulic profiles for the 2- and 100-year frequency events for the existing and proposed conditions at the existing natural drainages.
- Determine the minimum required change in the existing structures for flow mitigation as required by the Planning Regulation No. 3 of the PRPB. These structures will be designed for the 2- and 100-year peak flow discharges.

This study is divided in three phases: 1) Hydrologic Analysis, 2) Hydraulic Analysis and 2) Sedimentation Analysis. The results will be described below for each phase.

II. HYDROLOGIC AND RETENTION ANALYSES

Methodology

A hydrologic analysis was performed to determine the peak flow discharges corresponding to the 2-year (2-yr), 10-year (10-yr), and 100-year (100-yr) rainfall events within the studied watersheds. This analysis employed the Unit Hydrograph methodology developed by the U.S. Soil Conservation Service (SCS) and implemented in the April 2008 in the HEC-HMS (Hydrologic Modeling System) computer program (version 3.2) [USACE, 2008].

Peak flow discharges were computed at each sub-basin outlet for two (2) conditions: the ***existing condition*** for the actual condition in the studied area and, the ***proposed condition*** for the proposed changes as part of the expansion of the existing landfill. Figures 3 and 4 show the sub-basins delimitation for the existing and proposed conditions, respectively.

The retention ponds were simulated using the reservoir simulation option included in the HEC-HMS model. This model uses the Elevation-Area and Elevation-Discharge relations to simulate the storage and peak flow discharge within the simulated ponds. For the existing ponds the Elevation-Area relation was developed using the topographic layout plan and the USGS topographic maps of Yauco and Punta Verraco. The Elevation-Discharge relations were determined using the weir flow equation. The existing ponds discharge through weirs structures and by overflow. The Appendix C includes the Elevation-Area and Elevation-Discharge relations calculated for the existing and proposed retention ponds within the studied area.

Watershed Description

Drainage watersheds were delimited using the USGS 1:20,000 topographic maps of Yauco and Punta Verraco, and the existing and proposed topographic layout plans include in Appendix H. Figures 3 and 4 show the drainage watershed for the existing and proposed conditions, respectively.

Existing Condition

For the existing condition a total of ten (10) sub-basins were delimited: ***Basin North*** (***BASN***) and ***Landfill North*** (***LN***) for the areas draining to an existing retention pond named ***Pond North*** located at the north limit of the property. ***Basin West*** (***BASW***) and ***Landfill West*** (***LW***) for drainage areas draining to an existing pond named ***Pond West***. ***Basin East*** (***BASE***) and ***Landfill East*** (***LE***) for drainage areas draining to an existing pond named ***Pond East***. ***Basin South West*** (***BASSW***), ***Basin South East*** (***BASSE***), and ***Landfill South*** (***LS***) for the areas draining to the existing ***Pond South 1***. This, at once, is interconnected with a pond named ***Pond South 2*** which receives runoff from ***Basin South*** (***BASS***).

For the existing condition, the hydrologic model was conceptualized as ***Basin North*** (***BASN***) and basin ***Landfill North*** (***LN***) draining to the existing ***Pond North***. Sub-basins ***Basin West*** (***BASW***) and ***Landfill West*** (***LW***) draining to the existing ***Pond West***. Sub-basins ***Basin East*** (***BASE***) and ***Landfill East*** (***LE***) draining to the existing ***Pond East***. Peak flows from ***Pond West*** and ***Pond East*** were routed using the

Muskingum method. The reaches were named ***RW*** and ***RE***, respectively, and were combined at combination point ***COM***. It was routed through a reach named ***RC***. ***Landfill South*** (***LS***), ***Basin South West*** (***BASSW***) and ***Basin South East*** (***BASSE***) drain into ***Pond South 1*** which drain into the existing ***Pond South 2***. Finally, ***Basin South*** (***BASS***) drains into ***Pond South 2***.

Proposed Condition

For the proposed condition a total of ten (10) sub-basins were delimited. The sub-basins were delimited as described for the existing condition. Only the sub-basins ***Landfill North*** (***LN***), ***Landfill West*** (***LW***), ***Landfill East*** (***LE***), and ***Landfill South*** (***LS***) were rearranged according to the proposed layout plan.

Pond Description

The retention ponds were simulated for the existing and proposed conditions. The Elevation – Area and the Elevation – Discharge relations were determined using the USGS 1:20,000 topographic maps of Yauco and Punta Verraco and, the current topographic survey of the site. Figures 5 and 6 show the retention ponds location for the existing and proposed conditions, respectively.

Existing Condition

As mentioned earlier, for the existing condition five (5) retention ponds were simulated: ***Pond North*** for the existing pond located at north limit of the landfill property, ***Pond***

West for the existing pond at the west limit, ***Pond East*** for the existing pond at the east limit and, ***Pond South 1*** and ***Pond South 2*** for the ponds located at the south limit. For the existing condition the outlet points for ***Pond West*** and ***Pond East*** were determined using the current topographic survey. The effective storage volume for ***Pond West*** and ***Pond East*** are controlled by the pond's dimensions and the topography. That is, when the structures are flooded the surrounding area operates as large storage volumes. These areas were established using the existing topographic survey and the USGS topographic maps. Figure 5 shows the location plan and the storage volume limits for the existing retention ponds.

Proposed Condition

The proposed condition scenario corresponds to the simulation of the minimum required combination of pond storage and outlet structure to avoid flood condition, minimize the sediments discharge and, provide a peak flow discharge equal or lower than the calculated for the existing condition.

Rainfall Data

Average point rainfalls depths for the 2- and 100-year events were taken from the Frequency Data Server available online and developed for NOAA's National Weather Services in NOAA Atlas 14 Volume 3, Version 3.0 [U.S. Department of Commerce, et. al 2006]. This data is included in Appendix B. Frequency based hypothetical storm used by the runoff model for the basin hydrologic response analysis were generated using the data in Table 1. The total storm is automatically distributed according to the specified depth per duration data for each analyzed exceedance probability (2- and 100-year) for 24 hours duration event. A triangular precipitation distribution (balanced storm) was constructed such that the depth specified for any duration occurs during the central part of the storm.

Table 1. Rainfall depths used in the HEC-1 model.

Duration, hr	0.083	0.25	1	2	3	6	12	24
2-yr, in	0.64	1.13	2.68	3.37	3.55	4.06	4.47	5.15
100-yr, in	1.03	1.81	4.31	6.19	7.14	9.69	13.34	17.56

Time of Concentration

The time of concentration (*Tc*) is defined as the time it takes to the most hydraulically remote drop of water to travel to the basin outlet. Time of Concentration within each sub-basin was estimated using the Kirpich equation,

$$T_c = 0.00013*(L^{0.77}/S^{0.385}) \qquad (1)$$

where, T_c is the time of concentration in hours, L is the length of the overland flow in feet, and S is the average overland slope in ft/ft. Appendix A includes the computations for the time of concentration for the studied basins for the existing and proposed conditions.

Soils

The soil types were taken from the new Soil Survey of the San Germán Area, Southwestern Puerto Rico, prepared by the Natural Resources Conservation Service [NRCS, 2007]. This document replaces the Soil Survey of the Lajas Valley Area issued in April 1965. The soils within the project area are mostly composed of hydrologic soil group D. The weighted curve numbers for each sub-basin are included in the Appendix A.

III. HYDROLOGIC AND RETENTION ANALYSES RESULTS

Existing Condition

Peak Discharges

The HEC-HMS modeling was performed using: 1) the rainfall depths; 2) the basin's areas; 3) the curve numbers (CN) and; 4) the basin's lag times ($0.6T_c$). Peak flow discharges for the 2-, 10-, and 100-year events were computed at each sub-basin outlet.

The hydrologic analysis results for the existing condition are shown in Table 2 for the 2-, 10- and 100-year rainfall events. The estimated CN values and peak flow discharges for each sub-basin are shown in this table (for antecedent moisture condition II [AMC II]). The Appendix A includes the CN and time of concentration calculation. The

Appendix D includes the hydrologic simulation results obtained from HEC-HMS model (simulation runs: ***EXISTING 2-YR, EXISTING 10-YR, EXISTING 100-YR***).

Table 2. Hydrologic analysis results for the 2-, 10-, and 100-year rainfall events for the existing condition.

BASIN	AREA, ac (mi^2)	CN	T_C, min	Q, cfs (cms)
2-YR				
BASW	36.89 (0.0576)	82	6	180 (5.10)
LW	1.92 (0.0030)	89	5	12 (0.34)
POND WEST				85 (2.41)
Routing (RW)				83 (2.35)
BASE	70.97 (0.1109)	82	8	318 (9.00)
LE	7.50 (0.0117)	87	5	43 (1.22)
POND EAST				1 (0.03)
Routing (RE)				1 (0.03)
Combination (COM)				84 (2.38)
Routing (RC)				83 (2.38)
LS	6.91 (0.0108)	89	5	42 (1.19)
BASSW	10.85 (0.0170)	82	5	57 (1.61)
BASSE	1.57 (0.0025)	82	5	8 (0.23)
POND SOUTH 1				109 (2.38)
BASS	7.61 (0.0119)	82	5	39 (1.10)
POND SOUTH 2				144 (4.08)
Combination (COMT)				144 (4.08)
LN	17.78 (0.0231)	88	5	88 (2.49)
Routing (RN)				81 (2.29)
BASN	45.45 (0.0710)	84	7	224 (6.34)
POND NORTH				273 (7.73)
10-YR				
BASW	36.89 (0.0576)	82	6	268 (7.59)
LW	1.92 (0.0030)	89	5	16 (0.45)

Table 2. Hydrologic analysis results for the 2-, 10-, and 100-year rainfall events for the existing condition (continued).

BASIN	AREA, ac (mi^2)	CN	T_C, min	Q, cfs (cms)
10-YR				
POND WEST				218 (617)
Routing (RW)				211 (5.97)
BASE	70.97 (0.1109)	82	8	476 (13.48)
LE	7.50 (0.0117)	87	5	61 (17.73)
POND EAST				32 (0.91)
Routing (RE)				32 (0.91)
Combination (COM)				211 (5.97)
Routing (RC)				206 (5.83)
LS	6.91 (0.0108)	89		57 (1.61)
BASSW	10.85 (0.0170)	82	5	84 (2.38)
BASSE	1.57 (0.0025)	82	5	12 (0.34)
POND SOUTH 1				268 (7.59)
BASS	7.61 (0.0119)	82	5	58 (1.64)
POND SOUTH 2				299 (8.47)
Combination (COMT)				299 (8.47)
LN	17.78 (0.0231)	88	5	121 (3.43)
Routing (RN)				111 (3.14)
BASN	45.45 (0.0710)	84	7	327 (9.26)
POND NORTH				396 (11.21)
100-YR				
BASW	36.89 (0.0576)	82	6	371 (10.51)
LW	1.92 (0.0030)	89	5	21 (0.59)
POND WEST				313 (8.86)
Routing (RW)				301 (8.52)
BASE	70.97 (0.1109)	82	8	659 (18.66)
LE	7.50 (0.0117)	87	5	81 (2.29)
POND EAST				264 (7.48)
Routing (RE)				263 (7.45)
Combination (COM)				486 (13.76)

Table 2. Hydrologic analysis results for the 2-, 10-, and 100-year rainfall events for the existing condition (continued).

BASIN	AREA, ac (mi^2)	CN	T_C, min	Q, cfs (cms)
100-YR				
Routing (RC)				480 (13.59)
LS	6.91 (0.0108)	89		75 (2.12)
BASSW	10.85 (0.0170)	82	5	116 (3.28)
BASSE	1.57 (0.0025)	82	5	17 (0.48)
POND SOUTH 1				555 (15.72)
BASS	7.61 (0.0119)	82	5	81 (2.29)
POND SOUTH 2				534 (15.12)
Combination (COMT)				584 (16.54)
LN	17.78 (0.0231)	88	5	160 (4.53)
Routing (RN)				148 (4.19)
BASN	45.45 (0.0710)	84	7	446 (12.63)
POND NORTH				423 (11.98)

Retention Analysis

A flow retention analysis was performed to determine the storage capacity of the existing ponds. The results for this analysis for the existing condition are shown in Table 3.

Results show that for the 2-, 10- and 100-yr events, ponds ***North***, ***South 1*** and ***South 2*** are flooded. For the 2-yr event, ponds ***South 1*** and ***South 2*** are flooded 0.17 meter each one. For the 10-yr event, ponds ***North***, ***South 1*** and ***South 2*** are flooded 0.06, 0.35 and 0.35 meter, respectively. For the 100-yr event, ponds ***North***, ***South 1*** and ***South 2*** are flooded 0.13, 0.60 and 0.60 meter, respectively.

For that reason, these structures should be modified to comply with two purposes: 1) provide the minimum required storage capacity and; 2) to produce an outflow discharge equal or lower than the calculated for the existing condition. As mentioned earlier, these modifications will be performed to comply with the 2- and 100-year design events. Detailed results from the hydrologic modeling of the existing condition are included in Appendix D.

Table 3. Retention analysis for the existing condition.

Structure	Peak Inflow, cfs, (m^3/s)	Peak Outflow, cfs (m^3/s)	Peak Elevation, ft (m)	Top Elevation, ft, (m)	Outlet Structure	Outlet Elevation, ft, (m)
2-YR						
Pond North	283 (8.01)	249 (7.05)	242.6 (73.94)	242.78 (74.00)	Weir	73.00 (239.50)
Pond West	191 (5.41)	85 (2.41)	388.6 (118.44)	393.70 (120.00)	Weir	118.00 (387.14)
Pond East	355 (10.05)	1 (0.03)	385.5 (115.76)	396.98 (121.00)	Weir	119.00 (390.42)
Pond South 1	109 (3.08)	109 (3.08)	318.8 (97.17)	318.24 (97.00)	Weir	97.00 (318.24)
Pond South 2	144 (4.08)	144 (4.08)	318.8 (97.17)	318.24 (97.00)	Weir	97.00 (318.24)
10-YR						
Pond North	406 (11.50)	401 (11.35)	243.0 (74.06)	242.78 (74.00)	Weir	73.00 (239.50)
Pond West	284 (8.04)	218 (6.17)	390.0 (118.87)	393.70 (120.00)	Weir	118.00 (387.14)
Pond East	529 (14.97)	32 (0.91)	391.0 (119.18)	396.98 (121.00)	Weir	119.00 (390.42)
Pond South 1	268 (7.59)	268 (7.59)	319.4 (97.35)	318.24 (97.00)	Weir	97.00 (318.24)
Pond South 2	300 (8.50)	299 (8.47)	319.4 (97.35)	318.24 (97.00)	Weir	97.00 (318.24)
100-YR						
Pond North	551 (15.60)	546 (15.46)	243.2 (74.13)	242.78 (74.00)	Weir	73.00 (239.50)
Pond West	391 (11.07)	313 (8.86)	390.7 (119.09)	393.70 (120.00)	Weir	118.00 (387.14)
Pond East	730 (20.67)	265 (7.50)	394.1 (120.12)	396.98 (121.00)	Weir	119.00 (390.42)
Pond South 1	555 (15.72)	555 (15.72)	320.2 (97.60)	318.24 (97.00)	Weir	97.00 (318.24)
Pond South 2	585 (16.57)	584 (16.54)	320.2 (97.60)	318.24 (97.00)	Weir	97.00 (318.24)

Proposed Condition

Peak Discharges

The hydrologic analysis results for the proposed condition are shown in Table 4 for the 2-, 10-, and 100-year rainfall events, respectively. The estimated CN values and peak flow discharges for each basin are shown in this table (for antecedent moisture condition II [AMC II]). Appendix A includes the CN and lag time calculations. Appendix D includes the hydrologic simulation results obtained from HEC-HMS model (simulation runs: ***PROPOSED 2-YR, PROPOSED 10-YR, PROPOSED 100-YR***).

Table 4. Hydrologic analysis results for the 2-, 10-, and 100-year rainfall events for the proposed condition.

BASIN	AREA, ac (mi^2)	CN	T_C, min	Q, cfs (cms)
2-YR				
BASW	36.89 (0.0576)	82	6	180 (5.10)
LW	1.92 (0.0030)	89	5	12 (0.34)
POND WEST				86 (2.44)
Routing (RW)				85 (2.41)
BASE	58.01 (0.0906)	82	9	304 (8.61)
LE	15.64 (0.0244)	89	5	58 (1.64)
POND EAST				1 (0.03)
Routing (RE)				1 (0.03)
Combination (COM)				85 (2.41)
Routing (RC)				84 (2.38)
LS	6.91 (0.0108)	89	5	43 (1.22)
BASS	7.61 (0.0119)	82	5	39 (1.10)
BASSW	10.85 (0.0170)	82	5	56 (1.59)
BASSE	1.57 (0.0025)	82	5	8 (0.23)
POND SOUTH				102 (2.89)

Table 4. Hydrologic analysis results for the 2-, 10-, and 100-year rainfall event for the proposed condition (continued).

BASIN	AREA, ac (mi^2)	CN	T_C, min	Q, cfs (cms)
2-YR				
Combination (COMT)				102 (2.89)
LN	21.21 (0.0331)	89	5	74 (2.10)
Routing (RN)				68 (1.93)
BASN	43.84 (0.0685)	86	7	206 (5.83)
POND NORTH				5 (0.14)
10-YR				
BASW	36.89 (0.0576)	82	6	269 (7.62)
LW	1.92 (0.0030)	89	5	18 (0.51)
POND WEST				220 (6.23)
Routing (RW)				213 (6.03)
BASE	58.01 (0.0906)	82	9	458 (12.97)
LE	15.64 (0.0244)	89	5	79 (2.24)
POND EAST				37 (1.05)
Routing (RE)				37 (1.05)
Combination (COM)				213 (6.03)
Routing (RC)				210 (5.95)
LS	6.91 (0.0108)	89	5	59 (1.67)
BASS	7.61 (0.0119)	82	5	58 (1.64)
BASSW	10.85 (0.0170)	82	5	83 (2.35)
BASSE	1.57 (0.0025)	82	5	12 (0.34)
POND SOUTH				284 (8.04)
Combination (COMT)				284 (8.04)
LN	21.21 (0.0331)	89	5	101 (2.86)
Routing (RN)				93 (2.63)
BASN	43.84 (0.0685)	86	7	315 (8.92)
POND NORTH				173 (4.90)
100-YR				
BASW	36.89 (0.0576)	82	6	371 (10.51)
LW	1.92 (0.0030)	89	5	24 (0.68)

Table 4. Hydrologic analysis results for the 2-, 10-, and 100-year rainfall event for the proposed condition (continued).

BASIN	AREA, ac (mi^2)	CN	T_C, min	Q, cfs (cms)
100-YR				
POND WEST				316 (8.95)
Routing (RW)				304 (8.61)
BASE	58.01 (0.0906)	82	9	635 17.98)
LE	15.64 (0.0244)	89	5	104 (2.94)
POND EAST				281 (7.96)
Routing (RE)				280 (7.93)
Combination (COM)				497 (14.07)
Routing (RC)				492 (13.93)
LS	6.91 (0.0108)	89	5	77 (2.18)
BASS	7.61 (0.0119)	82	5	81 (2.29)
BASSW	10.85 (0.0170)	82	5	115 (3.26)
BASSE	1.57 (0.0025)	82	5	17 (0.48)
POND SOUTH				570 (16.14)
Combination (COMT)				570 (16.14)
LN	21.21 (0.0331)	89	5	133 (16.14)
Routing (RN)				123 (3.48)
BASN	43.84 (0.0685)	86	7	439 (12.43)
POND NORTH				423 (11.98)

Retention Analysis

For the proposed condition the existing ponds ***North***, ***South 1*** and ***South 2*** were modified to avoid flooding conditions and produce an outflow discharge equal or lower than the calculated for the existing condition for the 2- and 100-yr events.

During the proposed condition, the depth in pond ***North*** was increased 1.0 meter. The ponds ***South 1*** and ***South 2*** were joined in one single pond named ***Pond South***. The outlet structures for the proposed modified structures are composed weirs. Figure 6 shows a plan view of the proposed retention structures.

The results of the retention pond analysis for the proposed condition are shown in Table 5. For the proposed condition a freeboard of 0.30 and 0.35 meter was calculated for the 100-yr event in ponds ***North*** and ***South***, respectively.

Table 5. Retention analysis for the proposed condition.

Structure	Peak Inflow, cfs, (m^3/s)	Peak Outflow, cfs (m^3/s)	Peak Elevation, ft (m)	Top Elevation, ft, (m)	Outlet Structure	Outlet Elevation, ft, (m)
2-YR						
Pond North	256 (7.25)	5 (0.14)	239.4 (72.97)	242.78 (74.00)	Weir	73.00 (239.50)
Pond West	193 (5.46)	86 (2.43)	388.6 (118.44)	393.70 (120.00)	Weir	118.00 (387.14)
Pond East	354 (10.02)	1 (0.03)	385.9 (117.62)	396.98 (121.00)	Weir	119.00 (390.42)
Pond South	146 (4.13)	102 (2.88)	317.9 (96.90)	321.52 (98.00)	Weir	96.70 (317.26)
10-YR						
Pond North	380 (10.76)	174 (4.92)	240.8 (73.40)	242.78 (74.00)	Weir	73.00 (239.50)
Pond West	286 (8.10)	220 (6.23)	390.0 (118.87)	393.70 (120.00)	Weir	118.00 (387.14)
Pond East	525 (14.87)	37 (1.05)	391.1 (119.20)	396.98 (121.00)	Weir	119.00 (390.42)
Pond South	305 (8.63)	284 (8.04)	318.8 (97.17)	321.52 (98.00)	Weir	96.70 (317.26)
100-YR						
Pond North	525 (14.86)	423 (11.98)	241.8 (73.70)	242.78 (74.00)	Weir	73.00 (239.50)
Pond West	394 (11.16)	316 (8.95)	390.8 (119.11)	393.70 (120.00)	Weir	118.00 (387.14)
Pond East	723 (20.47)	281 (7.96)	394.3 (120.18)	396.98 (121.00)	Weir	119.00 (390.42)
Pond South	597 (16.91)	570 (16.14)	320.4 (97.65)	321.52 (98.00)	Weir	97.00 (318.24)

The proposed ponds will reduce the peak flow discharge at the north and south limits. These flows are reduced from 249 ft^3/s to 5 ft^3/s and from 546 ft^3/s to 423 ft^3/s for the 2-

and 100-yr events, respectively, in the north limit (***Pond North*** outflow). The peak flow is reduced from 149 ft^3/s to 102 ft^3/s and from 584 ft^3/s to 570 ft^3/s for the 2- and 100-yr events, respectively, in the south limit (***Pond South*** outflow). Detailed results from the hydrologic modeling of the proposed condition are included in Appendix D.

The recommended outlet structure for pond ***North*** consists of two weirs located at the north limit. The first weir has a length of 7.0 meters and an invert elevation of 73.00 m (msl). The second weir has a length of 12.0 meters and an invert elevation of 73.60 m (msl). Figure 22 shows the proposed outlet structure for pond ***North***.

The recommended outlet structure for pond ***South*** consists of two weirs located at the north limit. The first weir has a length of 6.5 meters and an invert elevation of 96.70 m (msl). The second weir has a length of 9.0 meters and an invert elevation of 97.70 m (msl). Figure 23 shows the proposed outlet structure for pond ***South***.

IV. FLOOD PROFILE ANALYSIS

A hydraulic analysis was performed for the natural drainages crossing through the property. The purpose of this analysis was to determine the water surface levels for the 2- and 100-year rainfall events. All the peak flow values used at the River Analysis

System (HEC-RAS 4.0) model (existing and proposed) were taken from the HEC-HMS (Hydrologic Modeling System) computer program (version 3.2).

Methodology

The hydraulic analysis was performed using the River Analysis System (HEC-RAS 4.0) model, developed by the U.S. Army Corps of Engineers [USACE, 2008]. Two scenarios were analyzed: 1) Existing and 2) Proposed. These scenarios were computed to determine the water surface elevations along the natural drainage around the landfill area, during the 2- and 100-year flood events. As stated by the Planning Regulation No. 13 of the PRPB [2005a], the 100-year flood levels along the stream should not be increased more than 0.15 meter at any location, as a result from the proposed expansion to the existing landdfill.

Nineteen (19) cross sections were field surveyed (1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18 and, 19) along the natural drainages under study. These field survey data was submitted by Mr. Ricardo Flores Ortega, P.E., representative of RF Engineering, P.S.C. All these data was used to construct the hydraulic model geometry. A copy of the field surveyed data is included in Appendix H. The cross sections location plan is shown in Figure 7.

Coefficients Values

Manning's roughness coefficients were determined by visual inspection of the site. The values used for the existing condition model were 0.03 in the main channel and 0.04 in the floodplain. Contraction and expansion coefficients of 0.1 and 0.3, respectively, were used for gradual transitions and 0.2 and 0.4 for abrupt transitions.

Existing Condition Model

The existing condition model was developed to simulate the natural drainage conditions based on the submitted field data and the HEC-HMS [version 3.2] peak discharge data.

Computed water levels and velocities at each cross section along the ***Natural Drainage*** for the 2- and 100-year flood events are presented in Table 4. Figures 8, 9, 10, 15, 16 and, 17 shows the water surface profiles for the 2- and 100-year flood events for the ***Natural Drainage*** at ***North***, ***South***, ***East*** and, ***West***. The limits of the 2- and 100-year frequency floodplain, occurring inside and outside of the property limits are shown in Figures 11 and 18, respectively. HEC-RAS input and output data (geometry: ***EXISTING***) is included in Appendix E.