HARRIS

Harris Civil Engineers, LLC

MECATOS BAKERY – EDGEWOOD

5645 HANSEL AVE, EDGEWOOD

STORMWATER SYSTEM
DESIGN REPORT

City of Edgewood

PREPARED BY
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This item has been digitally signed and sealed by **David Taylor, P.E.** on the date adjacent o the seal. Printed copies of this document are not considered signed and sealed and the signature must be verified on any electronic copies.

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Date: 2021.11.05 11:49:49 -04'00'

October 25, 2021

HCE Project No. 7391000

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INTRODUCTION

This report describes the process and presents the results of the stormwater run-off calculations that were performed for the construction activities that will shortly be undertaken for a new Mecatos Bakery at 5645 Hansel Ave in Edgewood, Florida.

The proposed 0.92-acre site is currently developed. This project involves the demolition of the existing infrastructure and proposes a new building expansion and facilities along with site improvements. For further details of the proposed construction activities, see the construction documents dated October 25th, 2021.

METHODOLOGY

The design of the post-construction system was based on criteria set forth in the Saint John's River Water Management District (SJRWMD) Permit Information Manual, latest edition.

Per the SJRWMD water quality criteria, the site, East and West basins, must hold back the greater of 1" of runoff over the entire basin or 2.5" of runoff over the impervious area.

EAST BASIN

The East Basin was designed such that the pre-development peak run-off was less than the post-development peak run-off from the 25-year/24-hour rainfall event and retain the volumetric difference of run-off from a 100-year/24-hour rainfall event. The Rational Method was used to determine the pre-development versus the post-development peak flow rates to prove that they decreased. This was achieved by increasing the water attenuation volume from the pre-construction condition to the post-construction condition, while also decreasing the overall impervious area for the basin. In the case of this project, the rainfall amount is 8.6 inches for a 25-year/24-hour rainfall event and 10.6 inches for a 100-year/24-hour rainfall event.

WEST BASIN

The stormwater treatment facilities must discharge water in the post-development condition at a peak rate equal to or less than that of the pre-development condition for a 25-year/24-hour rainfall event. The West Basin was designed such that the pre-development peak run-off was less than the post-development peak run-off from the 25-year/24-hour rainfall event. This was accomplished using a positive rain outfall and the Rational Method to determine the pre-development versus the post-development peak flow rates. In the case of this project the rainfall amount is 8.6 inches for a 25-year/24-hour rainfall event.

PRE-CONSTRUCTION CONDITIONS

The existing site has two basins, West and East, respectively, that cumulatively covers the 0.92 acres. The pre-construction areas and runoff coefficients (C-values) were calculated and used to determine the peak runoff of the pre-development condition.

The West Basin covers 0.36 acres, and using Rational Method has a peak runoff of 2.53 cfs for a 25-year/24-hour storm. This basin has a positive outfall over land into Hansel Avenue.

The East Basin covers 0.56 acres, and using Rational Method has a peak runoff of 3.07 cfs for a 25-year/24-hour storm. This basin's runoff flows to an existing pond that does not have an outfall.

See Appendix A for the pre/post construction calculations.

POST-CONSTRUCTION CONDITIONS

EAST BASIN

The design of the post-construction system includes a pond plus a series of perforated underdrains below the proposed locations of the permeable pavers. The underdrains are below a gravel envelope, with a minimum depth of 27". The pond and permeable pavers combined provides more water storage volume than the existing standalone pond.

See Appendix B for water quality calculations.

The post-construction areas and runoff coefficients (C-values) were calculated to determine the peak runoff of the post-development site condition for the East Basin.

The East Basin covers 0.56 acres, and using the Rational Method has a peak runoff of 2.98 cfs for a 25-year/24-hour storm.

See Appendix A for the pre/post construction calculations.

<u>WEST BASIN</u>

The design of the post-construction system basin shall decrease the amount of impervious surface and therefore, the peak flow discharge rate is decreased.

The post-construction areas and runoff coefficients (C-values) were calculated to determine the peak runoff of the post-development site condition for the West Basin.

The West basin covers 0.36 acres, and using the Rational Method has a peak runoff of 2.53cfs for a 25-year/24-hour storm.

See Appendix A for the pre/post construction calculations.

WATER QUALITY

The required volume of water that must be treated for the site was according to the criteria of 2.5 inches of runoff over the impervious area for a wet detention system. The required water quality volume is 0.047 ac-ft. The provided water quality volume of the system, from the bottom of the gravel envelope to the top control level of the control structure, was a volume of 0.093 ac-ft. The water quality volume will infiltrate in under the required 72 hours and the 100-year volume will infiltrate within the required 14 days.

See Appendix B for the water quality calculations.

RESULTS

<u>EAST BASIN</u>

The post-development stormwater system has been designed to meet water quality requirements of the SJRWMD. The 0.16 ac-ft of water quality volume provided is greater than the 0.047 ac-ft required.

The pre-construction discharge rate for the East Basin is 3.07 cfs for a 25-year/24-hour storm event. The post-construction discharge rate is 2.98 cfs.

<u>WEST BASIN</u>

The pre-construction discharge rate for the West basin is, 2.53 cfs for a 25-year/24-hour storm event. The post-construction discharge rate is 2.53 cfs.

Therefore, the post-development discharge rates on both the West and East Basins are lower than or equal to the pre-development discharge rates and the water quality requirements are met.

See Appendix C for Geotechnical Report

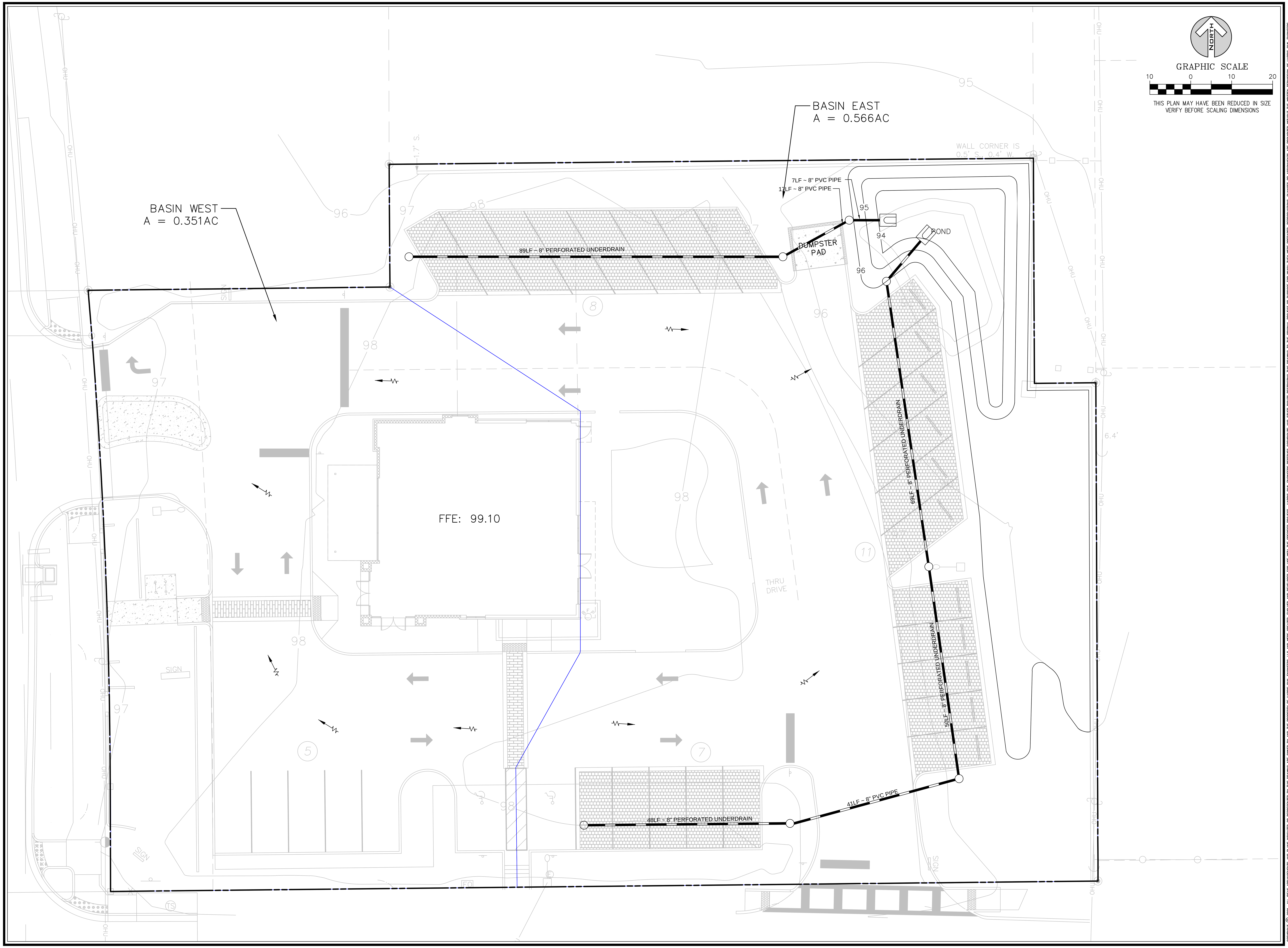
APPENDIX A – PRE/POST CONSTRUCTION CALCULATIONS

Pervious / Impervious Table

Overall			
Ground Cover Description	**Area (sf)**	**Area (ac-ft)**	**% Of Site**
Pre-Development Totals			
Building	1,443	0.033	3.6%
Impervious	21,053	0.483	52.7%
Pervious	17,442	0.400	43.7%
Total	39,938	0.917	100%
Post-Development Totals			
Building	2,659	0.061	6.7%
Pavers	4,464	0.102	11.2%
Impervious	19,031	0.437	47.7%
Pervious	13,784	0.316	34.5%
Total	39,938	0.917	100.0%

Basin 1 (East Side)			
Ground Cover Description	**Area (sf)**	**Area (ac-ft)**	**% Of Basin**
Pre-Development Totals			
Building	785	0.018	3.2%
Impervious	9,646	0.221	39.4%
Pervious	14,041	0.322	57%
Total	24,472	0.562	100%
Post-Development Totals			
Building	0	0.000	0.0%
Pavers	4,464	0.102	18.4%
Impervious	9,789	0.225	40.4%
Pervious	9,990	0.229	41.2%
Total	24,243	0.557	100.0%

Basin 2 (West Side)			
Ground Cover Description	**Area (sf)**	**Area (ac-ft)**	**% Of Basin**
Pre-Development Totals			
Building	658	0.015	4.3%
Impervious	11,407	0.262	73.8%
Pervious	3,401	0.078	22.0%
Total	15,466	0.340	100%
Post-Development Totals			
Building	2,659	0.061	16.9%
Pavers	0	0.000	0.0%
Impervious	9,242	0.212	58.9%
Pervious	3,793	0.087	24.2%
Total	15,694	0.360	100.0%



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Revisions:

No.	Date	Description
1.		
2.		
3.		
4.		
5.		
6.		
7.		
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9.		

GRADING AND DRAINAGE PLAN

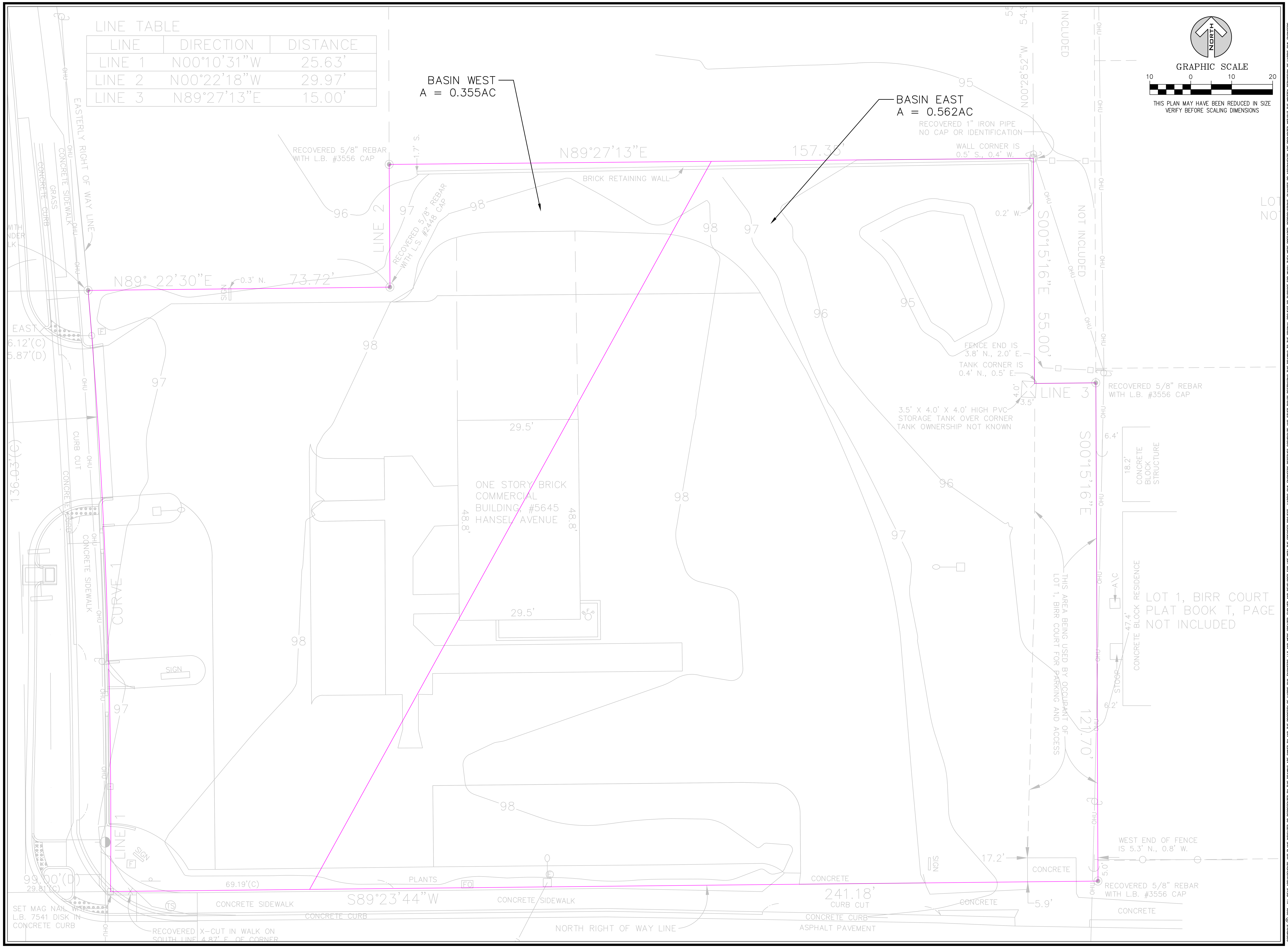
Scale: SEE SHEET
Date: 2021-09-03
Project No: 7391000

Design: DWT
Drawn: AJT
Checked: DWT

David W. Taylor, PE
FL PE # 60928

Release:

POST



LINE TABLE

LINE	DIRECTION	DISTANCE
LINE 1	N00°10'31"W	25.63'
LINE 2	N00°22'18"W	29.97'
LINE 3	N89°27'13"E	15.00'

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David W. Taylor, PE
FL PE # 60928

Release:

PRE

Rational Method for Pre vs Post Peak Discharge
East Basin

Ground Cover Description	C-Value	Pre-Development	C*A	Post-Development	C*A
Building	0.95	0.0	0.0	0.0	0.0
Pavers	0.40	0.0	0.0	0.1	0.0
Impervious	0.95	0.2	0.2	0.2	0.2
Pervious	0.40	0.3	0.1	0.2	0.1
Total Area	(acres)	0.6		0.6	
Composite C-Value			0.63		0.62
1" Rainfall	(in/hr)		1		1
Q=C*I*A	**(cfs)**		**0.36**		**0.35**
Intensity (25-yr/24-hr rainfall)	(in/hr)		8.6		8.6
Q=C*I*A	**(cfs)**		**3.07**		**2.98**

Rational Method for Pre vs Post Peak Discharge
WEST BASIN

Ground Cover Description	C-Value	Pre-Development	C*A	Post-Development	C*A
Building	0.95	0.02	0.01	0.06	0.06
Pavers	0.44	0.00	0.00	0.00	0.00
Impervious	0.95	0.26	0.25	0.21	0.20
Pervious	0.40	0.08	0.03	0.09	0.03
Total Area	(acres)	0.36		0.36	
Composite C-Value			0.83		0.82
1" Rainfall	(in/hr)		1		1
Q=C*I*A	**(cfs)**		**0.29**		**0.29**
Intensity (25-yr/24-hr rainfall)	(in/hr)		8.6		8.6
Q=C*I*A	**(cfs)**		**2.53**		**2.53**

APPENDIX B – WATER QUALITY CALCULATIONS

Water Quality Treatment Required

					Greater of These - Orange County		Greater of These - SJRWMD		Plus	Total
Drainage Basin I.D.	**Drainage Area (SF)**	**Impervious Area (SF)**	**Pervious Area (SF)**	**Percent Impervious (%)**	**0.5 inch of Runoff From Entire Site (ac-ft)**	**1.0 inch of Impervious Area (ac-ft)**	**0.5 inch of Runoff From Entire Site* (ac-ft)**	**1.25 inches of Impervious Area* (ac-ft)**	**Additional 0.5" of Runoff From Entire Site* (ac-ft)**	**(ac-ft)**
Post Basin 1 (East Side)	24472	9789	9990	40.0%	**0.023**	0.019	0.023	**0.023**	**0.023**	0.047
Post Basin 2 (West Side)	15466	11901	3793	76.9%	0.015	**0.023**	0.015	**0.028**	**0.015**	0.043
TOTAL					**0.038**	**0.041**				**0.090**

Required Water Quality Volume* = **0.047** **ac-ft**
2,039 **cf**

*Water quality volume only for east basin

1. West basin post impervious less then pre impervious therefor water quality treatment not needed
2. Calculation based upon the design examples provided in the SJRWMD Environmental Resource Permit Information Manual

MECATOS BAKERY
STORM WATER VOLUME AND AREA CALCS.
10-13-2021

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POND				
Stage (feet)	Area (feet)	Volume (CF)	Cumul. Volume (CF)	Cumul. Volume (ac-ft)
94.00	674.6	0.0	0.0	0.0
94.50	917.4	398.0	398.0	0.01
95.00	1198.3	528.9	926.9	0.02
95.50	1477.8	669.0	1596.0	0.04
96.00	3734.0	1303.0	2898.9	0.07

SOUTH PAVERS				
Stage (feet)	Area (feet)	Volume (CF)	Cumul. Volume (CF)*	Cumul. Volume (ac-ft)
94.00	836.0	0.0	0.0	0.0
94.50	836.0	418.0	167.2	0.01
95.00	836.0	418.0	334.4	0.02
95.50	836.0	418.0	501.6	0.03
96.00	836.0	418.0	668.8	0.04

SOUTH EAST PAVERS				
Stage (feet)	Area (feet)	Volume (CF)	Cumul. Volume (CF)*	Cumul. Volume (ac-ft)
94.00	810.0	0.0	0.0	0.000
94.50	810.0	405.0	162.0	0.009
95.00	810.0	405.0	324.0	0.019
95.50	810.0	405.0	486.0	0.028
96.00	810.0	405.0	648.0	0.037

MECATOS BAKERY
STORM WATER VOLUME AND AREA CALCS.
10-13-2021

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Harris Civil Engineers, LLC

NORTH EAST PAVERS

Stage (feet)	Area (feet)	Volume (CF)	Cumul. Volume (CF)*	Cumul. Volume (ac-ft)
94.00	1170.0	0.0	0.0	0.000
94.50	1170.0	585.0	234.0	0.005
95.00	1170.0	585.0	468.0	0.011
95.50	1170.0	585.0	702.0	0.016
96.00	1170.0	585.0	936.0	0.021
94.00	202.5	0.0	0.0	0.000
94.50	202.5	101.3	40.5	0.001
95.00	202.5	101.3	81.0	0.002
95.50	202.5	101.3	121.5	0.003
96.00	202.5	101.3	162.0	0.004

NORTH PAVERS

Stage (feet)	Area (feet)	Volume (CF)	Cumul. Volume (CF)*	Cumul. Volume (ac-ft)
94.00	1620.0	0.0	0.0	0.000
94.50	1620.0	810.0	324.0	0.007
95.00	1620.0	810.0	648.0	0.015
95.50	1620.0	810.0	972.0	0.022
96.00	1620.0	810.0	1296.0	0.030
94.00	202.5	0.0	0.0	0.000
94.50	202.5	101.3	40.5	0.001
95.00	202.5	101.3	81.0	0.002
95.50	202.5	101.3	121.5	0.003
96.00	202.5	101.3	162.0	0.004

SUMMARY

Stage (feet)	Area (feet)	Volume (CF)	Cumul. Volume (CF)*	Cumul. Volume (ac-ft)
94.00	5515.6	0.0	0.0	0.000
94.50	5758.4	2818.5	1366.2	0.031
95.00	6039.3	2949.4	2863.3	0.066
95.50	6318.8	3089.5	4500.6	0.103
96.00	8575.0	3723.5	6771.7	0.155

***Volume of paver area based on 40% voids**

APPENDIX C – GEOTECHNICAL REPORT

MIKE TANNOUS ENGINEERING
INCORPORATED

GEOTECHNICAL INVESTIGATION
PROPOSED MECATOS BAKERY
5645 HANSEL AVE, EDGEWOOD, FL.
MTE, INC. PROJECT NO. 21-325

Prepared For:

Mr. Nelson Lerma
Via Email at: nelson_lerma@hotmail.com

October 22, 2021

MIKE TANNOUS ENGINEERING
INCORPORATED
Geotechnical Engineering ● Construction Materials Testing / Quality Control

October 22, 2021

Mr. Nelson Lerma
Via Email at: nelson_lerma@hotmail.com

Subject: ***Geotechnical Study, Proposed Mecatos Bakery, 5645 Hansel Avenue, Edgewood, Florida (MTE, Inc. Project No. 21-325)***

Dear Nelson,

As requested, a representative of our firm performed numerous shallow auger borings at the above referenced site. The purpose of the borings was to evaluate the suitability of the shallow subgrade soils and groundwater table as they relate to the geotechnical engineering aspects of the planned development. The following report summarizes the results of our field and laboratory testing programs and presents our conclusions and recommendations.

SITE AND PROJECT DESCRIPTION

The subject property consists of Orange County Parcel No. 24-23-29-3400-00-014. More specifically, the property is an approximately 0.91± acre tract of land, located at the northeast corner of the intersection of Hoffner Avenue and Hansel Avenue in the City of Edgewood, Florida. The property currently contains an approximately 2,382 square feet, 1-story building (apparently a previous bank structure) which was purportedly built in 1963. The remaining portions of the site are mostly covered with asphaltic pavement. A retention pond occupies the northeast corner of the site, extending along a portion of the east side. A street view of the site is presented on the report cover sheet.

Based on review of a topographic map of the site, the majority of the site lies at elevations varying between approximately +97 to +98 feet. The area of the existing retention pond along the east side of the site lies at elevations of about +95 to +96 feet.

Based on our discussions and review of the conceptual site plan you provided, we understand that the proposed development includes the construction of a one-story addition along the west side of the existing structure and renovating the existing building for the intended use (a bakery store). We assume the existing pavement section will be removed and re-constructed, or at least milled and re-paved. Storm water management from the new development will be managed by constructing pervious pavers along most parking stall locations, along with re-configuring the existing retention pond. The proposed layout of the development, as we understand it, is graphically illustrated on the attached Figure 1.

PURPOSE AND SCOPE OF SERVICES

As mentioned earlier, our firm was retained to evaluate the shallow subsurface soil and groundwater table conditions within the site as they relate to the geotechnical engineering aspects of the proposed development. Towards that objective, the following scope of services was performed:

- Reviewed The United States Department of Agriculture Soil Conservation Soil Survey Report of Orange County.

- Performed numerous borings in proposed building, pavement, and storm water management facilities.

- Recovered soil samples from the borings for visual examination and classification by the Project Engineer and for laboratory testing.

- Measured the depth of the groundwater table at the boring locations and estimated the high wet season groundwater level.

- Prepared this geotechnical report which summarizes the results of our field and laboratory evaluations and presents our conclusions and recommendations relative to the geotechnical engineering aspects of the planned development.

- Lastly, after receiving estimated storm water volumes and stage storage calculations from the Project Civil Engineer, we performed seepage analyses to verify proper storm water runoff recovery time in the proposed retention pond and below pervious pavers.

Please be advised that the scope of work completed for this project included an evaluation of the relatively shallow sub-grade soils that will be most affected by the weight of the proposed building addition. The drilling of deeper borings for the purpose of evaluating the potential for sinkhole activity in the site was beyond our scope of work. Should you require it, we can perform a sinkhole assessment of the site under a separate scope of work.

Specific information regarding the anticipated structural loads of the proposed building addition was not available at the time of this study. For the purpose of our evaluations, we assumed maximum wall loads on the order of 2 to 3 kips per foot. In the event that support columns are proposed, we assumed maximum column loads of 30 kips per column. If it is determined that the structural loads are appreciably higher than the values presented above, please notify our firm so we may verify the conclusions outlined in this report.

REVIEW OF SOIL SURVEY REPORT

The predominant near-surface soils within and around the subject site were mapped by the United States Department of Agriculture and the Soil Conservation Service (USDA/SCS), now known as the National Resource Conservation Service (NRCS) and were, subsequently, published in the Orange County Soil Survey Report. A map illustrating the generalized delineation of the various near-surface soils in and around the site is presented on the following page of this report. The yellow arrow points to the site.

As indicated by the Soil Survey Map, the approximate eastern half of the site is situated in an area mapped as Map Unit #22; Lochloosa Fine Sand. According to the Soil Survey Report, the near surface soils in this soil series are typically sandy in the upper 29± inches, grading into sandy clay loam to a depth of 80 inches. The soil series typically forms on rises on marine terraces and the area is often somewhat poorly drained. The groundwater table is typically fluctuated between 18 and 60 inches below grade.

The approximate western half of the site is situated in an area mapped as Map Unit No. 55; Zolfo-Urban land Complex. This soil series typically forms on flats on marine terraces. The near surface soils are typically sandy in the upper 80 inches and the area is often somewhat poorly drained. The urban designation indicates that the area has been developed. The near-surface soils have been altered from their natural formation and the ground surface is mostly covered with impervious surfaces such as buildings, concrete, pavement, etc. The groundwater table is normally within 24 to 42 inches below grade, but can be deeper depending on the functionality of the drainage improvements made during development of the urban areas.

Aerial map illustrating near-surface soil delineation in and around the site. The map was obtained from the Orange County Soil Survey Report. The yellow arrow points to the site.

FIELD EXPLORATION PROGRAM

Auger Borings with Penetrometer Probes

A total of seven auger borings were performed for this project. Two borings were performed to a depth of 12 feet in the area of the proposed building addition; four borings were performed to a depth of 10 feet in areas proposed for storm water management; and one boring was performed to a depth of 7 feet near the entrance into the site. The borings were accomplished by manually twisting an open-ended bucket auger into the ground in 6-inch intervals, then recovering the soils collected by the auger. Auger borings allow for continuous sampling of the subgrade soils, in 6-inch intervals, for the entire depth of the borehole.

The approximate locations of the borings are graphically illustrated on the attached Figure 1. Note that the boring locations shown on Figure 1 were not surveyed and were determined on site by measurements from existing reference points found on the site. Therefore, the illustrated locations should be considered approximate and may not represent the exact boring locations.

Penetrometer probes were performed to depths of 7 to 12 feet at the auger boring locations in order to evaluate the relative compactness of the shallow subgrade soils. The hand cone penetrometer is a steel shaft with a conical point that is pushed into the ground in one-foot intervals. The resistance to penetration is registered by a dial gauge attached to the top of the shaft. The gauge reading provides a measure of the relative density of the subgrade soils. As a reference, it has been our experience that penetrometer probe readings of 0 to 4 in sandy soils generally indicate relatively very loose soils, readings of 5 to 10 indicate relatively loose soils, readings of 11 to 35 indicate relatively medium dense soils, and readings above 35 indicate relatively dense soil conditions.

Representative soil samples were collected from the borings during the field investigation. These samples were visually classified on-site by our field technician and were subsequently, returned to our office for further visual examination by the Project Engineer. Two undisturbed soil samples were recovered from the site and then returned to our laboratory facility for permeability testing; one sample was recovered from a depth of 3.5 feet at the location of Boring AB-2 (sample recovered from soil stratum #1) and the other sample was recovered from a depth of 3.5 feet at the location of Boring AB-7 (sample recovered from soil stratum #2).

In addition to the soil sampling, the groundwater level was measured in each open borehole at the time of the field investigation.

SUBSOIL AND GROUNDWATER TABLE

Shallow Stratigraphy

The stratigraphy observed at the borehole locations was determined based on our visual classification of the soil samples collected from the auger borings. The description and stratification of the soils were accomplished in general accordance with the Unified Method of Soil Classification. The results of our visual interpretations are presented in the form of soil profiles, shown on Figure 1. A legend of the terms and symbols used to create the soil profiles is also shown on Figure 1.

The results of the borings suggest that the site was previously filled/raised about 2 to 4 feet. The fill material typically consisted of a mixture of gray and brown fine sands to slightly silty fine sands. Below the fill material, relatively thin layers of grayish brown and very pale brown fine sands were encountered to typical depths of about 4± to 7± feet. The underlying subgrade soils then graded into pale brown slightly silty to silty fine sands, followed by reddish brown silty fine sands with cemented nodules. The silty sands extended down to the termination depths of all the boreholes. A more detailed delineation of the soils observed at each boring location can be reviewed in Figure 1.

The results of the penetrometer probes suggest that the subgrade soils are typically in a relatively loose to medium dense condition in the upper 5± feet, becoming predominantly medium dense to the maximum probed depth of 12 feet.

Groundwater Table

The groundwater table was encountered at depths varying between approximately 3½± and 6½± feet below ground surface at the boring locations. Based on review of the topographic map of the site, the measured groundwater levels typically correspond to an average elevation of about +92.5 feet. The measured groundwater level at each boring location is shown adjacent to the soil profiles presented in Figure 1.

It should be emphasized that the groundwater depth measured at each borehole location is indicative of the prevailing groundwater level at the time of measurement. The groundwater level in the site is expected to fluctuate throughout each year, mainly due to seasonal variations in rainfall amounts.

Based on the results of our borings and review of the Soil Survey Report, it is our opinion that the measured groundwater level on site is representative of the normal wet season groundwater level for the site. To accommodate for some temporary mounding of the groundwater level during heavy or extended rainfall amounts, we are estimating the high wet season groundwater level a few inches above the elevations measured during our study. For design purposes, we recommend assuming a high wet season groundwater table at an average elevation of +93 feet.

It should be noted that the estimated high wet season groundwater level provided above represents our anticipated high groundwater level, assuming normal rainfall events each year. This is not an assurance that the groundwater level cannot rise to shallower depths in the future. During years when normal rainfall quantities are exceeded and particularly during extended or prolonged rainfall events, the groundwater table could temporarily rise slightly above our estimated levels.

LABORATORY ANALYSIS

Our laboratory analyses included the visual classification of all soil samples recovered from the borings. As indicated earlier, the results of our visual interpretations are presented in the form of Soil Profiles shown in the attached Figure 1.

In addition to the visual classification of the soils, falling head permeability tests were performed on the recovered undisturbed soil samples. The results of these tests indicate that the coefficient of vertical permeability (k_v) of soil stratum #1 is equivalent to 18 feet per day. The coefficient of vertical permeability (k_v) of soil stratum #2 is equivalent to 26 feet per day Based on our experience, a coefficient of horizontal permeability (k_h) equivalent to at least 1.5 times the coefficient of vertical permeability (k_v) may be assumed.

CONCLUSIONS AND RECOMMENDATIONS

Suitability of Subsurface Soils

Based on the results of this geotechnical study, it is our opinion that following proper pre-construction site preparation activities, the shallow subgrade soils encountered in the property are generally suitable to provide proper bearing support to the proposed 1-story building addition supported on a conventional shallow foundation system (continuous wall footings and isolated column pads beneath concentrated loads). Furthermore, it is our opinion that the shallow subgrade soils are suitable for support of either a flexible or semi-flexible pavement section. Lastly, the shallow subgrade soil and groundwater table conditions are, in our opinion, favorable for proper operation of the dry bottom retention pond and the filtration/drainage system proposed beneath the pervious pavers. More detailed recommendations are provided below.

Pre-Construction Site Preparation

Pre-construction site preparation activities should include normal clearing, grubbing, and stripping of all top-soils, surficial vegetation, existing pavement sections, and other deleterious materials from beneath and to a minimum lateral distance of 5 feet beyond all proposed construction areas.

Once the clearing operations are completed, areas of the site proposed for construction should be proof rolled in order to achieve proper densification of the shallow subgrade soils at the stripped surface. Proof rolling can be performed using a relatively small roller in static mode or other on site construction equipment such as a front-end loader. Large vibratory compaction equipment should be avoided in order to minimize the potential for damaging near-by structures due to equipment vibrations. Areas that yield excessively during the proof rolling operation should be excavated and replaced with suitable granular soils.

Fill Soils

Fill material needed to achieve final site grades within proposed construction areas should consist of non-organic, non-plastic, and debris-free fine sands, preferably containing no more than 5 percent passing the U.S. Standard No. 200 Sieve (fines content). Slightly silty fine sands containing 6 to 12 percent fines may also be used as backfill soils, however, it must be understood that these soils are sensitive to moisture and become difficult to compact when the moisture content is too high. Therefore, moisture control must be exercised if these types of fill soils are used. If the soils become excessively wet and start pumping, then drying the soil by excavation/aeration will become necessary; otherwise removal and replacement with drier soils will become warranted. Fill material containing more than 12 percent fines content should be avoided, if possible.

Fill soils placed around the building and pavement areas should be placed in a manner to allow for positive drainage of storm water runoff away from the building and pavement areas. The fill soils should be placed in loose lifts not exceeding 12 inches in thickness and should be compacted as needed to achieve a minimum density equivalent to 95 percent of the soil's Maximum Modified Proctor Density (ASTM D-1557) value.

This compaction criterion should be achieved by all fill soils or by all foundation bearing soils that lie within a minimum depth of two feet below the bottom of the proposed foundation system, whichever is greater in depth. To facilitate the compaction efforts, the fill soils should have a moisture content that is within 2% of the soil's Optimum Moisture Content.

Fill soils placed in utility line trenches should also be properly placed and compacted as specified above. However, in these restricted working areas, compaction should be accomplished with lightweight, hand-guided compaction equipment and lift thicknesses should be limited to a maximum of 6 to 8 inches loose thickness.

Allowable Soil Bearing Pressure

Assuming that the site preparation activities are accomplished as recommended above, a maximum net allowable soil bearing pressure of 2,000 pounds per square foot (PSF) may be used for foundation sizing. This assumes that all new fill soils placed on site or all in-situ soils to a minimum depth of 2 feet below bottom of foundations, whichever is greater in depth, have been compacted to at least 95% of the soil's Maximum Modified Proctor Density value.

Foundation Support - Building Addition

Continuous footings below stem walls should be a minimum of 16 to 20 inches in width and 8 to 10 inches in thickness, regardless of the resulting bearing pressures. In the event any columns are proposed, column pads should be at least 24 inches by 24 inches (length and width) and 10 to 12 inches in thickness. All continuous footings below stem walls and individual column pads should be embedded at least 16 to 20 inches, as measured from the bottom of the footings/column pads to the overlying ground surface.

If a thickened edge slab-on-grade (monolithic) type foundation system is planned, then the height of the thickened edge (vertical distance between top of floor slab and bottom of thickened edge) should be no less than 16 to 20 inches and the bottom of the footing should be embedded at least 12 inches below the finished exterior ground cover. The bottom width of the thickened edge of the slab should be at least 16 to 20 inches.

All foundations should be properly reinforced with steel reinforcing bars. The Project Structural Engineer or Architect should determine the appropriate steel reinforcement in wall footings and column pads.

As discussed above, the depth of embedment of all foundations should be at least 16 to 20 inches as measured from the bottom of the footings to the overlying ground surface. However, at locations where new footings “butt up” to existing footings, we recommend that the new footings be placed at the same depth as the existing footing. This will minimize the potential for superimposing loads from one foundation system onto the other.

Settlement of Foundation Soils

Settlement of the subgrade soils will be primarily elastic in nature, i.e., occurring simultaneously upon application of the loads. The magnitude of settlement below the addition was computed using the estimated structural loading conditions discussed earlier in this report. The results of our calculations indicated that total settlement of the soils should not exceed ¾ - inch and differential settlement should be less than ⅓ - inch.

Building Slab-on-Grade

The interior floor slab of the addition may be supported on approved structural fill soils, compacted to at least 95% of the Modified Proctor Density value, as tested to a minimum depth of 12 inches below bottom of the slab.

It is recommended that the floor slab bearing soils be covered by a lapped polyethylene sheeting in order to minimize the potential for floor dampness which can affect the performance of tiles and carpet. This membrane should consist of a minimum six (6) mil single layer of non-corroding, non-deteriorating sheeting material placed to minimize seams and to cover all of the soil below the building floor. The membrane should be cut in cross shape for pipes or other penetrations and should extend to within ½ inch of all penetrations. All seams of the membrane should be lapped at least 12 inches. Punctures or tears in the membrane should be repaired with the same or compatible material.

The interior floor slab should be at least 4 inches in thickness and should be constructed using a concrete mix that is capable of achieving a minimum 28-day compressive strength equivalent to 2,500 psf. Within 24 hours after placement of the concrete, expansion joints should be cut in the floor slab as recommended by applicable standards. This will minimize the formation of random cracking in the slab as the concrete hydrates/cures. Control joints should be saw cut in a manner that creates a square configuration in the concrete pavement section. The depth of the saw cut/control joints should be at least one-third the thickness of the concrete section.

Pavement Construction

It is was not known to us whether you plan on removing all existing pavement sections and constructing new sections or if you plan on milling and re-paving the exiting asphalt layer. If you plan on removing the existing asphaltic pavement without removing the underlying base and subbase, then our firm should be notified once the asphalt layer is removed so we may inspect the existing base material and determine whether the in-situ base material can be utilized or whether removal and replacement of the existing base material is required.

The recommendations outlined below assume that the existing pavement section will be removed in its entirety and replaced with a new section. Based on the results of the borings performed, it is our opinion that the soils encountered are suitable for support of either a flexible or semi-flexible pavement section, following proper site preparation and subgrade soil compaction, as discussed earlier.

Provided that at least 18 inches of separation can be provided/maintained between the estimated high wet season groundwater level and the bottom of the proposed pavement base course material, pavement under drains will not be required.

Asphaltic Pavement Sub-base/Subgrade: It is recommended that a stabilized subgrade, consisting of 12 inches or more of soils compacted to at least 98 percent of the Modified Proctor Moisture-Density Test (AASHTO T-180), with a minimum Limerock Bearing Ratio (LBR) value of 40 psi, be constructed below the pavement base material. Please note that the in-situ, near ground surface soils encountered in the property are mostly granular in nature and therefore, will not produce the specified minimum LBR value discussed above. Therefore, the addition and mixing of clayey soils or crushed fines with the in-situ sandy soils should be anticipated in order to properly stabilize the pavement sub-base/subgrade. A properly mixed 40/60 blend of clayey soils / crushed fines with the in situ sandy soils should be sufficient to produce the desired LBR value. If needed, a mix design can be performed to determine the optimum proportions of soil/clay and/or soil/crushed fines to produce the desired LBR value.

Asphaltic Pavement Base: Depending upon the final proposed pavement elevations, either limerock or soil-cement may be selected as a pavement base course. Provided that a minimum separation of 18 inches can be maintained between the estimated wet season groundwater level and the bottom of the base course material, a limerock base may be used. A crushed concrete fines base material may be used in lieu of limerock, if desired. In the event that final pavement grades allow for a separation of less than 18 inches between the estimated wet season groundwater table and the bottom of the base course, then we recommend that a soil-cement base be utilized as the pavement base course, since it is more resistant to groundwater related degradation than the limerock alternative.

If either limerock or crushed fines is selected as the base material, the base should be at least 6 inches in thickness in relatively light traffic load areas (cars and small truck traffic, etc.) and 8 inches in thickness in heavy traffic load areas (such as entrance/exit driveways, loading/unloading areas, trash collection lanes, etc.). The base should be compacted to a minimum of 98 percent of the Modified Proctor Moisture-Density Test (AASHTO T-180). If limerock is selected, the limerock material should exhibit a minimum LBR value of 100 psi. If crushed concrete fines is selected, the crushed fines should exhibit a minimum LBR value of 125 psi.

If a soil cement base is utilized, the base should also be at least 6 inches in thickness in normal/light traffic areas and 8 inches in thickness in heavy traffic areas. The base should be compacted to a minimum of 95% of the Standard Proctor Moisture-Density Test (AASHTO T-134) and should achieve a 7-day laboratory compressive strength equivalent to at least 300 psi. It is our opinion that a stabilized sub-base is not needed with the soil-cement base, however, the upper 12 inches of subgrade (below the base course) shall be compacted to at least 98% of the Modified Proctor Moisture-Density Test (AASHTO T-180). Please note that some jurisdictions may require a stabilized subgrade below the pavement base material, regardless of the type of base material used.

Due to the shrinkage cracking which normally occurs during the hydration of a soil-cement base, we recommend that a curing period of at least 14 to 21 days be allowed prior to the placement of the overlying asphaltic concrete wearing surface. Although this will not eliminate the formation of cracks in the finished asphaltic surface (due to cracking of the underlying base), it should help minimize the magnitude/size of the cracks. An appropriate sealant should be applied on the base within a maximum of one day after placement in order to minimize moisture loss during the hydration process.

Asphaltic Pavement Surface Course: The asphaltic wearing surface should consist of a minimum of 1½ inches in light traffic load areas and 2 in heavy traffic load areas of Type SP (super pave) compacted to 93 to 94 percent of the Maximum Specific Gravity (G_{mm}) value of the design mix. Type S-I or S-III asphaltic concrete having a minimum Marshall Stability of 1,200 pounds in light traffic areas and 1,500 pounds in heavy traffic areas and compacted to at least 95 percent of the laboratory mix design can also be used. Specific requirements for the design and application of asphaltic concrete are outlined in the Florida Department of Transportation Standard Specifications for Road and Bridge Construction.

Before paving begins, the surface of the base should be machine swept to remove loose particles and other debris. Thereafter, an approved emulsified asphaltic tack coat should be applied, in sufficient quantities, on the surface of the clean base material in order to develop a sufficient bond between the base and the overlying asphalt.

Rigid/Concrete Pavement Section: Because concrete pavement is more rigid and transfers less wheel loads to the underlying subgrade soils, it is our opinion that a stabilized subgrade is not required if a concrete pavement section is used. However, the underlying subgrade soils must be compacted to at least 95% of the soil's Maximum Modified Proctor Density value.

The concrete pavement should be at least 6 inches in thickness in light traffic load areas and at least 7 inches in thickness in heavy traffic load areas. The concrete should exhibit a minimum 28-day compressive strength equivalent to 4,000 psi. Maximum spacing between control joints should not exceed 12 feet by 12 feet.

It is recommended that thickened edges be provided below the concrete pavement around landscaped islands and along the edges of the pavement. This will minimize the potential for soil erosion below the edges of the pavement section.

Retention Pond and Ex-Filtration Trenches/Pervious Paver System

As discussed earlier, we understand that storm water runoff from this project will be collected and treated on site via a dry-bottom retention pond, along with several pervious paver drain systems proposed in/below parking stall areas. Based on the results of our borings, it is our opinion that the subgrade soils and groundwater table are favorable for proper operation of the proposed storm water management system. For design purpose, the following soil and groundwater table parameters may be assumed:

- A high wet season groundwater level assumed at an elevation of +93 feet

- An average coefficient of vertical permeability through the drainage aquifer equivalent to 22 feet per day. An average coefficient of horizontal permeability equivalent to 1.5 times the vertical permeability (i.e., 33 feet per day) may be assumed. While performing the storm water recovery analyses, the rates of percolation provided above should be reduced by 50% in order to incorporate a Design Factor of Safety equivalent to 2.

- An average porosity of the soils underlying the pond bottom equivalent to 25%,

- A confining layer (bottom of aquifer) assumed at an average elevation of +91.5 feet.

Storm Water Recovery Analyses

Based on information provided by the project Civil Engineer, we understand that the pond and all drainage systems proposed in/below pervious pavers will have a bottom elevation of +94 feet and a top elevation of +96 feet. All systems are hydraulically connected and will therefore, essentially operate as a single unit. We were requested to perform seepage analyses to verify that the pollution abatement volume (2,102 cubic feet) can be recovered within 72 hours and that the total storage volume in the system (6,771.7 cubic feet) can be recovered in 14 days.

The configuration of the drainage system, the incremental stage area/volume, and the soil and groundwater table parameters determined from our soil study were used as input parameters into the computer program "PONDS" in order to evaluate the storm water recovery time through the system.

The results of the seepage analysis are attached. As indicated by our analyses, the pollution abatement volume and the total storage volume are both recovered within the 3 and 14 - day allowed time period, respectively.

CLOSURE

The conclusions and recommendations provided in this report were based on the subsoil conditions encountered in our shallow auger borings. It is assumed that the subsurface profile depicted by our borings is representative of the subsurface profile in all portions of the site. If during construction activities, variations in the subsurface profile are encountered, our firm should be notified immediately so we may re-evaluate the conclusions and recommendations provided in this report and make any revisions as deemed necessary by the Project Engineer.

We have appreciated the opportunity of providing our geotechnical engineering services to you on this project and trust that the information presented in this report is satisfactory. Please do not hesitate to contact the undersigned should you have any questions or require further information.

Sincerely,
MIKE TANNOUS ENGINEERING, INC.

10/22/21
Mike S. Tannous, P.E.
Principal Engineer
Florida Registration No. 46009

Attachment: Figure 1 - Boring Location Plan and Soil Profiles
Results of Seepage Analyses

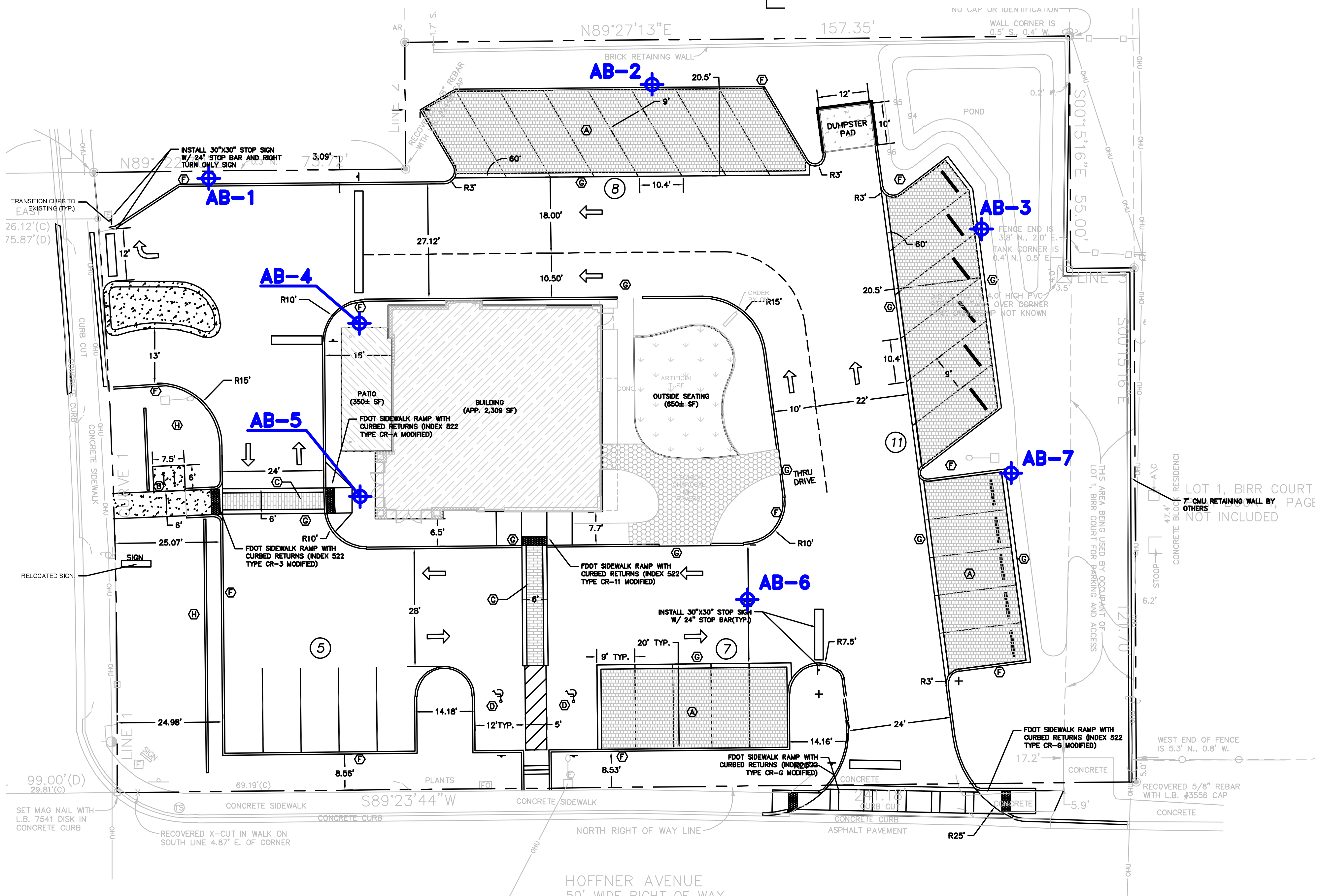
APPENDIX A

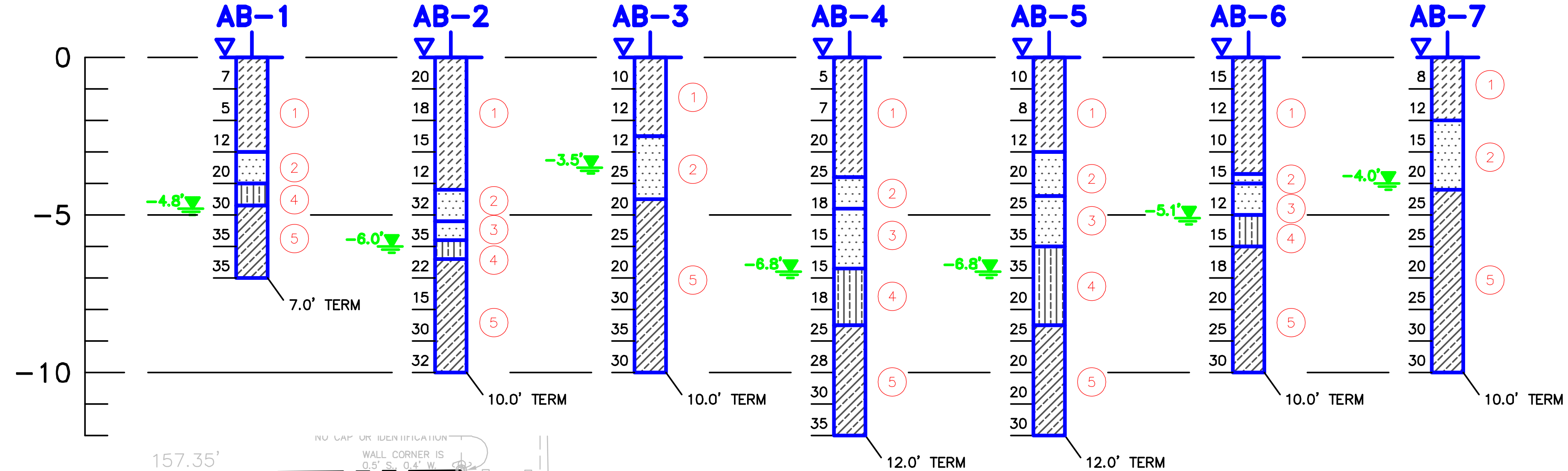
FIGURE 1 - BORING LOCATIONS AND SOIL PROFILES

LOCATION PLAN

APPROXIMATE SCALE: 1"=30'

APPROXIMATE AUGER BORING LOCATION





SOIL PROFILES

SCALE: 1"=5'

1. VARYING SHADES OF GRAY AND BROWN FINE SAND TO SLIGHTLY SILTY FINE SANDS (FILL)
2. GRAYISH BROWN FINE SAND (SP)
3. VERY PALE BROWN FINE SAND (SP)
4. VERY PALE BROWN SLIGHTLY SILTY TO SILTY FINE SAND (SP-SM)(SM)
5. DARK REDDISH BROWN TO REDDISH BROWN SILTY FINE SAND WITH CEMENTED SANDS (SM)

(SP-SM) UNIFIED SOIL CLASSIFICATION GROUP SYMBOL AS DETERMINED BY VISUAL EXAMINATION

▽ HAND PENETROMETER PROBE READING

-4.8' DEPTH TO GROUNDWATER TABLE

BORING LOCATIONS & SOIL PROFILES MECATOS BAKERY 5645 HANSEL AVENUE EDGEWOOD, FLORIDA		
MTE MIKE TANNOUS ENGINEERING INCORPORATED		
DRAWN: RLG	APPROVED: MT	DATE: 10/21/21
SCALE: NOTED	FIGURE: 1	JOB NO.: 21-325

APPENDIX B

STORM WATER RECOVERY ANALYSES
FOR
POLLUTION ABATEMENT VOLUME

PONDS Version 3.3.0278
Retention Pond Recovery - Refined Method

Devo Seereeram, Ph.D., P.E.

Project Data

Project Name: MECATOS BAKERY

Simulation Description: PAV

Project Number:

Engineer : mst

Supervising Engineer:

Date: 10-20-2021

Aquifer Data

Base Of Aquifer Elevation, [B] (ft datum):	91.50
Water Table Elevation, [WT] (ft datum):	93.00
Horizontal Saturated Hydraulic Conductivity, [Kh] (ft/day):	16.50
Fillable Porosity, [n] (%):	25.00

Vertical infiltration was not considered.

Geometry Data

Equivalent Pond Length, [L] (ft):	120.0
Equivalent Pond Width, [W] (ft):	70.0

Ground water mound is expected to intersect the pond bottom

Stage vs Area Data

Stage (ft datum)	Area (ft^2)
94.00	5515.0
94.50	5758.0
95.00	6039.0
95.50	6318.0
96.00	8575.0

Ditch Data

Ditch (or interceptor trench) parallel to length axis is inactive

Ditch (or interceptor trench) parallel to width axis is inactive

Discharge Structures

Discharge Structure #1 is inactive

Discharge Structure #2 is inactive

Discharge Structures (cont'd.)

Discharge Structure #3 is inactive

Scenario Input Data

Scenario 1 :: PAV

Hydrograph Type: Slug Load
Modflow Routing: Routed with infiltration

Treatment Volume (ft^3) 2102

Initial ground water level (ft datum) 93.00 (default)

Time After Storm Event (days)	Time After Storm Event (days)
0.100	2.000
0.250	2.500
0.500	3.000
1.000	3.500
1.500	4.000

PONDS Version 3.3.0278
Retention Pond Recovery - Refined Method

Devo Seereeram, Ph.D., P.E.

Detailed Results *:: Scenario 1 :: PAV*

Elapsed Time (hours)	Instantaneous Inflow Rate (ft³/s)	Outside Recharge (ft/day)	Stage Elevation (ft datum)	Infiltration Rate (ft³/s)	Combined Instantaneous Discharge Rate (ft³/s)	Cumulative Inflow Volume (ft³)	Cumulative Infiltration Volume (ft³)	Combined Cumulative Discharge (ft³)	Flow Type
0.000	350.3333	0.00000	94.00000	0.14963	0	0.000	0.0	0	N.A.
0.002	350.3333	0.00000	94.37479	0.14950	0	2102.000	0.9	0	S
2.400	0.0000	0.00000	94.29203	0.04247	0	2102.000	470.7	0	S
6.000	0.0000	0.00000	94.23570	0.02090	0	2102.000	788.6	0	S
12.000	0.0000	0.00000	94.17856	0.01306	0	2102.000	1109.5	0	S
24.000	0.0000	0.00000	94.10542	0.00824	0	2102.000	1517.9	0	S
36.000	0.0000	0.00000	94.05083	0.00632	0	2102.000	1821.0	0	S
48.000	0.0000	0.00000	94.00684	0.00325	0	2102.000	2064.3	0	S
60.000	0.0000	0.00000	93.93560	0.00044	0	2102.000	2102.0	0	S
72.000	0.0000	0.00000	93.87260	0.00000	0	2102.000	2102.0	0	S
84.000	0.0000	0.00000	93.82117	0.00000	0	2102.000	2102.0	0	S
96.000	0.0000	0.00000	93.77779	----	----	2102.000	2102.0	0	N.A.

PONDS Version 3.3.0278
Retention Pond Recovery - Refined Method
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Devo Seereeram, Ph.D., P.E.

Summary of Results :: *Scenario 1 :: PAV*

	Time (hours)	Stage (ft datum)	Rate (ft^3/s)	Volume (ft^3)
Stage				
Minimum	96.000	93.78		
Maximum	0.002	94.37		
Inflow				
Rate - Maximum - Positive	0.002		350.3333	
Rate - Maximum - Negative	None		None	
Cumulative Volume - Maximum Positive	0.002			2102.0
Cumulative Volume - Maximum Negative	None			None
Cumulative Volume - End of Simulation	96.000			2102.0
Infiltration				
Rate - Maximum - Positive	0.002		0.1495	
Rate - Maximum - Negative	None		None	
Cumulative Volume - Maximum Positive	60.000			2102.0
Cumulative Volume - Maximum Negative	None			None
Cumulative Volume - End of Simulation	96.000			2102.0
Combined Discharge				
Rate - Maximum - Positive	None		None	
Rate - Maximum - Negative	None		None	
Cumulative Volume - Maximum Positive	None			None
Cumulative Volume - Maximum Negative	None			None
Cumulative Volume - End of Simulation	96.000			0.0
Discharge Structure 1 - inactive				
Rate - Maximum - Positive	disabled		disabled	
Rate - Maximum - Negative	disabled		disabled	
Cumulative Volume - Maximum Positive	disabled			disabled
Cumulative Volume - Maximum Negative	disabled			disabled
Cumulative Volume - End of Simulation	disabled			disabled
Discharge Structure 2 - inactive				
Rate - Maximum - Positive	disabled		disabled	
Rate - Maximum - Negative	disabled		disabled	
Cumulative Volume - Maximum Positive	disabled			disabled
Cumulative Volume - Maximum Negative	disabled			disabled
Cumulative Volume - End of Simulation	disabled			disabled
Discharge Structure 3 - inactive				
Rate - Maximum - Positive	disabled		disabled	
Rate - Maximum - Negative	disabled		disabled	
Cumulative Volume - Maximum Positive	disabled			disabled
Cumulative Volume - Maximum Negative	disabled			disabled
Cumulative Volume - End of Simulation	disabled			disabled
Pollution Abatement:				
36 Hour Stage and Infiltration Volume	36.000	94.05		1821.0
72 Hour Stage and Infiltration Volume	72.000	93.87		2102.0

APPENDIX C

STORM WATER RECOVERY ANALYSES
FOR
ENTIRE STORAGE VOLUME

PONDS Version 3.3.0278
Retention Pond Recovery - Refined Method
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Devo Seereeram, Ph.D., P.E.

Project Data

Project Name:	MECATOS BAKERY
Simulation Description:	ENTIRE STORAGE VOLUME
Project Number:	21-325
Engineer :	MST
Supervising Engineer:	
Date:	10-20-2021

Aquifer Data

Base Of Aquifer Elevation, [B] (ft datum):	91.50
Water Table Elevation, [WT] (ft datum):	93.00
Horizontal Saturated Hydraulic Conductivity, [Kh] (ft/day):	16.50
Fillable Porosity, [n] (%):	25.00

Vertical infiltration was not considered.

Geometry Data

Equivalent Pond Length, [L] (ft):	120.0
Equivalent Pond Width, [W] (ft):	70.0

Ground water mound is expected to intersect the pond bottom

Stage vs Area Data

Stage (ft datum)	Area (ft^2)
94.00	5515.0
94.50	5758.0
95.00	6039.0
95.50	6318.0
96.00	8575.0

Ditch Data

Ditch (or interceptor trench) parallel to length axis is inactive

Ditch (or interceptor trench) parallel to width axis is inactive

Discharge Structures

Discharge Structure #1 is inactive

Discharge Structure #2 is inactive

Discharge Structures (cont'd.)

Discharge Structure #3 is inactive

Scenario Input Data

Scenario 1 ::

Hydrograph Type: Slug Load
Modflow Routing: Routed with infiltration

Treatment Volume (ft^3) 6771.7

Initial ground water level (ft datum) 93.00 (default)

Time After Storm Event (days)	Time After Storm Event (days)	Time After Storm Event (days)	Time After Storm Event (days)
0.100	2.000	6.000	15.000
0.250	2.500	8.000	16.000
0.500	3.000	10.000	
1.000	3.500	12.000	
1.500	4.000	14.000	

PONDS Version 3.3.0278
Retention Pond Recovery - Refined Method
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Devo Seereeram, Ph.D., P.E.

Detailed Results *:: Scenario 1 ::*

Elapsed Time (hours)	Instantaneous Inflow Rate (ft³/s)	Outside Recharge (ft/day)	Stage Elevation (ft datum)	Infiltration Rate (ft³/s)	Combined Instantaneous Discharge Rate (ft³/s)	Cumulative Inflow Volume (ft³)	Cumulative Infiltration Volume (ft³)	Combined Cumulative Discharge (ft³)	Flow Type
0.000	1128.6170	0.00000	94.00000	0.23597	0	0.000	0.0	0	N.A.
0.002	1128.6170	0.00000	95.16479	0.23578	0	6771.700	1.4	0	S
2.400	0.0000	0.00000	95.03077	0.07356	0	6771.700	818.1	0	S
6.000	0.0000	0.00000	94.94054	0.03577	0	6771.700	1362.3	0	S
12.000	0.0000	0.00000	94.84881	0.02229	0	6771.700	1910.8	0	S
24.000	0.0000	0.00000	94.73141	0.01399	0	6771.700	2606.0	0	S
36.000	0.0000	0.00000	94.64379	0.01070	0	6771.700	3119.7	0	S
48.000	0.0000	0.00000	94.57324	0.00873	0	6771.700	3530.2	0	S
60.000	0.0000	0.00000	94.51385	0.00740	0	6771.700	3873.7	0	S
72.000	0.0000	0.00000	94.46239	0.00644	0	6771.700	4169.7	0	S
84.000	0.0000	0.00000	94.41690	0.00571	0	6771.700	4430.3	0	S
96.000	0.0000	0.00000	94.37609	0.00511	0	6771.700	4663.2	0	S
144.000	0.0000	0.00000	94.25488	0.00354	0	6771.700	5350.3	0	S
192.000	0.0000	0.00000	94.15926	0.00282	0	6771.700	5887.2	0	S
240.000	0.0000	0.00000	94.08096	0.00232	0	6771.700	6323.6	0	S
288.000	0.0000	0.00000	94.01509	0.00130	0	6771.700	6688.4	0	S
336.000	0.0000	0.00000	93.92348	0.00016	0	6771.700	6771.7	0	S
360.000	0.0000	0.00000	93.87716	0.00000	0	6771.700	6771.7	0	S
384.000	0.0000	0.00000	93.83741	----	----	6771.700	6771.7	0	N.A.

PONDS Version 3.3.0278
Retention Pond Recovery - Refined Method

Devo Seereeram, Ph.D., P.E.

Summary of Results :: *Scenario 1* ::

	Time (hours)	Stage (ft datum)	Rate (ft^3/s)	Volume (ft^3)
Stage				
Minimum	384.000	93.84		
Maximum	0.002	95.16		
Inflow				
Rate - Maximum - Positive	0.002		1128.6170	
Rate - Maximum - Negative	None		None	
Cumulative Volume - Maximum Positive	0.002			6771.7
Cumulative Volume - Maximum Negative	None			None
Cumulative Volume - End of Simulation	384.000			6771.7
Infiltration				
Rate - Maximum - Positive	0.002		0.2358	
Rate - Maximum - Negative	None		None	
Cumulative Volume - Maximum Positive	336.000			6771.7
Cumulative Volume - Maximum Negative	None			None
Cumulative Volume - End of Simulation	384.000			6771.7
Combined Discharge				
Rate - Maximum - Positive	None		None	
Rate - Maximum - Negative	None		None	
Cumulative Volume - Maximum Positive	None			None
Cumulative Volume - Maximum Negative	None			None
Cumulative Volume - End of Simulation	384.000			0.0
Discharge Structure 1 - inactive				
Rate - Maximum - Positive	disabled		disabled	
Rate - Maximum - Negative	disabled		disabled	
Cumulative Volume - Maximum Positive	disabled			disabled
Cumulative Volume - Maximum Negative	disabled			disabled
Cumulative Volume - End of Simulation	disabled			disabled
Discharge Structure 2 - inactive				
Rate - Maximum - Positive	disabled		disabled	
Rate - Maximum - Negative	disabled		disabled	
Cumulative Volume - Maximum Positive	disabled			disabled
Cumulative Volume - Maximum Negative	disabled			disabled
Cumulative Volume - End of Simulation	disabled			disabled
Discharge Structure 3 - inactive				
Rate - Maximum - Positive	disabled		disabled	
Rate - Maximum - Negative	disabled		disabled	
Cumulative Volume - Maximum Positive	disabled			disabled
Cumulative Volume - Maximum Negative	disabled			disabled
Cumulative Volume - End of Simulation	disabled			disabled
Pollution Abatement:				
36 Hour Stage and Infiltration Volume	36.000	94.64		3119.7
72 Hour Stage and Infiltration Volume	72.000	94.46		4169.7