Table 4: Summary of Hydrologic Input Data

Drainage Basin ID	Drainage Area (acre)	Existing/ Proposed Pervious RCN	Existing Percent Impervious (%)	Proposed Percent Impervious (%)	Proposed Basin Slope (%)	Proposed Basin Width (feet)
10	77.1	74	31%	31%	0.6%	1019
20	28.8	74	34%	34%	0.8%	995
30	19.3	74	40%	40%	1.0%	454
40	4.1	74	44%	44%	1.1%	176
50	3.6	74	21%	21%	1.9%	301
60	5.2	74	8%	8%	1.7%	269
70	9.2	74	100%	100%	1.3%	331
80	14.3	71	99%	100%	1.9%	568
90	8.9	74	24%	82%	2.0%	286
100	19.3	74	82%	43%	0.6%	573
110	6.1	74	43%	29%	1.3%	531
120A	13.95	74	92%	85%	1.6%	409
120B	2.43	74	92%	50	3.3	165
130	4.5	74	92%	47%	3.7%	207
142	9.9	74	44%	32%	5.0%	608
144-A	0.84	74	2%	100%	0.5%	266
144-B	0.86	74	2%	100%	0.5%	302
144-C	0.75	74	2%	100%	0.5%	239
144-D	0.88	74	2%	100%	0.5%	252
144-E	0.51	74	2%	100%	0.5%	126
144-F	0.47	74	2%	100%	0.5%	201
144-G	0.54	74	2%	100%	0.5%	220
144-H	1.74	74	2%	42%	0.5%	320
146-A	0.81	74	2%	100%	0.5%	86
146-B	1.71	74	2%	100%	0.5%	184
146-C	1.53	74	2%	100%	0.5%	164
146-D	1.99	74	2%	100%	0.5%	193
146-E	0.67	74	2%	28%	0.5%	274
148	1.08	74	2%	10%	16.2%	171

2.2.5 NRCS Curve Numbers

The NRCS curve number approach was used in computing the runoff response in SWMM. Runoff curve numbers (RCNs) were generated for the pervious areas of the sub-basins using the NRCS document entitled Urban Hydrology for Small Watersheds, dated June 1986 and commonly referred to as TR-55. This method relates the drainage characteristics of soil group, land use category, and antecedent moisture conditions to assign a runoff curve number. The runoff curve number and an estimate of the initial surface moisture storage capacity are used to calculate a total runoff depth for a storm in a basin.

2.2.6 *Channel/Storage Routing*

Flood peaks attenuate, or reduce, as they travel downstream due to the storage characteristic of the channel itself. Channel routing was simulated in the hydraulic block of SWMM. Routing was modeled using dynamic wave routing. Dynamic wave routing uses the actual shape and condition of the stream channel input into the hydraulic model to calculate the attenuated downstream flows.

2.2.7 *Summary of Hydrologic Model Results*

The EPA SWMM model was used to compute peak runoff for the 2-, 10-, 25-, 50- and 100- year design storms for the existing and proposed conditions. The results of the existing conditions hydrologic model are summarized in Table 5. A CD containing the digital files for the SWMM model is included in Appendix E.

Table 5: Comparison of Peak Flows at Radar Road

Condition	Storm Event				
	WQ Event (cfs)	2-year (cfs)	10-year (cfs)	25-year (cfs)	100-year (cfs)
Existing	12	63	108	138	185
Proposed	40	288	420	466	513

Although Session Law 2012-200 precludes the project from having to provide detention, a detailed hydrologic and hydraulic evaluation was performed to confirm there are no adverse impacts to downstream properties with regards to flooding. A summary of this evaluation is found in the following Hydraulics section of the report.

2.3 Hydraulics

EPA SWMM 5.0 was chosen as the hydrologic/hydraulic model because of its ability to model complex drainage systems and to evaluate downstream flooding. The project involves the construction of a single central high flow rate bioretention pond to provide water quality treatment for the proposed site development. The airport desires to reduce the potential for bird strikes by eliminating two existing wet ponds referred to in this report as the fire suppression wet pond and the existing HAECO site wet pond. The existing conditions SWMM model attenuates peak flows through these two ponds to more accurately determine the proposed projects effects on peak flows. To fully evaluate the project’s impacts on downstream properties, the SWMM model was extended through the Harris Teeter distribution site and immediate downstream open channel. In addition, a HEC-RAS model was developed to provide a quality control measure for the changes to water surface elevations developed using EPA SWMM.

2.3.1 *Energy Loss Coefficients*

Contraction and expansion of flow produces energy losses caused by transitioning. The magnitude of these losses is related to the velocity and the estimated loss coefficient. Where the transitions are gradual, the losses are small. At abrupt changes in cross-sectional area, the losses are higher. Energy losses resulting from expansion are greater than losses associated with contraction. Energy loss coefficients used for the SWMM models are presented in Table 6:

Table 6: Energy Loss Coefficients

Type of Transition	Expansion	Contraction
None	0	0
Manhole/Inlet	0.35	0.25
Culvert	1.0	0.9 - Projecting from fill CMP
Open Channel	0.3	0.1

Additional energy losses for structures having bends were divided between the two joining pipes. The bend losses used for this project are based on NCDOT values, and are shown below in Table 7.

Table 7: Bend Loss Coefficients

Angle (°)	Loss Coefficient	Angle (°)	Loss Coefficient
90	0.70	40	0.38
80	0.66	30	0.28
70	0.61	25	0.22
60	0.55	20	0.16
50	0.47	15	0.10

2.3.2 *Starting Water Surface Elevation*

The downstream limit of the HAECO Facility Improvements study area is located near the mouth of Horsepen Creek. The starting water surface elevations for the SWMM models were generated using the normal depth method based of the channel slope at the outfall (0.008 ft/ft).

2.3.3 *Model Run Descriptions*

The EPA SWMM model was used to compute flood elevations at each structure located in the HAECO Facility Improvements project study area for the water quality event, 2-, 10-, 25-, 50- and 100-year storm events. A digital copy of the SWMM model is included on the CD provided in Appendix E.

2.3.4 *Hydraulic Evaluation of Radar Road*

The following table summarizes the performance of the twin 8.9′ x 6.6′ corrugated metal pipe (CMP) arches at Radar Road:

Table 8: Culvert Performance for at Radar Road

Flood Frequency	Culvert Invert Elevation (feet NAVD 1988)	Roadway Elevation (feet NAVD 1988)	Existing Water Surface Elevations (feet NAVD 1988)	Proposed Water Surface Elevations (feet NAVD 1988)
WQ Event	831.29	840.90	831.76	832.14
2-Year	831.29	840.90	832.38	834.07
10-Year	831.29	840.90	832.72	835.59
25-Year	831.29	840.90	832.89	836.23
100-Year	831.29	840.90	833.19	837.18

Although there are increases to peak flows, the downstream drainage system can accommodate these increased flows. The existing twin 9.8′ by 6.6′ arched CMPs pass 896 cfs when flowing full. The 104″ diameter closed CMP located at the Harris Teeter distribution center conveys 753 cfs when flowing full. **The Radar Road culverts and Harris Teeter closed pipe will be flowing approximately half full during a 100-year storm event therefore there are no impacts to the performance of either of these drainage systems.**

2.3.5 Evaluation of Downstream Flooding

Approximately 85 feet from the top of bank (in the left overbank) is the toe of the water quality pond embankment for the Harris Teeter distribution center. For this reason, a check was made to confirm that the additional flows from the HAECO Facility Improvements project would not cause adverse impacts to the existing water quality pond embankment. Table 9 summarizes the size, slope and hydraulic characteristics of the channel located immediately downstream of Harris Teeter.

Table 9: Hydraulic Summary of Harris Teeter Open Channel

Bottom Width (feet)	Top Width (feet)	Depth (feet)	Side Slopes (ft/ft)	Channel Slope (ft/ft)	Channel Capacity (cfs)	Floodplain Capacity (cfs)
10	25	4	2:1	0.014	300	2,150

Assumed Manning's 'n' value = 0.06
Floodplain capacity is the flow needed to inundate the toe of the existing Harris Teeter pond

As shown in Table 9, the existing channel can almost convey the proposed conditions 10-year flood without overtopping its banks. The flow needed to inundate the lowest toe elevation of the Harris Teeter pond is 2,150 cfs which is significantly more than the 513 cfs that will leave the proposed HAECO site.

This existing open channel extends approximately 290 feet downstream of the Harris Teeter culvert prior to entering Horsepen Creek which is a FEMA stream with an 832-acre (1.3 square miles) drainage area and 100-year peak flow of 1,598 cfs. On the upstream side of Radar Road (along Horsepen Creek), the drainage area increases to

1,344 acres (2.1 square miles) with a 100-year peak flow of 3,018 cfs. A field walk and inspection of aerial topography shows this reach of Horsepen Creek does not have any insurable structures located in the reach upstream of Radar Road where the 329 cfs increase would be roughly 10% of the total flow in Horsepen Creek. **As shown in this report, the proposed HAECO Facility Improvements Project will not adversely cause flooding downstream to an insurable structure or road.**

Figure 1: FEMA FIRM Panel

2.3.6 *Closed Drainage Systems*

Closed systems were designed to pass the 10-year flood without surcharging the pipe. With the exception of the SCM underdrain system, all drainage pipes are reinforced concrete (RCP).

2.3.7 *Outfall Protection for Closed Drainage System*

Rip-rap pads are proposed at two locations in the high flow rate bioretention. These outfalls are located where the flows enter back into the natural drainage system or the bioretention ponds. The NY DOT method was used to design the length, width, depth and size of the rip-rap pads. Appendix F shows the calculation used to size the rip-rap pads.

3.1 Overview

To satisfy the water quality requirements outlined in Session Law 2012-200, a proposed 0.8-acre high flow rate bioretention pond is being proposed. Session Law 2012-200 requires runoff generated from the 1st inch of rainfall for a development project shall be infiltrated into the ground. There are no specific requirements to remove total suspended solids (TSS), nitrogen, or phosphorus. In addition, there are no requirements to detain the 1-year or any other storm event to at or below pre-project conditions. As shown in this report, the proposed high flow rate bioretention pond exceeds the minimum infiltration requirements set forth in Session Law 2012-200.

3.1.1 Proposed Impervious Areas

The separately attached construction plans and concept plan provided in Appendix A show the proposed pond, new and existing impervious areas, location of flow splitters and overall site layout. The following table summarizes the proposed impervious areas associated with the HAECO Facility Improvements project:

Table 10: Minimum Area of Impervious Cover Required for Treatment

Location	Impervious Cover (acres)
Proposed Hangar	5.06
Proposed Apron	5.11
Proposed Fire Access Roads	0.32
Proposed Sidewalk	0.09
Existing HAECO Site to the East	14.03
	TOTAL = 24.61 acres

Because the existing fire suppression pond is being abandoned as part of this project, the proposed SCM will need to be designed to accept runoff from the system currently going to the existing fire suppression pond. The stormwater runoff generated in sub-basins 60, 70 and 80 will be redirected into the proposed SCM. Appendix C highlights the areas that will drain to the pond along with a breakdown for the impervious area contributed from each sub-basin. As a result, an additional 20.49 acres of impervious area will be infiltrated in the proposed SCM as shown in the following table:

Table 11: Proposed Impervious Cover to SCM

Location	Impervious Cover (acres)
Proposed Hangar	3.35
Proposed Apron	5.11
Proposed Fire Access Roads	0.32
Proposed Side Walk	0.09
Existing HAECO Site to the East	11.92
Sub-Basin 60	0.42
Sub-Basin 70	9.22
Sub-Basin 80	14.09
	TOTAL = 44.52 acres

In total, the proposed SCM will have a contributing drainage area of 54.0 acres with 44.52 acres of impervious cover.

3.1.2 High Flow Rate Bioretention Pond Design Criteria

The State BMP Manual does not specifically have a set of design guidelines for a high flow bioretention pond so the following guidelines were used in the design of the proposed high flow bioretention pond:

- Infiltrate 100% of the runoff generated from the 1st inch of rainfall;
- Side slopes shall be no steeper than 3(H):1(V);
- SCM shall be located in a recorded drainage easement;
- A bypass or internal overflow is required for bypassing storm flows in excess of the design flow;
- Media permeability shall be between 6 and 10 inches per hour with a targeted detention time of 10 to 15 hours for infiltrating the water quality volume;
- Ponding depth for the water quality event shall be limited to 4.0 feet;
- Media depth will be 2 feet for each of the two soil media zones of the bioretention pond;
- An underdrain shall be located under the soil media to keep the pond dry and prevent groundwater from entering the pond; and
- A rip-rap energy dissipater shall be located at the outfall of each pipe entering the pond.

3.1.3 Water Quality Volume (WQV)

The volume of runoff generated from the 1st inch of rainfall was calculated using an in-house spreadsheet based on the Schuler Simple Method. This spreadsheet shows the calculated water quality volume along with proposed SCMs stage-storage sizing (see Appendix G). The following table summarizes the minimum required volume along with the provided volume:

Table 12: Calculated Storage Volumes

Description	Impervious Area (acres)	Surface Runoff (ft^3)
Required Area for Treatment	25.01	72,582
Compensatory Treatment of Sub-basins 60, 70, 80	23.72	82,679
Total Provided Area for Treatment	44.52	155,260
Net Credit for WQ Treatment	19.51	82,678

As shown in Table 12, the proposed high flow rate bioretention pond will infiltrate an additional 82,678 cubic feet of runoff and 19.51 acres of impervious cover more than required.

3.1.4 Pond Design Summary

A concrete riser structure is proposed to control flows leaving the high flow rate bioretention pond. The primary spillway will include the following elements: a poured, reinforced concrete box riser and reinforced concrete outfall pipe with gaskets at joints. Because the weir length on these structures is 12′ and the flows entering the ponds are generally very small, there were no emergency spillways proposed for the pond. The following is a summary of the design for the proposed high flow rate bioretention pond (See the separately attached plan set for additional details):

- Surface Area: The proposed high flow rate bioretention pond is larger than the minimum size needed to achieve the water quality goals of the project. The surface area of the pond was achieved by targeting a pond depth of less than 4.0 feet and a detention time between 10 and 40 hours. The more well-draining the soils the smaller the footprint of the pond needed to drain the pond in approximately 10 hours. As shown in this report, the surface area that drains the pond in approximately 11 hours is 28,005 square feet (0.64 acres).
- Primary Outfall: A concrete box riser with an outside dimension of 7′x7′ is proposed with a primary weir elevation set at 583.75 feet NAVD 1988. The total weir length of the primary outfall is 18 feet (four 4.5′ long weirs).
- Emergency Overflow: The primary spillway was designed to pass flow larger than the 100-year flood without overtopping the top of dam. A 15′ wide rip-rap lined emergency overflow will convey flows over the top of dam should the riser be clogged for some unforeseen reason. This emergency spillway ties into a grass lined swale until it reaches an 18″ RCP with a flared section opening.
- Top of Dam: The top of dam is set at elevation 855.25 feet which is approximately 1.7 feet above the 100-year flood elevation. The total dam height measured from the toe of the embankment on the downstream side is approximately 3.0 feet.

The following table summarizes the water surface elevations at the proposed pond for the water quality event, 1-, 10- and 100-year floods:

Table 13: Water Surface Elevations at Proposed Pond

Water Quality Event	1-Year Storm (NAVD ′88)	10-Year Storm (NAVD ′88)	100-Year Storm (NAVD ′88)
853.28	853.48	853.50	853.52

<u>Riser</u>

The riser detail provided in the separately attached plan set shows the 6′x6′ concrete box to control water surface elevations inside the proposed SCM. The primary spillway was set at elevation 853.30 feet which is the dynamic elevation calculated inside EPA SWMM for the water quality storm event (an NRCS Type II distribution with 1.0 inches of rainfall). The riser has a 42″ diameter RCP barrel that conveys flow from the pond to a new 48″ diameter closed drainage system. This 48″diameter closed system conveys the by-pass flows for larger storm events from the eastern side of the existing HAECO development. An anti-floatation calculation (See Appendix H) was performed for the pond riser resulting in a factor of safety of 1.22. This calculation ignores the friction forces of the underlying soil and therefore a factor of safety larger than 1.22 would be achieved in real conditions. Because this is a dry pond and water levels will rarely reach 6″ above the crest of the weir therefore a factor of safety of 1.22 is acceptable.

Flow Splitters

Three flow splitters are proposed to divert stormwater runoff from the proposed closed drainage system into the high flow rate bioretention pond. For water quality rainfall event (1.0 inch of rain), 100% of the runoff generated will flow directly into the high flow rate bioretention pond. Inside each flow splitter is a weir wall that will direct flows generated from larger storm events into a closed by-pass pipe. The elevation of this weir wall was calculated in EPA SWMM by iteratively adjusting the elevation of the wall until no flow was being diverted in the water quality rainfall event.

The splitter box located just north and west of the pond (Structure 5) will require a special design. Flows that go over the weir wall will drop into a concrete manhole structure and eventually into the sites main 72 inch diameter RCP. The following table summarizes the key elevations for the three proposed concrete flow splitters:

Table 14: Summary of Flow Splitter Design

Splitter #	Pipe to Pond Invert Elevation (feet NAVD 1988)	Weir Wall Height (ft)	Pipe Sizes Entering Splitter Box (in)	Pipe to Pond Diameter (in)
1 (structure 20)	872.21	0.85	30″	15″
2 (structure 25)	855.10	2.75	48″	24″
3 (structure 5)	851.68	1.45	54″ and 42″	30″

The separately attached design plans provide additional details on the size and construction of the flow splitters being used for this project.

Table 15: Flow Splitter Performance

Splitter #	Water Quality Event		10-Year Storm Event	
	Flow To Pond (cfs)	Flow Around Pond (cfs)	Flow To Pond (cfs)	Flow Around Pond (cfs)
1 (structure 20)	4	0	6	17
2 (structure 25)	10	0	20	50
3 (structure 5)	27	0	40	121

As shown in Table 15, approximately 67% of the peak flows from the larger storm events will be diverted around the pond.

<u>Detention Time and Soil Media for High Flow Rate Bioretention Pond</u>
Per discussions with DEQ, it was agreed that the proposed high flow rate bioretention pond would detain the water quality event for between 10 and 40 hours. To achieve this goal, a well-draining sand media is needed that promotes infiltration at a rate that is not too quick (3 or 4 hours) and not too long (over 40 hours). With an assumed infiltration rate of 10 inches per hour for this well-draining sand, a footprint was iteratively determined until the time to drain the pond was 10 hours. This area was calculated to be 14,563 square feet. For those areas outside the well-draining sands an infiltration rate of 2 inches/hour was assumed. As shown in Appendix I, the combined flow rate passing through the soil media and leaving the pond is 3.9 cfs.

For the area of well-draining sand, the construction of the high flow rate bioretention pond will mimic the design of a PGA golf green. It is assumed that the best draining soils that can be stockpiled from the onsite borrow area will be used for those areas outside the well-draining sands. At a minimum this media in Zone 1 will have a permeability of 2 inches/hour. The following is a summary of the construction for the area of the pond that mimics the PGA golf green:

Option #1 for Zone 2 (No. 57 Stone at base)
- 12″ thick base of No. 57 stone (approximately ¾″ in size)
- 4″ of washed sand
- 2′ of well-draining sand-soil mix (with a permeability of 10 inches/hour)

Option #2 for Zone 2 (Pea Gravel at base)
- 12″ thick base of peak gravel (100% passage of 3/8″ sieve)
- 2′ of well-draining sand-soil mix (with a permeability of 10 inches/hour)

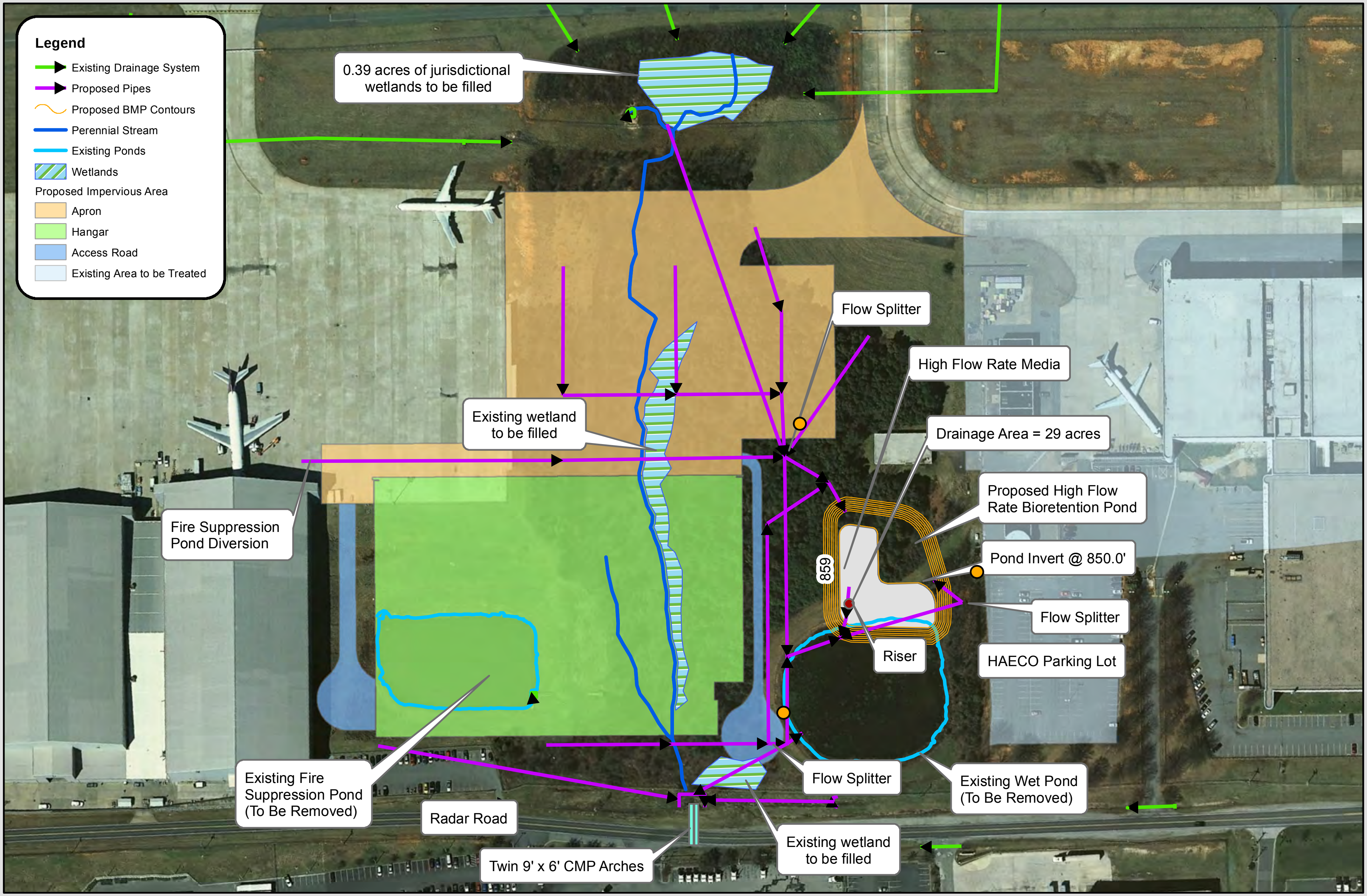
Specifications for the two soil zones will be prepared at final design.

<u>Channel Liner</u>
As shown on the separately attached design plans, two shallow rip-rap lined swales are proposed to convey runoff from small storm events to the side of the SCM with the riser. The swales were designed to be relatively shallow (1 foot in depth) and flat in order to promote infiltration. It was assumed that the entire pond bottom would be inundated fairly quickly and the need to size a large swale to minimize erosion would not be necessary. A calculation for the channel liner design is provided in Appendix J.

<u>Maintenance and Operation Procedures</u>
A maintenance and operation plan for the bioretention facilities has been included with this report as Appendix K.

3.2 Conclusion

As shown in this report, the proposed high flow rate bioretention pond is designed to bring the HAECO Facility Improvements project at the Piedmont-Triad International Airport in compliance with the State's requirements for water quality as outlined in Session Law 2012-200. By diverting runoff for the water quality rainfall event from basins 60, 70 and 80 into the proposed SCM, the airport is providing treatment for 44.52 acres of impervious cover. As shown in this report, the proposed SCM is providing approximately 19.51 acres more than the minimum required amount. The airport would like to request a water quality credit to offset the need to provide or minimize treatment with a future onsite development.



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Proposed Concept Plan - Appendix A

High Flow Rate Bioretention Pond

Piedmont-Triad International Airport
HAECO Site Development

200 100 0 200 Feet

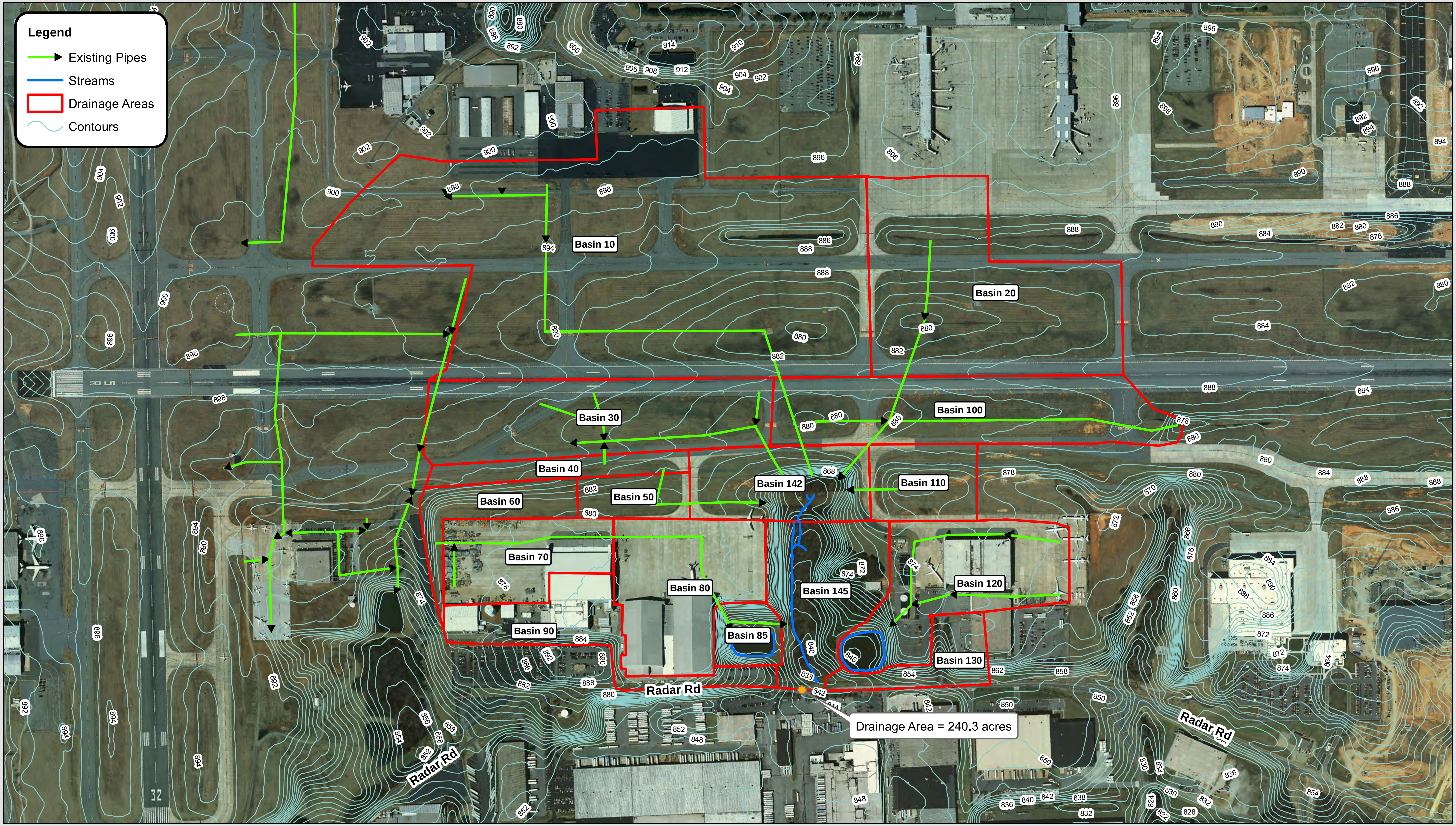
1 inch = 200 feet

Project: HAECO Facility Improvement @ PTIA, Greensboro, NC
Prepared by: DJK
Date: November 9, 2015

SWMM Input Data

EXISTING CONDITIONS SUBBASINS								
SWMM Sub-Basin ID	**Pervious RCN**	**Area (acres)**	**Area (sq. ft.)**	**Flow Length (ft.)**	**Width (ft.)**	**Elevation Change (ft.)**	**Basin Slope (%)**	**Percent Impervious (%)**
10	74	77.1	3356329	3294	1019	21	0.64%	31%
20	74	28.8	1253061	1259	995	11	0.83%	34%
30	74	19.3	842697	1856	454	19	1.03%	40%
40	74	4.1	180429	1027	176	12	1.12%	44%
50	74	3.6	156421	520	301	10	1.92%	21%
60	74	5.2	225908	841	269	14	1.66%	8%
70	74	9.2	401743	1215	331	16	1.34%	100%
80	71	14.4	627272	1095	573	21	1.91%	100%
85	71	3.0	131013	338	388	36	10.65%	24%
90	74	8.9	388935	1359	286	27	1.98%	82%
100	74	19.3	840092	1465	573	9	0.58%	43%
110	74	6.1	263574	496	531	7	1.31%	29%
120	74	15.3	665778	958	695	25	2.56%	92%
130	74	4.5	195877	944	207	35	3.65%	44%
142	74	9.4	407511	707	576	35	4.95%	26%
145	74	12.2	529689	1513	350	49	3.25%	2%
		240.27						

PROPOSED CONDITIONS SUBBASINS								
SWMM Sub-Basin ID	**Pervious RCN**	**Area (acres)**	**Area (sq. ft.)**	**Flow Length (ft.)**	**Width (ft.)**	**Elevation Change (ft.)**	**Basin Slope (%)**	**Percent Impervious (%)**
10	74	77.05	3356329	3294	1019	21	0.6%	31%
20	74	28.77	1253061	1259	995	11	0.8%	34%
30	74	19.35	842697	1856	454	19	1.0%	40%
40	74	4.14	180429	1027	176	12	1.1%	44%
50	74	3.59	156421	520	301	10	1.9%	21%
60	74	5.19	225908	841	269	14	1.7%	8%
70	74	9.22	401743	1215	331	16	1.3%	100%
80	71	14.09	613567	1095	560	21	1.9%	100%
90	74	8.93	388935	1359	286	27	2.0%	82%
100	74	19.29	840092	1465	573	9	0.6%	43%
110	74	6.05	263574	496	531	7	1.3%	29%
120A	74	13.95	607823	1487	409	25	1.6%	85%
120B	74	2.43	105911	640	165	21	3.3%	50%
130	74	4.50	195877	944	207	35	3.7%	47%
142	74	9.36	407511	707	576	35	5.0%	32%
144-A	74	0.84	38104	143	266	0.715	0.5%	100%
144-B	74	0.86	38293	127	302	0.635	0.5%	100%
144-C	74	0.75	32703	137	239	0.685	0.5%	100%
144-D	74	0.88	38306	152	252	0.76	0.5%	100%
144-E	74	0.51	22097	175	126	0.875	0.5%	100%
144-F	74	0.47	20479	102	201	0.51	0.5%	100%
144-G	74	0.54	23709	108	220	0.54	0.5%	100%
144-H	74	1.74	75931	237	320	1.185	0.5%	42%
146-A	74	0.81	35111	406	86	2.03	0.5%	100%
146-B	74	1.71	74691	406	184	2.03	0.5%	100%
146-C	74	1.53	66657	406	164	2.03	0.5%	100%
146-D	74	1.99	86824	450	193	2.25	0.5%	100%
146-E	74	0.67	29025	106	274	0.53	0.5%	28%
148	74	1.08	47109	275	171	44	16.2%	10%
		240.27						



Existing Conditions Drainage Area Map - Appendix C

Piedmont-Triad International Airport

HAECO Site Development

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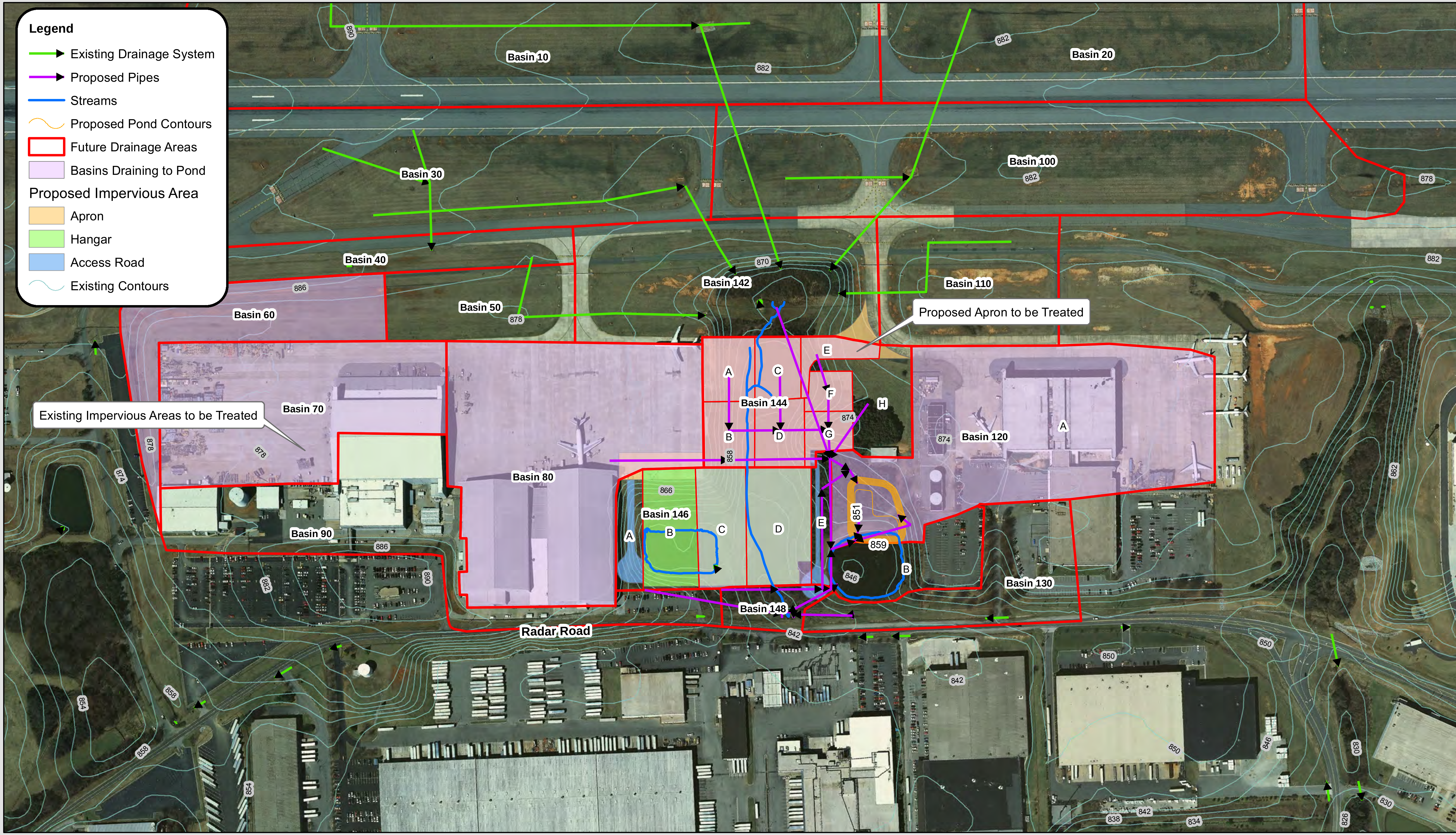
500 250 0 500 Feet

1 inch = 500 feet



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Contributing Drainage Areas and Impervious Cover to SCM - Appendix C

Piedmont-Triad International Airport

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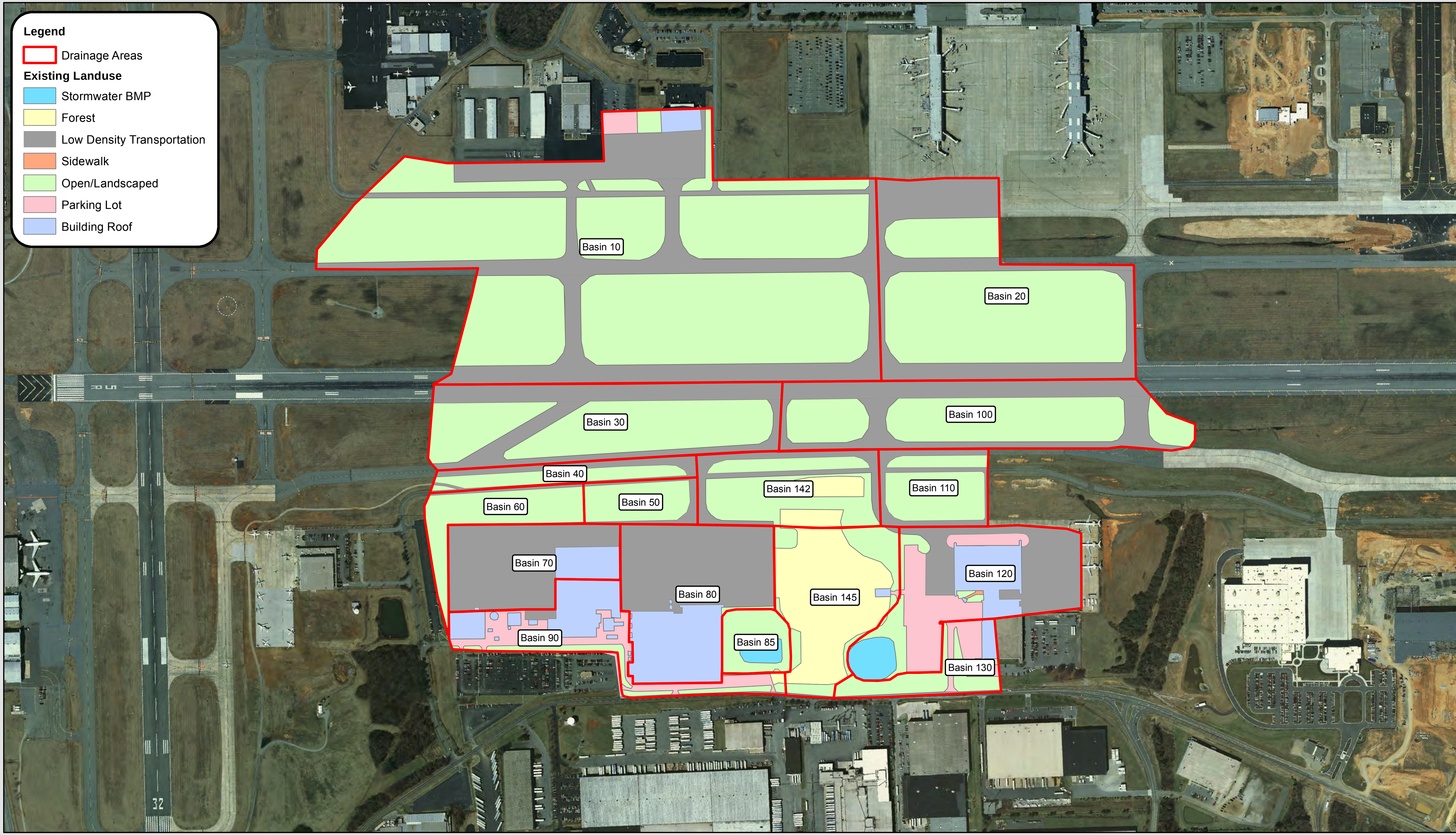

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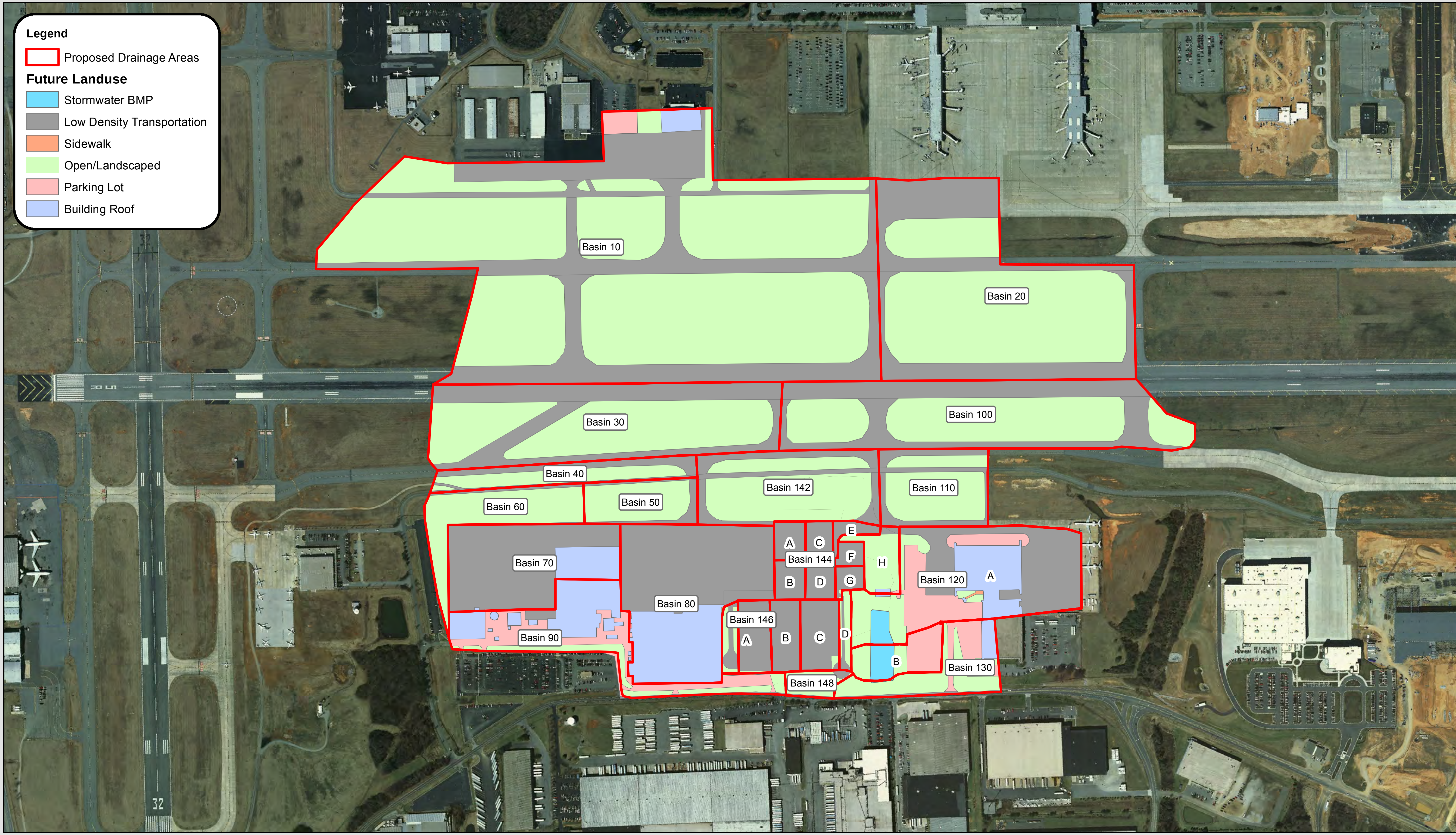
Existing Landuse Map - Appendix D

Piedmont-Triad International Airport

HAECO Site Development



WK DICKSON
community infrastructure consultants

Proposed Landuse Map - Appendix D

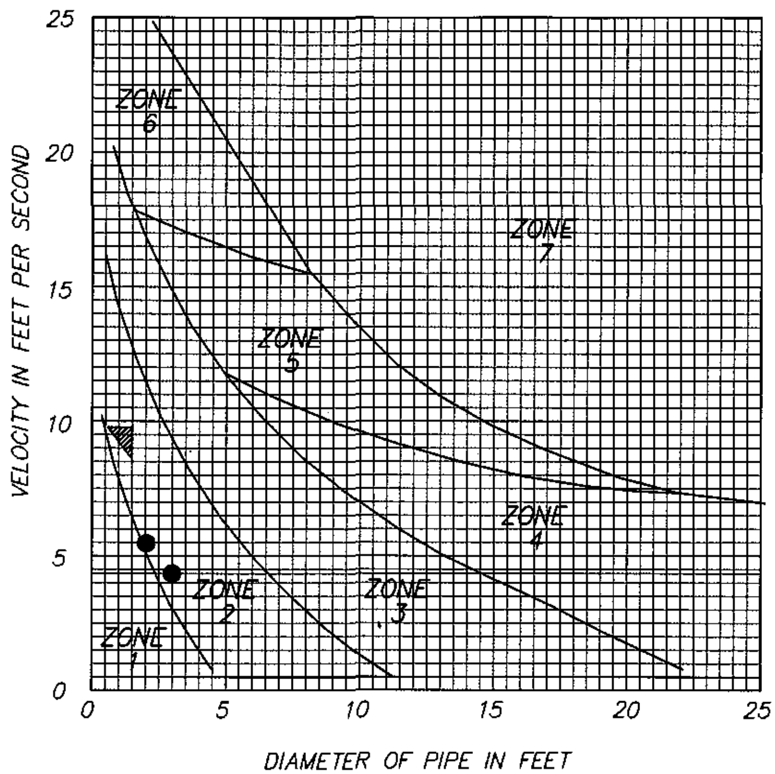
Piedmont-Triad International Airport

HAECO Site Development

500 250 0 500 Feet

1 inch = 500 feet

APPENDIX F



ZONE	APRON MATERIAL	CLASS OF STONE	SIZE OF STONE	LENGTH OF APRON *	MINIMUM THICKNESS OF STONE
1	STONE	FINE	3"	4 X D	9"
2	STONE	LIGHT	6"	6 X D	12"
3	STONE	MEDIUM	13"	8 X D	18"
4	STONE	HEAVY	23"	8 X D	30"
5	STONE	HEAVY	23"	10 X D	30"
6	STONE	HEAVY	23"	12 X D	30"
7	REQUIRES LARGER STONE OR ANOTHER TYPE OF DEVICE. DESIGN IS BEYOND THE SCOPE OF THIS PROCEDURE.				
WIDTH = DIAMETER + 0.4 (LENGTH) : 6' MINIMUM					

* LENGTH TO PREVENT SCOUR HOLE, MIN LENGTH 10'

NAME	WEIGHT (LBS.)	SIZE	SPECIFICATIONS
RIP-RAP			
CLASS 1	5 - 200		30% SHALL WEIGH AT LEAST 100 LBS EACH. NO MORE THAN 10% SHALL WEIGH LESS THAN 15 LBS. EACH.
CLASS 2	25 - 250		60% SHALL WEIGH AT LEAST 100 LBS EACH. NO MORE THAN 5% SHALL WEIGH LESS THAN 50 LBS. EACH.
EROSION CONTROL STONE			
CLASS A		2" - 6"	10% TOP & BOTTOM SIZES. NO GRADATION SPECIFIED.
CLASS B	15 - 300		NO GRADATION SPECIFIED.

STRUCTURE OR LINE	LOCATION	Q-FLOW (CFS)	DIAMETER OF PIPE (IN.)	OUTLET VELOCITY (FPS.)	DEPTH OF FLOW (FT.)	NYDOT ZONE	APRON LENGTH (FT.)	APRON WIDTH 3Do (FT.)	APRON WIDTH W1 (FT.)	APRON THICKNESS (IN.)	RIP-RAP CLASS	REMARKS
PAD#1	BIORETENTION POND	30	36	4.3	1.3	2	18	9	10	24"	TYPE 1	RIP-RAP
PAD#2	BIORETENTION POND	26	24	5.4	1.4	2	15	6	8	24"	TYPE 1	RIP-RAP

SOURCE: 'BANK & CHANNEL LINING PROCEDURES',
NEW YORK DEPARTMENT OF TRANSPORTATION,
DIVISION OF DESIGN AND CONSTRUCTION, 1971.

Water Quality Volume and Stage Storage at Proposed Central High Flow Bioretention Pond

Project: HAECO Facility Improvement @ PTIA, Greensboro, NC
Prepared by: DJK
Checked by:
Date: November 3, 2015

WK DICKSON
Engineers • Planners • Surveyors
Landscape Architects

Summary of Impervious Areas		
Description	Impervious Area (ac)	Total Drainage Area (ac)
Basin 60	0.42	5.19
Basin 70	9.22	9.22
Basin 80	14.09	14.09
Basin 120	11.92	13.95
Proposed Apron	5.11	6.59
Proposed Hangar	5.06	5.95
Proposed Access Rd	0.32	0.66
Hangar Area Not Draining to Pond	-1.71	-1.71
Proposed Sidewalk	0.09	0.09
Total	44.52	54.03

R_v, Runoff coefficient
The R_v is a measure of the site response to rainfall events, and in theory is calculated as:

$R_v = r/p$, where r and p are the volume of storm runoff and storm rainfall, respectively, expressed as inches.

The R_v for the site depends on the nature of the soils, topography, and cover. However, the primary influence on the R_v in urban areas is the amount of imperviousness of the site. Impervious area is defined as those surfaces in the landscape that cannot infiltrate rainfall consisting of building rooftops, pavement, sidewalks, driveways, etc. In the equation $R_v = 0.05 + 0.009(I)$, "I" represents the percentage of impervious cover expressed as a whole number. A site that is 75% impervious would use I = 75 for the purposes of calculating R_v.

Calculate the runoff coefficient:

Rv=0.05+0.009(Ia)

Rv = runoff coefficient = storm runoff (inches) / storm rainfall (inches)

Ia = percent impervious = impervious portion of the drainage area (ac.)/drainage area (ac.)

I_a 82.40

Rv= 0.79 (in./in.)

Calculate the required volume to be detained for the first 1" of runoff:

Volume = (Design rainfall)(Rv)(Drainage Area)

Volume = 1" rainfall * Rv * 1/12 (feet/inches) * Drainage Area

Volume = 3.6 acre-feet

Volume = 155,260 ft^3

Stage Storage Relationship

$$V_{12} = \frac{h}{3}\left(A_1 + A_2 + \sqrt{A_1 \bullet A_2}\right)$$

Stage-Storage from Contours - Proposed Detention Facility - High Flow Bioretention Pond

NODE INVERT	CONTOUR	SWMM DEPTH	CONTOUR AREA		INCREMENTAL VOLUME			S ACCUMULATIVE VOLUME			S TOTAL VOLUME
(FT)	(FT)	(FT)	(AC)	(SF)	(GAL)	(CF)	(AC*FT)	(GAL)	(CF)	(AC*FT)	(%)
848.50	848.50	0.00	0.00	1							
	849.00	0.50	0.00	2	6	1	0.000	6	1	0.000	0%
	850.00	1.50	0.00	3	19	2	0.000	24	3	0.000	0%
Pond Bottom	851.00	2.50	0.33	14,563	36842	4925	0.113	36,866	4,928	0.113	2%
	852.00	3.50	0.56	24,328	143909	19238	0.442	180,775	24,166	0.555	8%
	853.00	4.50	0.60	26,138	188715	25228	0.579	369,490	49,394	1.134	17%
	854.00	5.50	0.64	28,005	202469	27066	0.621	571,959	76,460	1.755	26%
	855.00	6.50	0.69	29,929	216648	28962	0.665	788,607	105,421	2.420	36%
	856.00	7.50	0.73	31,910	231254	30914	0.710	1,019,862	136,336	3.130	47%
	856.60	8.35	0.76	33,133	145958	19512	0.448	1,165,819	155,847	3.578	54%
	857.00	8.50	0.78	33,948	246287	32924	0.756	1,266,148	169,259	3.886	58%
	858.00	9.50	0.83	36,043	362123	48409	1.111	1,628,271	217,668	4.997	75%
	859.00	10.50	0.88	38,196	539362	72102	1.655	2,167,634	289,770	6.652	100%

Incremental volume determined using "conic" method as described in USACE HEC-1 manual

(green)	Pond bottom
(blue)	Elevation that exceeds the water quality volume (assuming static elevation with no infiltration)

Riser Structure Flotation Calculation

Project: HAECO Site Development
Prepared by: DJK
Dated: 11-3-15

Invert Out Elev.	848.00	Bottom of pond with regards to soil (invert of underdrain system is 848.5
Primary Weir Elev.	853.30	
Overflow Weir Elev.	853.30	
Secondary Weir Hght (ft)	1.50	99.20 Weir capacity
Secondary Weir Width (ft)	4.67	calc
Primary Weir Hght (ft)	1.50	calc
Primary Weir Width (ft)	4.67	calc
Top of Box Elev.	855.30	calc
Inside Lgth (ft) (perpendicular to flow)	4.67	
Inside Width (ft)	4.67	
Outside Lgth (ft) (perpendicular to flow)	6.00	calc
Outside Width (ft)	6.00	calc
Primary weir hgth (ft) (CALCULATED)	5.30	calc
Overflow weir hgth (ft) (CALCULATED)	5.30	calc
Wall thickness (ft)	0.67	
Top thickness (ft)	0.50	
Base thickness (ft)	**1.25**	**Design Input (Target factor of safety of 1.2)**
Orifice diameter (in)	0.00	
Orifice area (sq-ft)	0.00	calc
Outlet pipe dia (in)	42.00	
Outlet pipe area (sq ft)	9.62	calc
Concrete weight (lbs/cu ft)	146.00	
Water weight (lbs/cu ft)	62.40	
	Probable	
Str volume (cu-yd)	**4.89**	
Str weight (lbs)	**19,267**	
Buoyant force (lbs)	**15,837**	
Resultant weight (lbs)	**3,430**	
Factor of Safety	**1.22**	
Bearing Weight (lbs/sq ft)	**535.19**	

QC Check on Calcs		
Check on Volume		
Inside width of box	5.67	
Outside width of box	7.00	
Area of inside box	32	
Area of outside box	49	
Height of box (below top)	4.8	
Net Volume of Walls	81.066667	cu ft
Top Area of structure	49	
Thickness of top	0.50	
Volume of top	24.5	cu ft
Volume of base	61.25	cu ft
Total Volume of Concrete	166.81667	
Weight of Concrete	24,355	
Volume of displaced water	320.95	cu ft
Unit weight of water	62.4	
Force of displaced water	20,027	
Factor of Safety	1.22	

Conservative Assumptions:
Bouyant force measured at top of structure lid
100-year flood depth is 8.6 feet in depth (calculation went to elevation 9.5 feet)
Weight of soil on outfall pipe not accounted for in calculation

Detention Time and Design of High Flow Rate Media

Project: HAECO Facility Improvement @ PTIA, Greensboro, NC
Prepared by: DJK
Checked by:
Date: October 23, 2015

Description	Impervious Area (ac)	Total Drainage Area (ac)
Basin 60	0.4	5.2
Basin 70	9.2	9.2
Basin 80	14.1	14.1
Basin 120	11.9	14.0
Proposed Apron	5.1	6.6
Proposed Hangar	5.1	6.0
Proposed Access Rd	0.3	0.7
Total	44.5	54.0

Calculate the runoff coefficient:

$R_v=0.05+0.009(I_a)$

R_v = runoff coefficient = storm runoff (inches) / storm rainfall (inches)

I_a = percent impervious = impervious portion of the drainage area (ac.)/drainage area (ac.)

I_a 82.40

R_v= 0.79 (in./in.)

Calculate the runoff volume for the water quality event (first 1" of runoff):

Volume = (Design rainfall)(R_v)(Drainage Area)

Volume = 1" rainfall * R_v * 1/12 (feet/inches) * Drainage Area

Volume = 3.6 acre-feet

Volume = 155,260 ft^3

Infiltration Zone and Assumed Infiltration Rates for Pond				
Zone	Area (sq ft)	Assumed Infiltration Rate (inch/hr)	Assumed Infiltration Rate (ft/hr)	Assumed Infiltration Rate (ft/sec)
1 (moderately draining soils)	12,403	2	0.2	0.000046
2 (well draining sand)	14,563	10	0.8	0.000231

Calculate Peak Flows and Drawdown Time for WQ Event					
Zone 1 Peak Flow (cfs)	Zone 2 Peak Flow (cfs)	Total Flow (cfs)	Time to Drain Pond (sec)	Time to Drain Pond (min)	Time to Drain Pond (hours)
0.6	3.4	3.9	39,354	656	10.9

Shear Stress Analysis of Rip-Rap Ditches Inside SCM Pond
Project: HAECO Site Development Project, PTIA Airport
Engineer: DJK
Date: 11-3-15

$$Mannings\ Equation,\ Q = (A)\left(\frac{1.49}{n} R_h^{0.66} S^{0.5}\right)$$

36 Inch RCP on Western Side of Pond															
Storm Event	Design Flow (cfs)	Chan Bot Width	Side Slope	Side Slope Length	Design Depth	Chan Area	Wetted Perim., Pw	Hydraulic Radius	Mann. "n"	Channel Slope	Q Allow.	Calc. Depth	Calc. Velocity	Shear Stress	Temp. Liner	Perm. Liner
10-Year	30	3	3	6.3	2	18	16	1.2	0.040	0.002	32	1.9	1.8	0.2	Straw w/ net	Class A
1-Year	9	3	5	5.1	1	8	13	0.6	0.040	0.002	9	1.0	1.2	0.1	Straw w/ net	Class A

24 Inch RCP on Eastern Side of Pond																
Storm Event	Design Flow (cfs)	Chan Bot Width	Side Slope	Side Slope Length	Design Depth	Chan Area	Wetted Perim., Pw	Hydraulic Radius	Mann. "n"	Channel Slope	Q Allow.	Calc. Depth	Calc. Velocity	Shear Stress	Temp. Liner	Perm. Liner
10-Year	26	3	3	6.3	2	18	16	1.2	0.040	0.002	33	1.9	1.8	0.2	Straw w/ net	Class A
1-Year	9	3	5	5.1	1	8	13	0.6	0.040	0.002	9	1.0	1.2	0.1	Straw w/ net	Class A

$$Shear\ Stress,\ T = yds$$

T = shear stress in lb/sq. ft.
y = unit weight of water, 62.4 lb/cu. ft.
d = flow depth in ft.
s = channel slope in ft./ft.

Temporary Liners	
Material	Allow Shear Stress (lb/sqft)
Tacked Mulch	0.35
Jute Net	0.45
Straw w/ Net	1.45
Sythetic Mat	2.00
Class A	1.25
Class B	2.00
Class I	3.40
Class II	4.50

Max. Permissible Velocities for Unprotected Soils in Ex. Channels	
Material	Max. Permissible Velocity (ft/s)
Fine Sand (noncollidal)	2.5
Sand Loam (noncollidal)	2.5
Silt Loam (noncollidal)	3.0
Ordinary Firm Loam	3.5
Fine Gravel	5.0
Stiff Clay (very collidal)	5.0
Graded, Silt to Cobbles	5.0

Max. Allow. Design V for Vegetative Channels			
Channel Slope	Soil	Grass Lining	Permissible V (ft/s)
0-5%	Sands/Sils	Bermuda	5.0
		Tall Fescue	4.5
		KY Bluegrass	4.5
		Grass-legume mix	3.5
	Clay Mixes	Bermuda	6.0
		Tall Fescue	5.5
		KY Bluegrass	5.5
		Grass-legume mix	4.5
5-10%	Sands/Sils	Bermuda	4.5
		Tall Fescue	4.0
		KY Bluegrass	4.0
		Grass-legume mix	3.0
	Clay Mixes	Bermuda	5.5
		Tall Fescue	5.0
		KY Bluegrass	5.0
		Grass-legume mix	3.5

Notes:
Side slope = horiz./vert.

(Shaded cells) Depth and Velocity calculated using AutoCAD's Hydroflow Express

Permit Number:______________________
(to be provided by DEMLR)
Drainage Area Number: _____________

High Flow Rate Bioretention Pond
Operation and Maintenance Agreement

I will keep a maintenance record on this BMP. This maintenance record will be kept in a log in a known set location. Any deficient BMP elements noted in the inspection will be corrected, repaired or replaced immediately. These deficiencies can affect the integrity of structures, safety of the public, and the removal efficiency of the BMP.

Important maintenance procedures:

- The drainage area of the high flow rate bioretention pond will be carefully managed to reduce the sediment load to the sand filter.
- Once a year, sand media will be skimmed.
- The sand filter media will be replaced whenever it fails to function properly after maintenance.

The high flow rate bioretention pond will be inspected **once a quarter and within 24 hours after every storm event greater than 1.0 inches.** Records of operation and maintenance will be kept in a known set location and will be available upon request.

Inspection activities shall be performed as follows. Any problems that are found shall be repaired immediately.

BMP element:	Potential problem:	How I will remediate the problem:
The entire BMP	Trash/debris is present.	Remove the trash/debris.
The grass filter strip or other pretreatment area	Areas of bare soil and/or erosive gullies have formed.	Regrade the soil if necessary to remove the gully, and then plant a ground cover and water until it is established. Provide lime and a one-time fertilizer application.
	Sediment has accumulated to a depth of greater than six inches.	Search for the source of the sediment and remedy the problem if possible. Remove the sediment and dispose of it in a location where it will not cause impacts to streams or the BMP.
The flow diversion structure (if applicable)	The structure is clogged.	Unclog the conveyance and dispose of any sediment off-site.
	The structure is damaged.	Make any necessary repairs or replace if damage is too large for repair.

BMP element:	Potential problem:	How I will remediate the problem:
The bioretention cell: soils and mulch	Mulch is breaking down or has floated away.	Spot mulch if there are only random void areas. Replace whole mulch layer if necessary. Remove the remaining mulch and replace with triple shredded hard wood mulch at a maximum depth of three inches.
	Soils and/or mulch are clogged with sediment. Water is ponding on the surface for more than 24 hours after a storm.	Check to see if the collection system is clogged and flush if necessary. If water still ponds, remove the top few inches of the filter bed material and replace. If water still ponds, then consult an expert.
Outlet device	Clogging has occurred.	Clean out the outlet device and dispose sediment in a location that will not impact a stream or the BMP.
	The outlet device is damaged.	Repair the outlet device.
The observation well(s)	The water table is within one foot of the bottom of the system for a period of three consecutive months.	Contact DEMLR Stormwater Permitting staff immediately at 919-707-9220.
	The outflow pipe is clogged.	Provide additional erosion protection such as reinforced turf matting or riprap if needed to prevent future erosion problems.
	The outflow pipe is damaged.	Repair or replace the pipe.
The emergency overflow berm	Erosion or other signs of damage have occurred at the outlet.	The emergency overflow berm will be repaired or replaced if beyond repair.
The receiving water	Erosion or other signs of damage have occurred at the outlet.	Contact the N.C. Division of Water Resources 401 Certification Program staff at 919-707-8789.

I acknowledge and agree by my signature below that I am responsible for the performance of the maintenance procedures listed above. I agree to notify DEMLR of any problems with the system or prior to any changes to the system or responsible party.

Project name: HAECO Site Development Project ______________________

BMP drainage area number:______________________

Print name:______________________

Title:______________________

Address:______________________

Phone:______________________

Signature:______________________

Date:______________________

Note: The legally responsible party should not be a homeowners association unless more than 50% of the lots have been sold and a resident of the subdivision has been named the president.

I, ______________________, a Notary Public for the State of ______________________, County of ______________________, do hereby certify that ______________________ personally appeared before me this _____ day of __________________, ________, and acknowledge the due execution of the forgoing high flow rate bioretention pond maintenance requirements. Witness my hand and official seal,

SEAL

My commission expires______________________

DIVISION OF MITIGATION SERVICES (DMS)

IN-LIEU FEE REQUEST FORM Revised 4/23/2015

Complete requested information, sign and date, email to kelly.williams@ncdenr.gov. Attachments are acceptable for clarification purposes (location map, address *or* lat long is required). Information submitted is subject to NC Public Records Law and may be requested by third parties. Review meetings are held on Tuesday afternoons.

CONTACT INFORMATION	APPLICANT'S AGENT	APPLICANT
1. Business/Company Name	Michael Baker Engineering, Inc.	Piedmont Triad Airport Authority
2. Contact Person	Richard Darling	Alex Rosser, PE
3. Street Address or P O Box	8000 Regency Pkwy., Suite 600	1000-A Ted Johnson Pkwy.
4. City, State, Zip	Cary, NC 27518	Greensboro, NC 27409
5. Telephone Number	919.481.5740	336.665.5600
6. E-Mail Address	rdarling@mbakerintl.com	rossera@gsoair.org

PROJECT INFORMATION

7. Project Name	HAECO Facility Improvements		
8. Project Location (nearest town, city)	623 Radar Road; Greensboro, NC 27410		
9. Lat-Long Coordinates or attach a map	36 05 49 N, 79 55 53 W		
10. County	Guilford		
11. River Basin & Cataloging Unit (8-digit) (See Note 1)	03030002		
12. Project Type **indicate owner type and write in project type (e.g. school, church, retail, residential, apartments, road, utilities, military, etc.)**	Owner Type: ⦿ Government ○ Private Project Type: aviation development		
13. Riparian Wetland Impact (ac.) (e.g., 0.13)	0.81		
14. Non-Riparian Wetland Impact (ac.)	0		
15. Coastal Marsh Impact (ac.)	0		
16. Stream Impact (ft.) (e.g. 1,234)	**Warm**	**Cool**	**Cold**
	1,601		
17. Riparian Buffer Impact (sq. ft.) Include subwatershed if Jordan or Falls Lake:	Zone 1:	Zone 2:	
18. Regulatory Agency Staff Contacts USACE: David Bailey	NCDWR: Other: Sue Homewood		

Check (√) below if this request is for a:

revision to a current acceptance

renewal of an expired acceptance

extension of unexpired acceptance

By signing below, the applicant is confirming they have read and understand the DMS refund policy posted at nceep.net and attached to this form.

Signature of Applicant or Authorized Agent: [signature]

Date: 11/13/2015

Note 1: For help in determining the Cataloging Unit, visit:www.nceep.net or contact DMS

Direct questions to Kelly Williams at 919-707-8915 or kelly.williams@ncdenr.gov or to the front desk at 919-707-8976

Print Form

DMS ILF Mitigation Request Statement of Compliance with §143-214.11 & 143-214.20

(link to G.S. 143-214.11)

Prior to accessing the Division of Mitigation Services (DMS), all applicants must demonstrate compliance with **G.S. § 143-214.11** and 143-214.20. All requests MUST include this form signed and dated by the permit applicant or an authorized agent. Please refer to DENR's Implementation Policy for more details.

Compliance Statement:
I have read and understand G.S. § 143-214.11 and 214.20 and have, to the best of my knowledge, complied with the requirements. I understand that participation in the DMS is voluntary and subject to approval by permitting agencies.

Please check all that apply:

- ☐ **Applicant is a Federal or State Government Entity or a unit of local government meeting the requirements set forth in G.S. 143-214.11 and is not required to purchase credits from a mitigation bank.**
- ☐ **There are no listed mitigation banks with the credit type I need located in the hydrologic unit where this impact will take place (link to DWR list)**
- ☒ **Mitigation bank(s) in the hydrologic unit where the impacts will occur have been contacted and credits are not currently available.**
- ☐ **The DWR or the Corps of Engineers did not approve of the use of a mitigation bank for the required compensatory mitigation for this project.**
- ☐ **This is a renewal request and the permit application is under review. Bank credits were not available at the time the application was submitted. Enter date permit application was submitted for review: ____________**

Note: It is the applicant's responsibility to document any inquiries made to private mitigation banks regarding credit availability.

JAR (initials) **I have read and understand the DMS refund policies (attached)**
initial here

[signature]
Signature of Applicant or Agent

J. Alex Rosser, PE; Deputy Executive Director
Printed Name

Date

HAECO Facility Improvements
Project Name

Piedmont Triad International Airport
Location

Refund Policy for Fees Paid to DMS In-Lieu Fee Programs (9/21/2009)

Purpose: The purpose of this policy is to make clear the circumstances and process under which a permittee can obtain a refund while simultaneously balancing customer service and responsible business practices. This policy applies to all refund requests made on or after the publication date of this policy.

Policy Statement: The policy of DMS is to allow for refunds under certain conditions.

1. All refund requests must be made in writing to the DMS In-Lieu Fee Program Coordinator at kelly.williams@ncdenr.gov.

2. All refund requests are subject to fund availability. DMS does not guarantee fund availability for any request.

3. The request must either come from the entity that made the payment or from an authorized agent. Third parties requesting refunds must provide written authorization from the entity that made the payment specifying the name and address of the authorized refund recipient.

4. Refund requests related to unintended overpayments, typographical errors or incorrect invoices should be brought the attention of the In-Lieu Fee Program Coordinator as soon as possible. Such requests are typically approved without delay.

5. Payments made under the incremental payment procedure are not eligible for refunds.

6. Refund requests made within nine months of payment to DMS will only be considered for requests associated with projects that have been terminated or modified where the permittee's mitigation requirements have been reduced. Such requests must be accompanied by written verification from the permitting agency that the project has been cancelled, the permits have been rescinded or have been modified, or the mitigation requirements have been reduced.

7. Refund requests made more than nine months from the payment date will only be considered for permits that were terminated or modified to not require any mitigation. Such requests must be accompanied by written verification from the permitting agency that the project has been cancelled, the permits have been rescinded and/or mitigation is no longer required.

8. Refund requests not meeting the criteria specified above are not eligible for a refund.

9. Refund requests that meet the criteria above will be elevated to DMS Senior Management for review. The following considerations apply to all refund requests:

   a. availability of funds after consideration of all existing project and regulatory obligations
   b. the date the payment was made
   c. the likelihood DMS can use the mitigation procured using the payment to meet other mitigation requirements

10. Once a refund has been approved, the refund recipient must provide a completed W-9 form to the DMS In-Lieu fee Program Coordinator within two weeks in order to process the refund though the State Controller's Office.

11. All decisions shall be final.

FedEx Shipment Receipt

Address Information

Ship to:
David Bailey
US Army Corps of Engineers
3331 Heritage Trade Dr.
Suite 105
WAKE FOREST, NC
27587
US
919-554-4884

Ship from:
Phyllis Best
Michael Baker Corp.
8000 Regency Parkway
Suite 600
Cary, NC
27518
US
919-463-5488

Shipment Information:

Tracking no.: 775064672806
Ship date: 11/25/2015
Estimated shipping charges: 8.60

Package Information

Pricing option: FedEx Standard Rate
Service type: Priority Overnight
Package type: FedEx Pak
Number of packages: 1
Total weight: 1 LBS
Declared Value: 0.00 USD
Special Services:
Pickup/Drop-off: Use an already scheduled pickup at my location

Billing Information:

Bill transportation to: MICHAEL BAKER JR INC-994
Project Number: 148092
Task Number: 1
Invoice no. :
Department no:

Thank you for shipping online with FedEx ShipManager at fedex.com.

Please Note

FedEx Shipment Receipt

Address Information

Ship to:
Karen Higgins
NCDENR-DWQ 401 Permitting
512 Salisbury St.
Archdale Blding, 9th Floor
RALEIGH, NC
27604
US
919-807-6360

Ship from:
Phyllis Best
Michael Baker Corp.

8000 Regency Parkway
Suite 600
Cary, NC
27518
US
919-463-5488

Shipment Information:
Tracking no.: 775064584645
Ship date: 11/25/2015
Estimated shipping charges: 22.66

Package Information
Pricing option: FedEx Standard Rate
Service type: Priority Overnight
Package type: Your Packaging
Number of packages: 2
Total weight: 10 LBS
Declared Value: 0.00 USD
Special Services:
Pickup/Drop-off: Use an already scheduled pickup at my location

Billing Information:
Bill transportation to: MICHAEL BAKER JR INC-994
Project Number: 148092
Task Number: 1
Invoice no. :
Department no: