STORMWATER MANAGEMENT PLAN

Industrial Building
901 Northview Road

City of Waukesha, Wisconsin

PEG Project Number: 426.00

June 8, 2015

PINNACLE ENGINEERING GROUP
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APPENDICIES

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INTRODUCTION

The 11.5 acre site is located at the southwest corner of Northview Road and Aviation Drive in the City of Waukesha, Wisconsin. Included on the site plan is a 214,500 SF building, along with 83 car stalls and 62 truck stalls parking surrounding the building.

The site is located just south of the Waukesha County Expo Center and of the Waukesha County Airport. There are wetlands located on the south part of the site along with an existing storm water detention pond. The pond provides a place for outfall for the site, but on-site measures will be needed to meet local water quality standards.

DESIGN CRITERIA

City of Waukesha:......... Stormwater Management and Erosion Control Ordinance, Chapter 32

Wisconsin Department of Natural Resources:... NR 216 & NR 151

Water Quantity: *City Ordinance 32.10* – Peak Discharge Requirements- Standard presents maximum allowable, post development runoff release rates shall not exceed the calculated pre-development discharge rates for the 2-year, 10-year, and 100-year storm.

Water Quality: *WDNR NR 151.122/City Ordinance 32.10* – Removal of 40% of the annual total suspended solids (TSS) load for a redevelopment.

Infiltration: *City Ordinance 32.10 - Non-residential* **-** Infiltrate 60% of the average annual pre-development infiltration volume, or 10% of the 2-year, 24-hour storm, or provide an effective infiltration area equal to at least 2% of the total site area.

ANALYSIS METHODS

HydroCAD® (Version 10.1) Storm Water Modeling System software has been used to analyze stormwater characteristics for the Uline Corporate Campus management plan. HydroCAD® uses the accepted TR-55 methodology for determining peak discharge runoff rates. Curve Numbers for the proposed ground cover were selected using the standard values specified in TR-55 for a “C” soil type. Stormwater modeling was conducted using 2-year, 10-year, and 100-year storm events with respective rainfall amounts of 2.7, 4.0, and 5.6 inches in accordance with the City of Waukesha Stormwater Ordinance. HydroCAD® output data for post-development conditions is located in **Appendix 3**.

Sediment reduction characteristics for the proposed water quality facilities were determined using WinSLAMM® (Version 10.1.6) Source Loading and Management Model. WinSLAMM® output can be viewed in **Appendix 4**.

EXISTING CONDITIONS

Currently, the site consists of approximately 11.5 acres of impervious land. The site is a former bowling alley surrounded by an asphalt parking lot. The site predominantly drains from the north to the south of the site to an existing storm water pond. A portion of the north side of the site, along with some roof drainage, discharges into an existing ditch on the south side of Northview Road. The closest storm sewer is found on the southeast side of the site within the Aviation Drive ROW.

POST-DEVELOPMENT CONDITIONS

The proposed development includes a new parking area, three driveways, and a 214,500 square foot building. Underground storm sewer is proposed to capture the roof drainage, as well as the runoff from the parking lot. The roof drainage will not be connected to the parking lot storm sewer running to the underground chambers because it does not have to be treated per NR151 (for redevelopment). The parking lot storm sewer (excluding roof) will be sent to two underground storm water chambers where the water will be treated. The south portion of the site will outfall into the existing storm water pond. The north portion of the site will discharge into the existing ditch on the south side of Northview Road. The proposed site reduces imperviousness, so the existing flows will be reduced without any on site detention. However, additional storage is provided within the underground chambers. This storage however, is not modeled.

EXISTING/PROPOSED 2, 10, 100 YEAR FLOW TABLES

EXISTING SITE

Area	Area (ac)	CN	Tc (min)	Peak Flows 2-year (cfs)	Peak Flows 10-year (cfs)	Peak Flows 100-year (cfs)
Exist Site North	3.83	94	6.0*	12.81	20.04	28.82
Exist Site South	7.67	94	6.0*	25.65	40.12	57.71
Total	11.5	94	6.0*	**38.46**	**60.16**	**86.53**

*** A Tc of 6.0 min is used as the actual computed Tc is less than the minimum allowable Tc of 6.0 min per TR 55.**

PROPOSED SITE

Area	Area (ac)	CN	Tc (min)	Peak Flows 2-year (cfs)	Peak Flows 10-year (cfs)	Peak Flows 100-year (cfs)
Prop Site North	3.7	95	6.0*	12.76	19.69	28.12
Prop Site South	7.8	92	6.0*	24.34	39.20	57.28
Total	11.5	93	6.0*	**37.1**	**58.89**	**85.4**

*** A Tc of 6.0 min is used as the actual computed Tc is less than the minimum allowable Tc of 6.0 min per TR 55.**

Runoff Water Quality

Two underground water quality chambers are proposed to effectively treat the runoff from the proposed parking area. See the table below for the summary. Please refer to **Appendix 4** for WinSLAMM modeling data.

Water Quality Summary

Area/Pond	Area (ac)	Pounds of TSS Generated	Pounds of TSS Remaining	Percent Removal
Underground Chambers	6.62	5458	3246	40.53%

The specified chambers are the Stormtech MC-4500. Additional details for the chamber can also be found in Appendix 4.

Stormwater Infiltration

Per DNR Technical Standard 1002 and City of Waukesha Ordinance, the site is classified as a redevelopment. Redeveloped sites are exempt from infiltration requirements.

Protective Areas

Protective areas are required along all wetlands in order to minimize impacts of pollutants from untreated impervious sources. On this project, wetlands are present on the south side of the existing site. However, all impervious surface drainage is directed to one of the two proposed water quality chambers. The chambers then treat the runoff to the 40% redevelopment standard. Thus, protective areas are not required under this plan.

Stormwater Conveyance System

Storm sewer will be sized to accommodate stormwater runoff associated with the Department of Safety and Professional Services (DSPS) design storm, generalized based on the 10-year design storm event intensity.

Maintenance

Maintenance is expected to occur on a regular basis for the proposed storm water structures. These structures include catch basins, storm sewer and underground chamber. An agreement with the City of Waukesha will be executed to ensure this occurs.

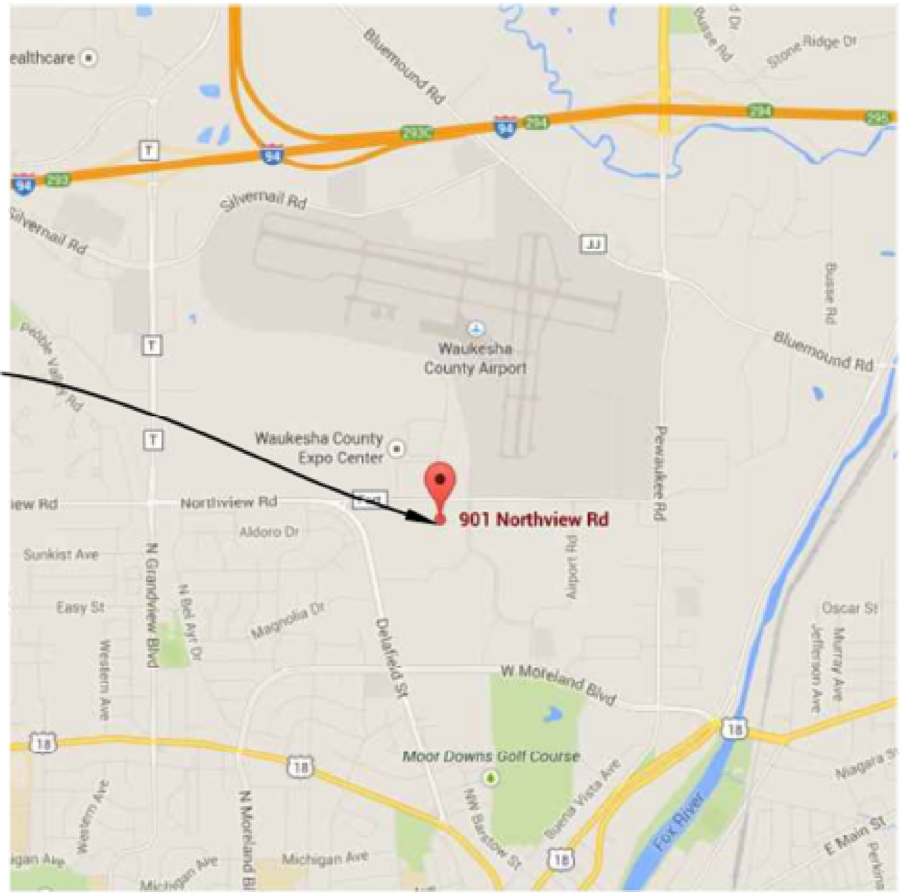
CONCLUSION

The stormwater management features will comply with the City of Waukesha requirements and WDNR technical standards NR216/151. The regional detention basin provides adequate detention for the proposed site, therefore, no on-site detention measures are proposed. Storm water runoff from the site will be treated to remove at least 40% total suspended solids annually from the proposed redevelopment via underground water quality chambers. Infiltration is not required as the site is classified as a redevelopment. Wetlands are present on site, but are not impacted.

Appendix 1

Maps

PROJECT
LOCATION



LOCATION MAP
NTS

PLAT OF SURVEY

CLIENT
HSA Acquisitions, Inc.

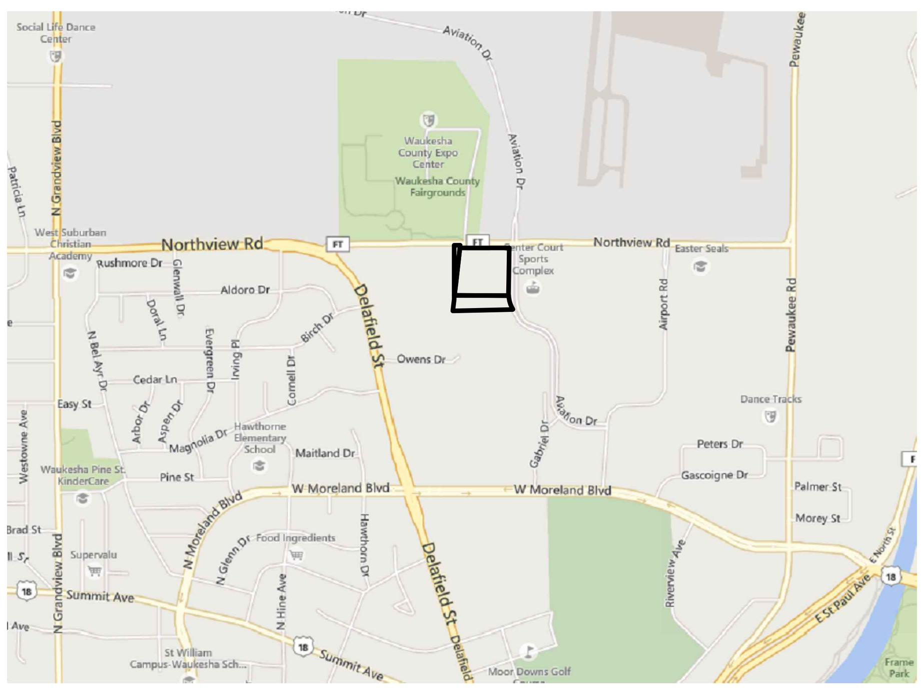
SITE ADDRESS
901 Northview Road, City of Waukesha, County of Waukesha, Wisconsin.

LEGAL DESCRIPTION
Parcel 1 of Certified Survey Map No. 2913, being a part of the Northwest 1/4 of Section 34, Town 7 North, Range 19 East, in the City of Waukesha, County of Waukesha, State of Wisconsin, together with lands adjoining said Parcel 1 which are bounded and described as follows:

Commencing at the Northeast corner of said 1/4 Section; thence due West along the North line of said 1/4 Section 385.85 feet to a point; thence South 01° 03' 10'' West and parallel to the East line of said 1/4 Section 453.00 feet to a point; thence due West 60.02 feet to the Southeast corner of said Parcel 1, being the point of beginning of the land to be described; thence continuing due West 643.15 feet to the Southwest corner of said Parcel 1; thence South 06° 29' 00'' West 161.43 feet to a point; thence South 89° 33' 54'' East 658.35 feet to a point; thence North 01' 03' 10'' East 165.43 feet to the point of beginning.

LAND AREA
The Land Area of the HSA property is 367,697 square feet or 8.4412 acres.
The Lands to be acquired property is 149,121 square feet or 3.4233 acres.

VICINITY MAP

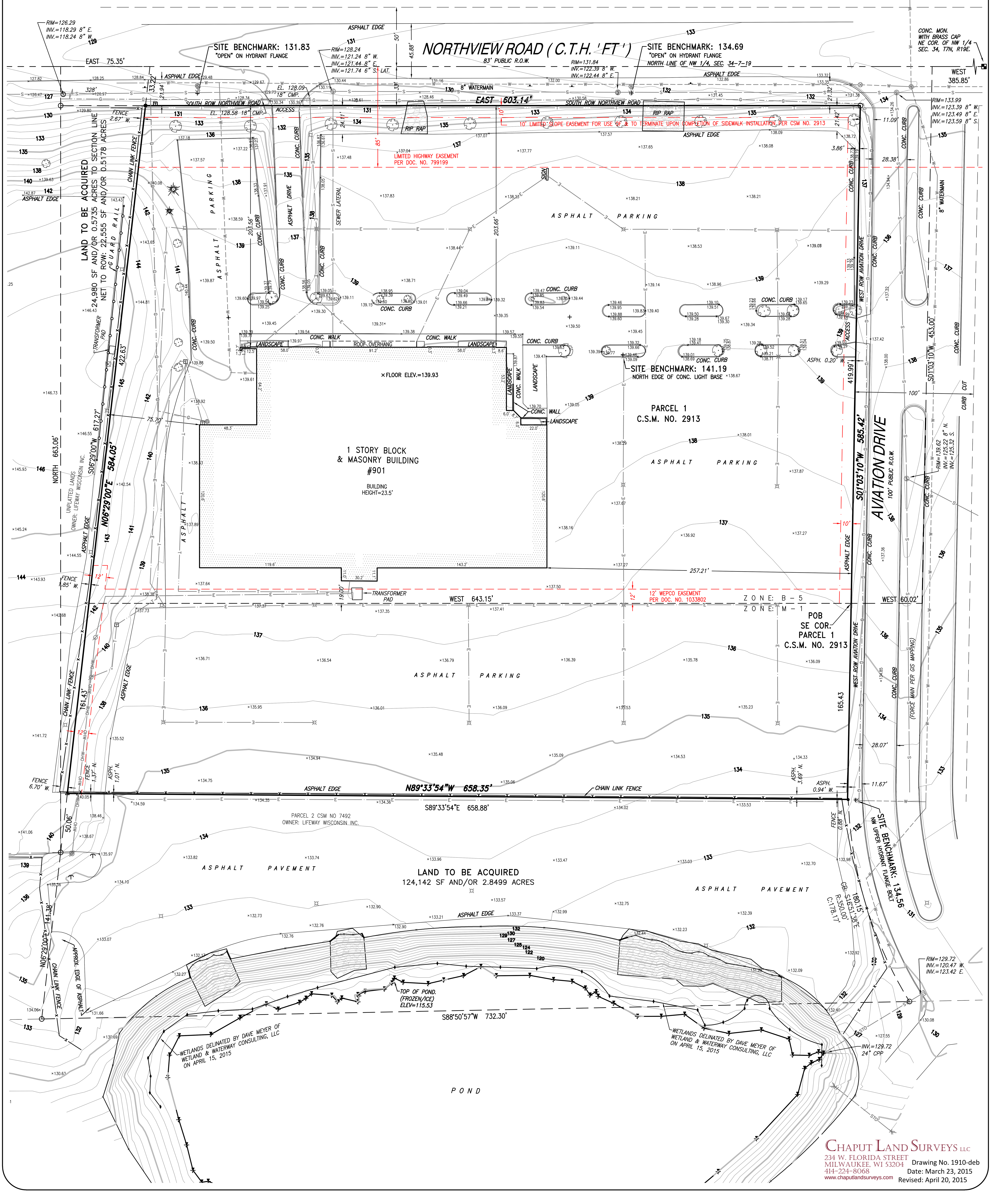


LEGEND

- INDICATES FOUND 1" IRON PIPE
- INDICATES SET 1" IRON PIPE
- INDICATES FOUND CHISELED CROSS
- SANITARY MANHOLE
- SANITARY CLEANOUT OR VENT
- M.I.S. MANHOLE
- UNKNOWN MANHOLE
- STORM MANHOLE
- INLET (ROUND)
- INLET (SQUARE)
- STORM SEWER END SECTION
- GAS VALVE
- GAS METER
- WATER VALVE
- HYDRANT
- WATER MANHOLE
- WATER SERVICE CURB STOP
- WELL HEAD
- STAND PIPE
- WALL INDICATOR VALVE
- POST INDICATOR VALVE
- LIGHT POLE
- SPOT/YARD LIGHT
- UTILITY POLE
- GUY POLE
- GUY WIRE
- ELECTRIC MANHOLE
- ELECTRIC PEDESTAL
- ELECTRIC METER
- TELEPHONE MANHOLE
- TELEPHONE PEDESTAL
- CABLE PEDESTAL
- CONTROL BOX
- FIBER OPTIC SIGN
- TRAFFIC LIGHT
- COMMUNICATION MANHOLE
- BOLLARD
- SOIL BORING/MONITORING WELL
- WATER SURFACE
- WETLANDS FLAG
- MARSH
- FLAGPOLE
- PARKING METER
- SIGN
- MAILBOX
- RAILROAD CROSSING SIGNAL
- HANDICAP SPACE
- CONIFEROUS TREE
- DECIDUOUS TREE
- SANITARY SEWER
- STORM SEWER
- WATERLINE
- MARKED GAS MAIN
- MARKED ELECTRIC
- OVERHEAD WIRES
- MARKED TELEPHONE
- MARKED CABLE TV LINE
- MARKED FIBER OPTIC
- FENCE

GRAPHIC SCALE
40 0 20 40 80 160
(IN FEET)
1 inch = 40 ft.

NOTE
Horizontal datum is based on the South line of CSM No. 2913 which is assumed to bear North 00°00'00" East.
Vertical datum is based on City of Waukesha Datum = NGVD Datum -780.55'.

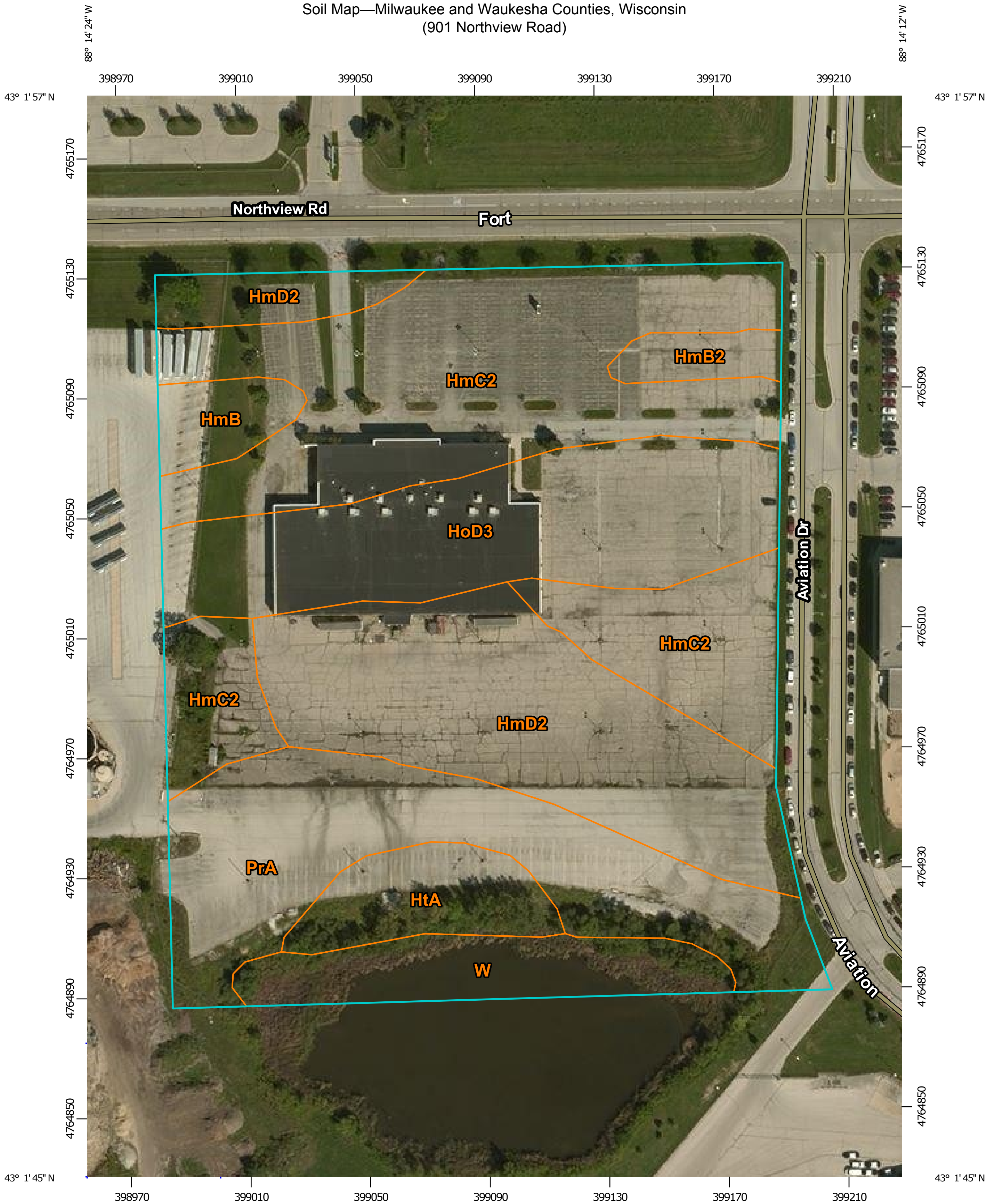


CHAPUT LAND SURVEYS LLC

CHAPUT LAND SURVEYS LLC
234 W. FLORIDA STREET
MILWAUKEE, WI 53204
414-224-8068
www.chaputlandsurveys.com

Drawing No. 1910-deb
Date: March 23, 2015
Revised: April 20, 2015

Soil Map—Milwaukee and Waukesha Counties, Wisconsin
(901 Northview Road)



Map Scale: 1:1,760 if printed on A portrait (8.5" x 11") sheet.

N

Meters: 0, 25, 50, 100, 150

Feet: 0, 50, 100, 200, 300

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 16N WGS84

MAP LEGEND

Area of Interest (AOI)

Area of Interest (AOI)

Soils

Soil Map Unit Polygons

Soil Map Unit Lines

Soil Map Unit Points

Special Point Features

Blowout

Borrow Pit

Clay Spot

Closed Depression

Gravel Pit

Gravelly Spot

Landfill

Lava Flow

Marsh or swamp

Mine or Quarry

Miscellaneous Water

Perennial Water

Rock Outcrop

Saline Spot

Sandy Spot

Severely Eroded Spot

Sinkhole

Slide or Slip

Sodic Spot

Spoil Area

Stony Spot

Very Stony Spot

Wet Spot

Other

Special Line Features

Water Features

Streams and Canals

Transportation

Rails

Interstate Highways

US Routes

Major Roads

Local Roads

Background

Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:15,800.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL: http://websoilsurvey.nrcs.usda.gov
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Milwaukee and Waukesha Counties, Wisconsin
Survey Area Data: Version 9, Sep 18, 2014

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Sep 7, 2014—Sep 22, 2014

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Milwaukee and Waukesha Counties, Wisconsin (WI602)			
Map Unit Symbol	**Map Unit Name**	**Acres in AOI**	**Percent of AOI**
HmB	Hochheim loam, 2 to 6 percent slopes	0.3	2.3%
HmB2	Hochheim loam, 2 to 6 percent slopes, eroded	0.2	1.7%
HmC2	Hochheim loam, 6 to 12 percent slopes, eroded	4.0	31.6%
HmD2	Hochheim loam, 12 to 20 percent slopes, eroded	2.6	20.8%
HoD3	Hochheim soils, 12 to 20 percent slopes, severely eroded	2.0	16.1%
HtA	Houghton muck, 0 to 2 percent slopes	0.6	4.5%
PrA	Pistakee silt loam, 1 to 3 percent slopes	2.1	16.8%
W	Water	0.8	6.2%
Totals for Area of Interest		**12.6**	**100.0%**

MAP SCALE 1" = 500'

250 0 500 1000 FEET

METERS

NFIP

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 0213G

FIRM

FLOOD INSURANCE RATE MAP

WAUKESHA COUNTY, WISCONSIN AND INCORPORATED AREAS

PANEL 213 OF 500

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
PEWAUKEE, CITY OF	550192	0213	G
WAUKESHA, CITY OF	550491	0213	G

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

U.S. DEPARTMENT OF HOMELAND SECURITY

MAP NUMBER
55133C0213G

MAP REVISED
NOVEMBER 5, 2014

Federal Emergency Management Agency

This is an official copy of a portion of the above referenced flood map. It was extracted using F-MIT On-Line. This map does not reflect changes or amendments which may have been made subsequent to the date on the title block. For the latest product information about National Flood Insurance Program flood maps check the FEMA Flood Map Store at www.msc.fema.gov

MAP SCALE 1" = 500'

250 0 500 1000

FEET

METERS

PANEL 0211G

NFIP

NATIONAL FLOOD INSURANCE PROGRAM

FIRM

FLOOD INSURANCE RATE MAP

WAUKESHA COUNTY, WISCONSIN

AND INCORPORATED AREAS

PANEL 211 OF 500

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
PEWAUKEE, CITY OF	550192	0211	G
WAUKESHA, CITY OF	550491	0211	G

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER
55133C0211G

MAP REVISED
NOVEMBER 5, 2014

Federal Emergency Management Agency

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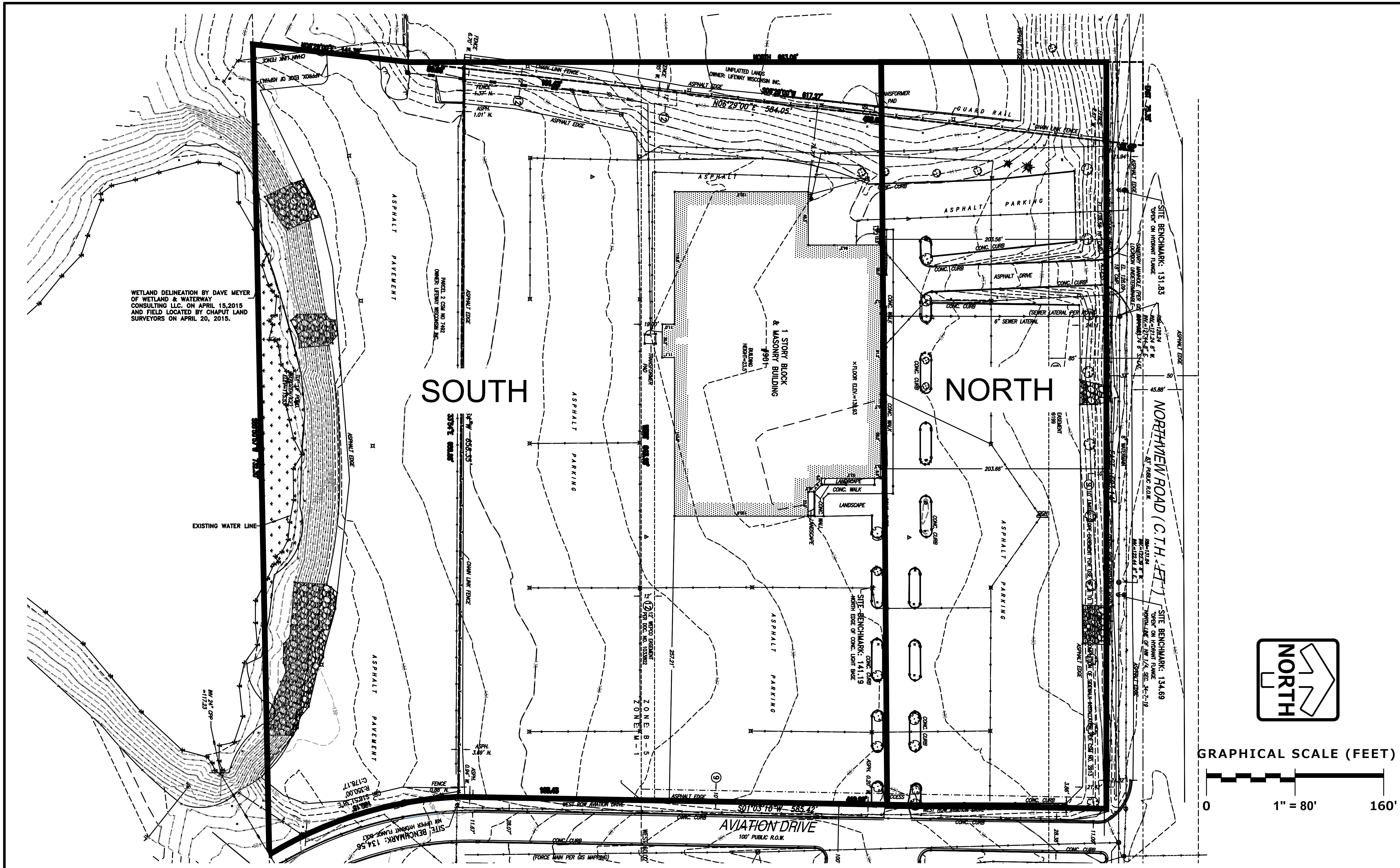
Appendix 2

Pre-Development Conditions Information



EXISTING HYDROLOGY EXHIBIT

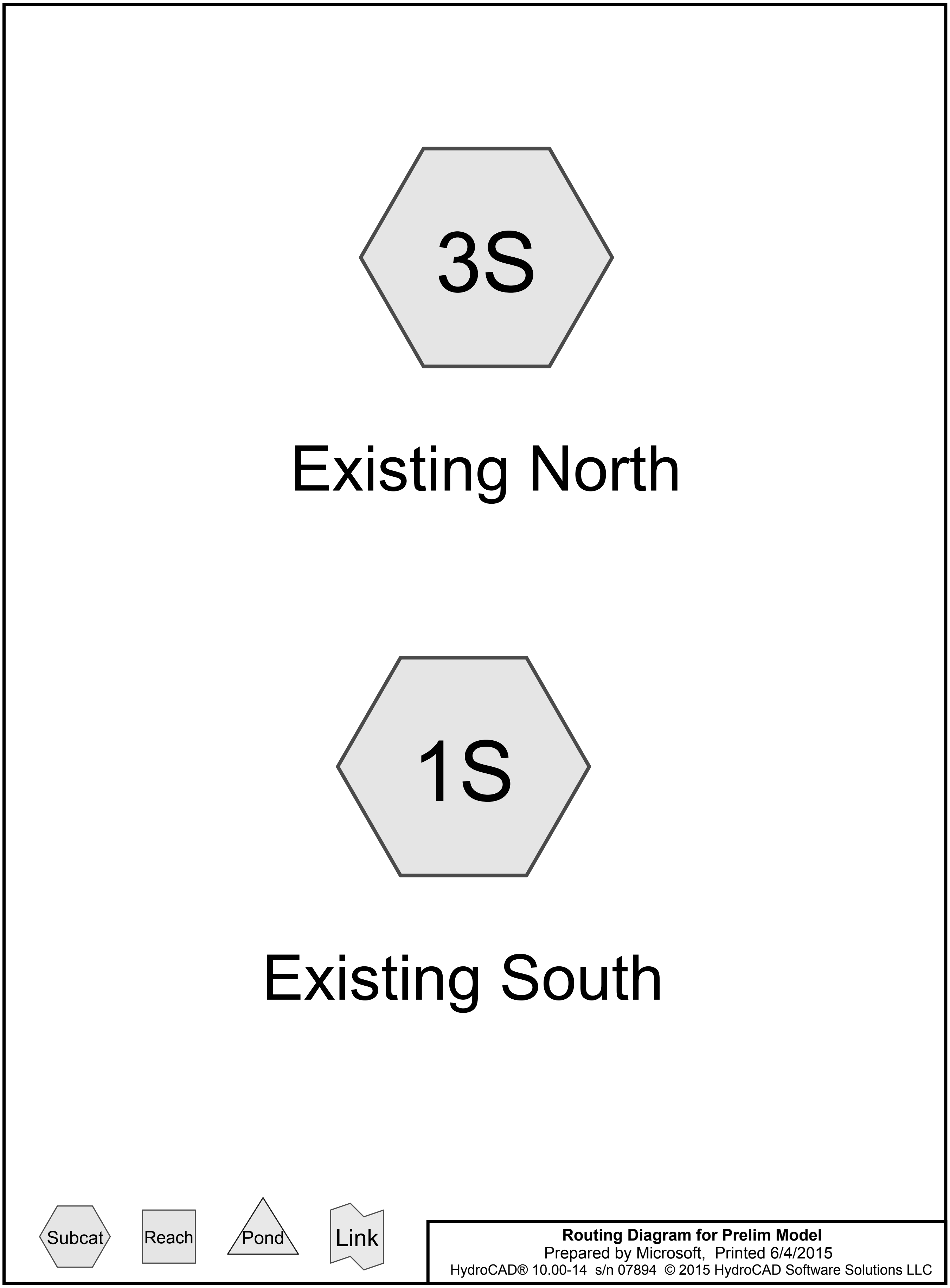
06/08/15

PINNACLE ENGINEERING GROUP

5850 W. BLUEMOUND ROAD | SUITE 210 | BROOKFIELD, WI 53005 | WWW.PINNACLE-ENGR.COM |

PLAN | DESIGN | DELIVER

PEG JOB#426.00



Routing Diagram for Prelim Model
Prepared by Microsoft, Printed 6/4/2015
HydroCAD® 10.00-14 s/n 07894 © 2015 HydroCAD Software Solutions LLC

Area Listing (selected nodes)

Area (acres)	CN	Description (subcatchment-numbers)
1.900	74	>75% Grass cover, Good, HSG C (1S, 3S)
8.460	98	Paved parking, HSG C (1S, 3S)
1.140	98	Roofs, HSG C (1S)
11.500	**94**	**TOTAL AREA**

Soil Listing (selected nodes)

Area (acres)	Soil Group	Subcatchment Numbers
0.000	HSG A	
0.000	HSG B	
11.500	HSG C	1S, 3S
0.000	HSG D	
0.000	Other	
11.500		**TOTAL AREA**

Ground Covers (selected nodes)

HSG-A (acres)	HSG-B (acres)	HSG-C (acres)	HSG-D (acres)	Other (acres)	Total (acres)	Ground Cover	Subcatchment Numbers
0.000	0.000	1.900	0.000	0.000	1.900	>75% Grass cover, Good	1S, 3S
0.000	0.000	8.460	0.000	0.000	8.460	Paved parking	1S, 3S
0.000	0.000	1.140	0.000	0.000	1.140	Roofs	1S
0.000	**0.000**	**11.500**	**0.000**	**0.000**	**11.500**	**TOTAL AREA**	

Time span=5.00-20.00 hrs, dt=0.05 hrs, 301 points
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment1S: Existing South Runoff Area=7.670 ac 83.44% Impervious Runoff Depth>1.93"
Tc=6.0 min CN=94 Runoff=25.65 cfs 1.235 af

Subcatchment3S: Existing North Runoff Area=3.830 ac 83.55% Impervious Runoff Depth>1.93"
Tc=6.0 min CN=94 Runoff=12.81 cfs 0.617 af

Total Runoff Area = 11.500 ac Runoff Volume = 1.852 af Average Runoff Depth = 1.93"
16.52% Pervious = 1.900 ac 83.48% Impervious = 9.600 ac

Summary for Subcatchment 1S: Existing South

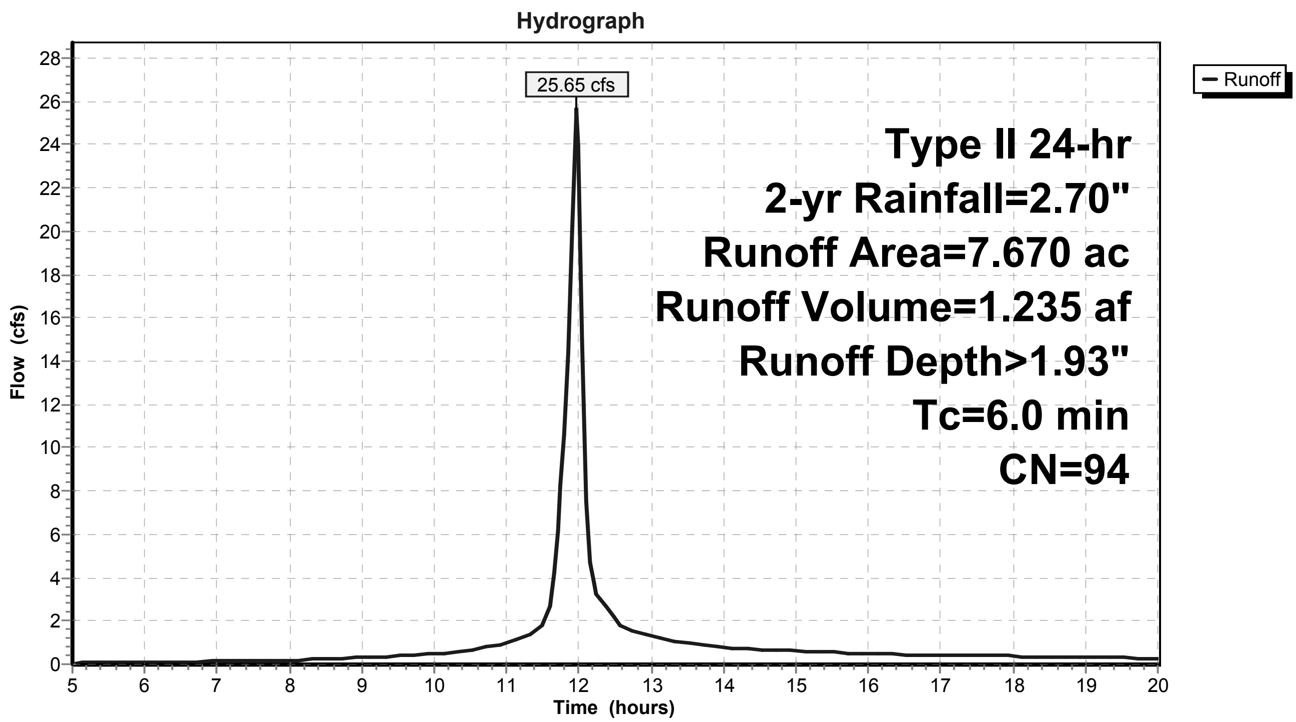
Runoff = 25.65 cfs @ 11.96 hrs, Volume= 1.235 af, Depth> 1.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Type II 24-hr 2-yr Rainfall=2.70"

Area (ac)	CN	Description
1.140	98	Roofs, HSG C
5.260	98	Paved parking, HSG C
1.270	74	>75% Grass cover, Good, HSG C
7.670	94	Weighted Average
1.270		16.56% Pervious Area
6.400		83.44% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 1S: Existing South



Summary for Subcatchment 3S: Existing North

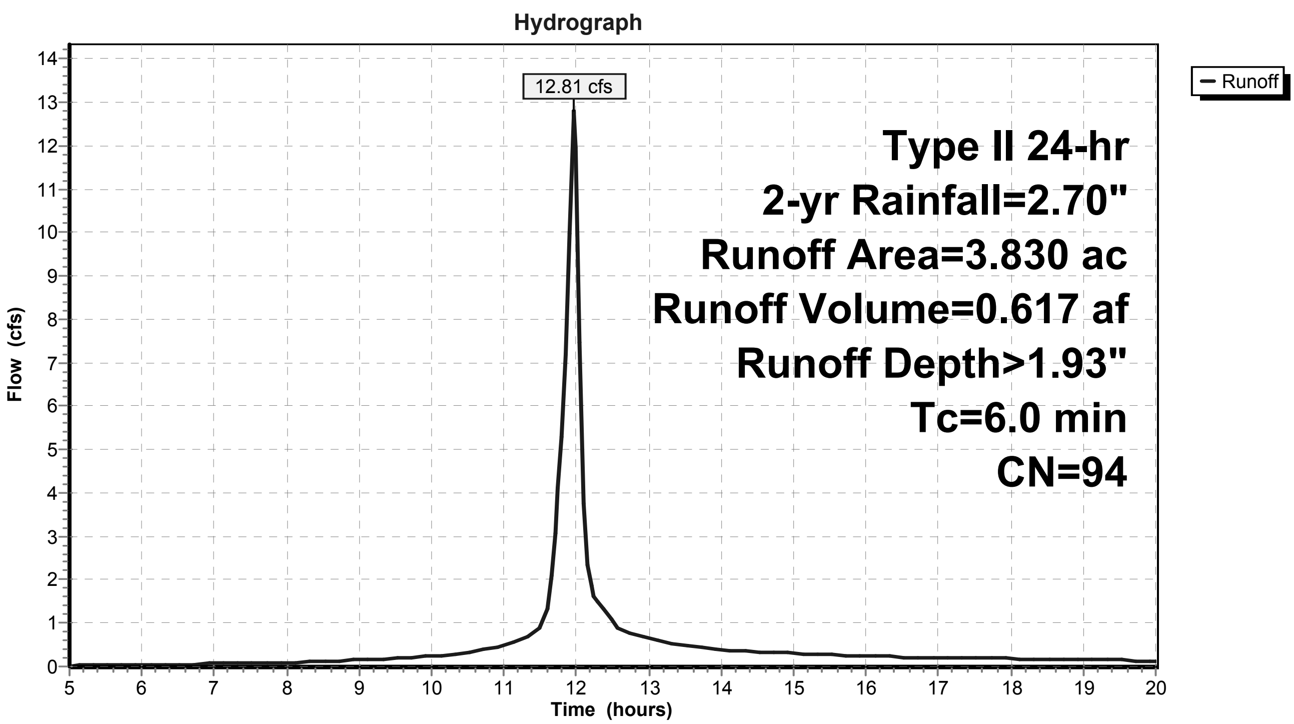
Runoff = 12.81 cfs @ 11.96 hrs, Volume= 0.617 af, Depth> 1.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Type II 24-hr 2-yr Rainfall=2.70"

Area (ac)	CN	Description
3.200	98	Paved parking, HSG C
0.630	74	>75% Grass cover, Good, HSG C
3.830	94	Weighted Average
0.630		16.45% Pervious Area
3.200		83.55% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 3S: Existing North



Time span=5.00-20.00 hrs, dt=0.05 hrs, 301 points
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment1S: Existing South Runoff Area=7.670 ac 83.44% Impervious Runoff Depth>3.11"
Tc=6.0 min CN=94 Runoff=40.12 cfs 1.991 af

Subcatchment3S: Existing North Runoff Area=3.830 ac 83.55% Impervious Runoff Depth>3.11"
Tc=6.0 min CN=94 Runoff=20.04 cfs 0.994 af

Total Runoff Area = 11.500 ac Runoff Volume = 2.985 af Average Runoff Depth = 3.11"
16.52% Pervious = 1.900 ac 83.48% Impervious = 9.600 ac

Summary for Subcatchment 1S: Existing South

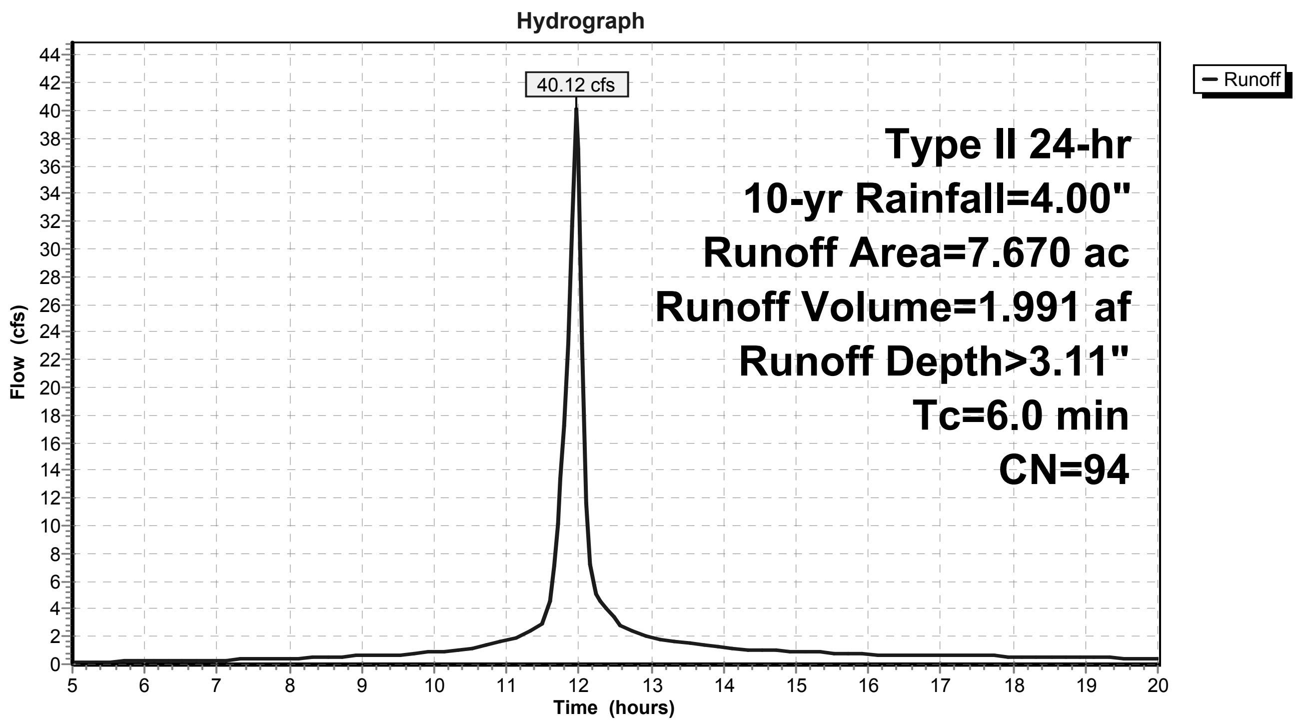
Runoff = 40.12 cfs @ 11.96 hrs, Volume= 1.991 af, Depth> 3.11"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Type II 24-hr 10-yr Rainfall=4.00"

Area (ac)	CN	Description
1.140	98	Roofs, HSG C
5.260	98	Paved parking, HSG C
1.270	74	>75% Grass cover, Good, HSG C
7.670	94	Weighted Average
1.270		16.56% Pervious Area
6.400		83.44% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 1S: Existing South



Summary for Subcatchment 3S: Existing North

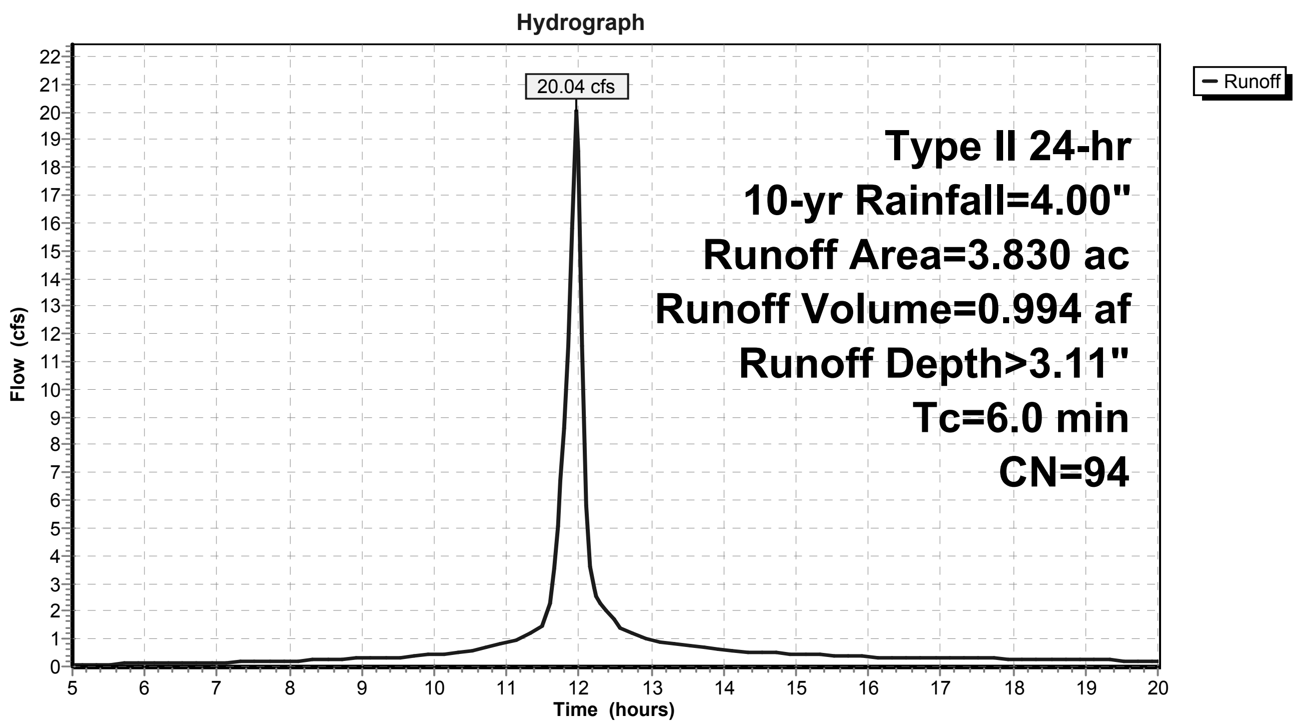
Runoff = 20.04 cfs @ 11.96 hrs, Volume= 0.994 af, Depth> 3.11"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Type II 24-hr 10-yr Rainfall=4.00"

Area (ac)	CN	Description
3.200	98	Paved parking, HSG C
0.630	74	>75% Grass cover, Good, HSG C
3.830	94	Weighted Average
0.630		16.45% Pervious Area
3.200		83.55% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 3S: Existing North



Time span=5.00-20.00 hrs, dt=0.05 hrs, 301 points
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment1S: Existing South Runoff Area=7.670 ac 83.44% Impervious Runoff Depth>4.58"
Tc=6.0 min CN=94 Runoff=57.71 cfs 2.925 af

Subcatchment3S: Existing North Runoff Area=3.830 ac 83.55% Impervious Runoff Depth>4.58"
Tc=6.0 min CN=94 Runoff=28.82 cfs 1.461 af

Total Runoff Area = 11.500 ac Runoff Volume = 4.385 af Average Runoff Depth = 4.58"
16.52% Pervious = 1.900 ac 83.48% Impervious = 9.600 ac

Summary for Subcatchment 1S: Existing South

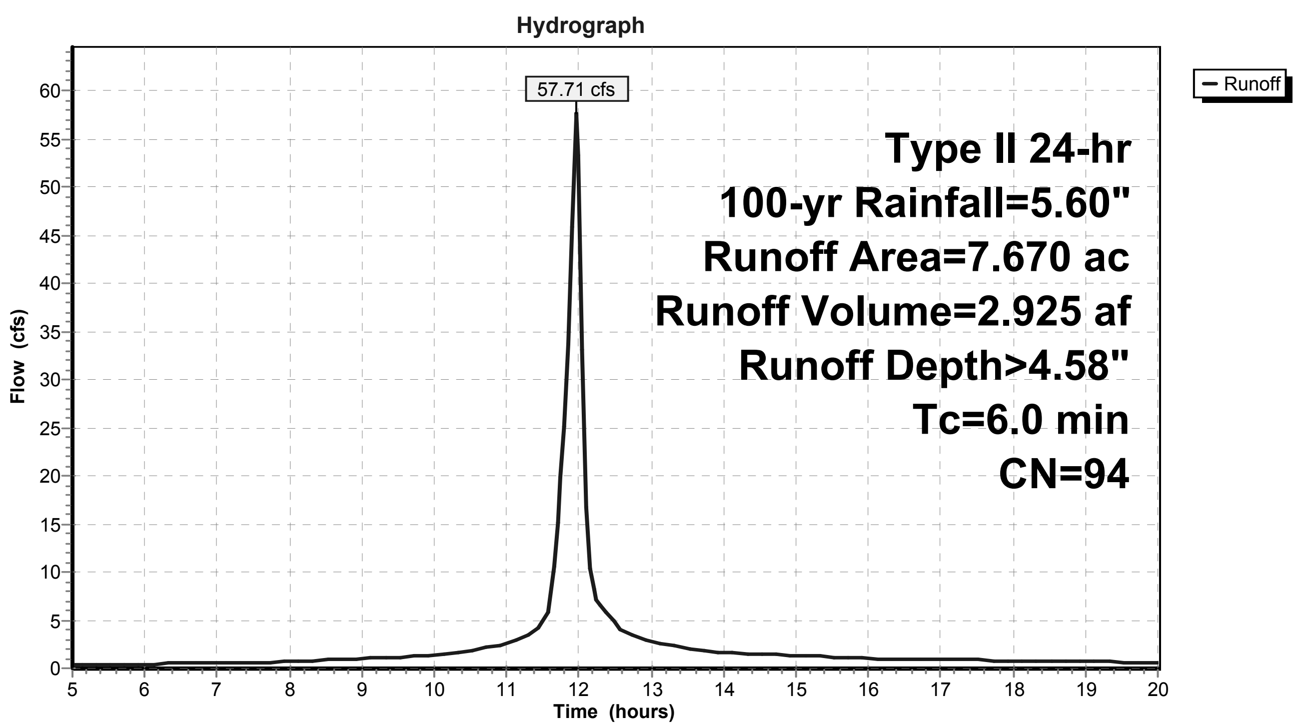
Runoff = 57.71 cfs @ 11.96 hrs, Volume= 2.925 af, Depth> 4.58"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Type II 24-hr 100-yr Rainfall=5.60"

Area (ac)	CN	Description
1.140	98	Roofs, HSG C
5.260	98	Paved parking, HSG C
1.270	74	>75% Grass cover, Good, HSG C
7.670	94	Weighted Average
1.270		16.56% Pervious Area
6.400		83.44% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 1S: Existing South



Summary for Subcatchment 3S: Existing North

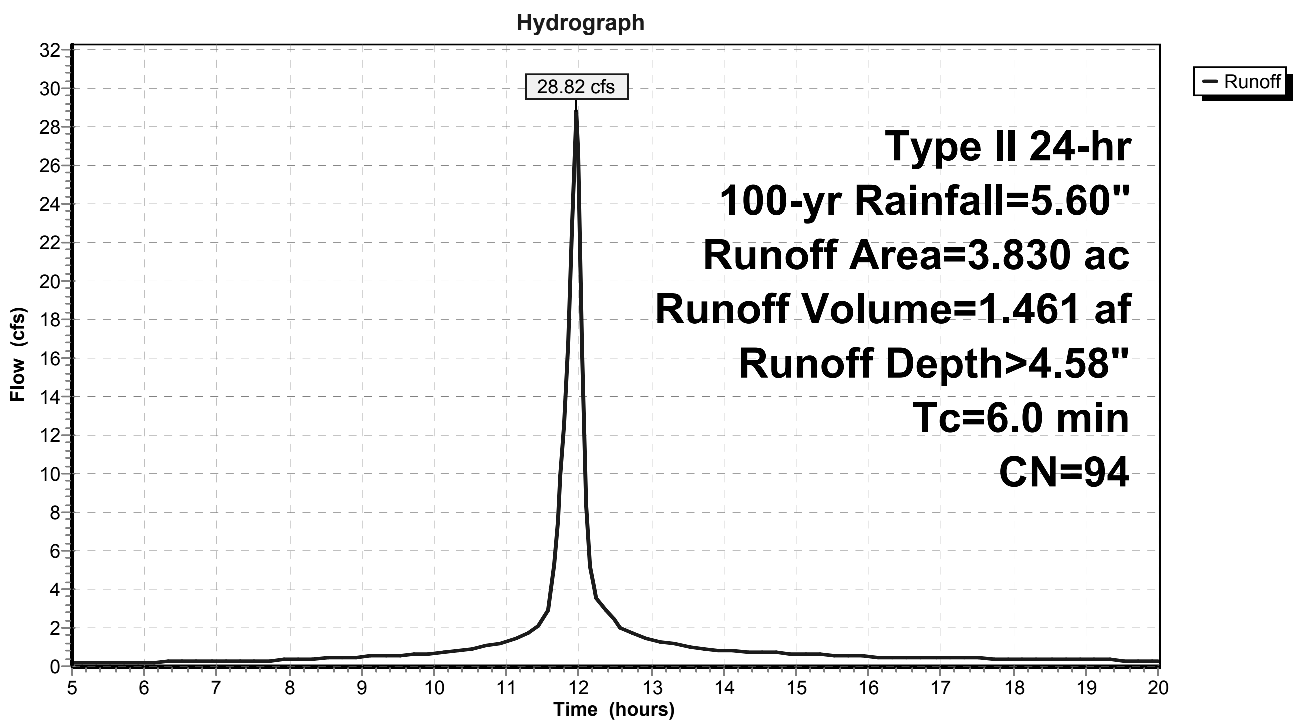
Runoff = 28.82 cfs @ 11.96 hrs, Volume= 1.461 af, Depth> 4.58"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Type II 24-hr 100-yr Rainfall=5.60"

Area (ac)	CN	Description
3.200	98	Paved parking, HSG C
0.630	74	>75% Grass cover, Good, HSG C
3.830	94	Weighted Average
0.630		16.45% Pervious Area
3.200		83.55% Impervious Area

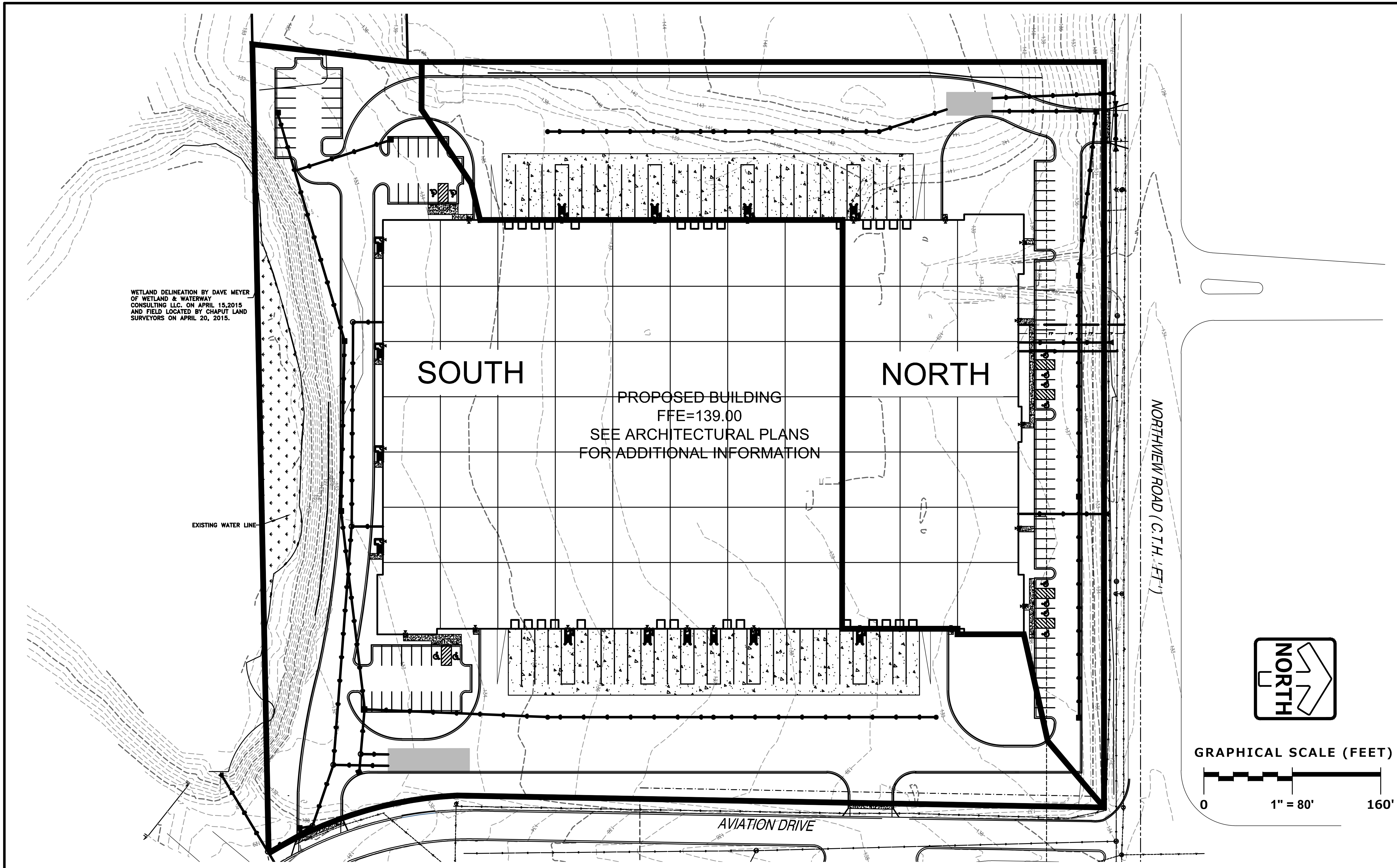
Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 3S: Existing North



Appendix 3

Post Development Conditions Infornmation



PROPOSED HYDROLOGY EXHIBIT

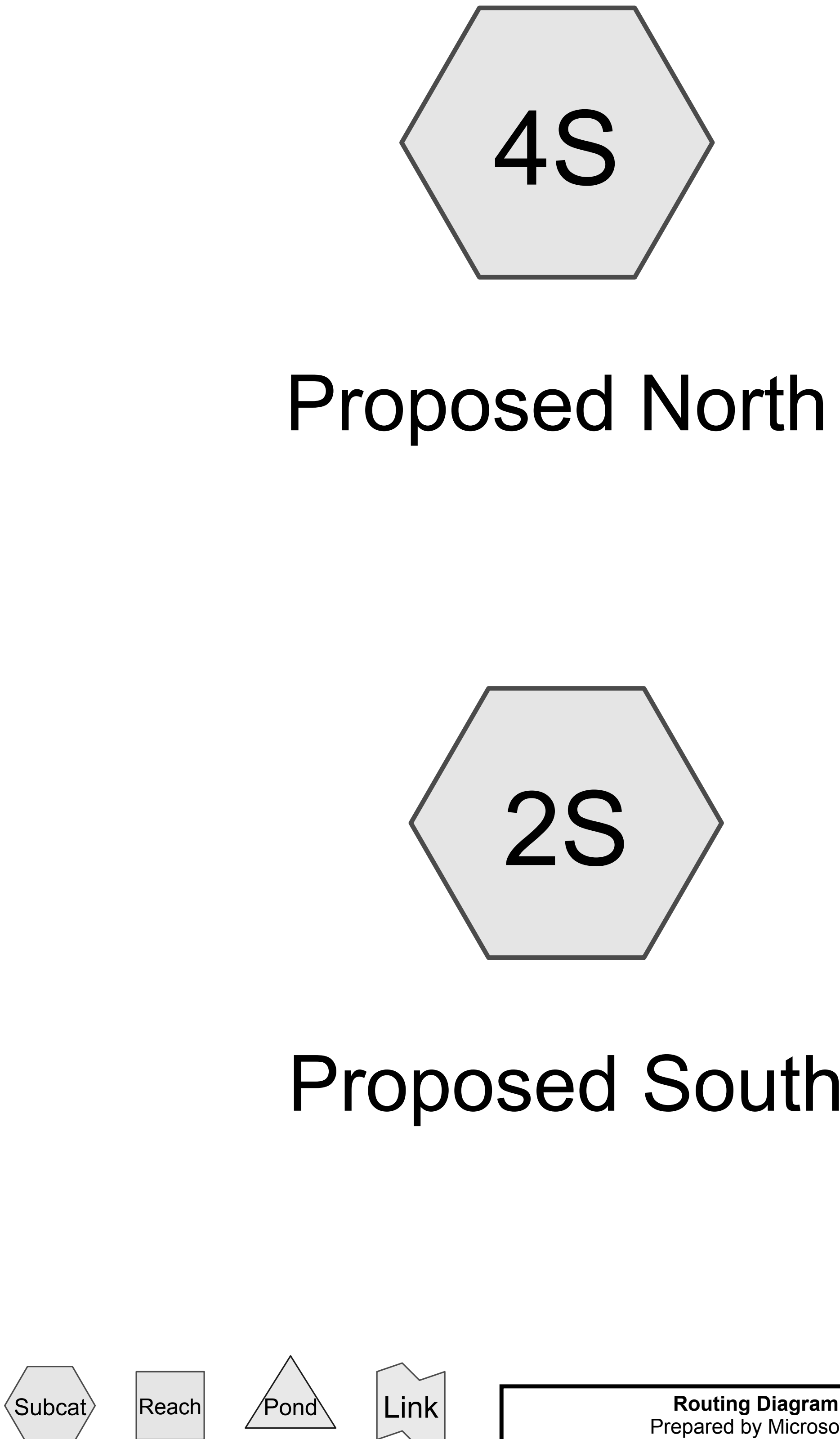
06/08/15

PINNACLE ENGINEERING GROUP

5850 W. BLUEMOUND ROAD | SUITE 210 | BROOKFIELD, WI 53005 | WWW.PINNACLE-ENGR.COM |

PLAN | DESIGN | DELIVER

PEG JOB#426.00



Routing Diagram for Prelim Model
Prepared by Microsoft, Printed 6/4/2015
HydroCAD® 10.00-14 s/n 07894 © 2015 HydroCAD Software Solutions LLC

Area Listing (selected nodes)

Area (acres)	CN	Description (subcatchment-numbers)
2.270	74	>75% Grass cover, Good, HSG C (2S, 4S)
4.350	98	Paved parking, HSG C (2S, 4S)
4.880	98	Roofs, HSG C (2S, 4S)
11.500	**93**	**TOTAL AREA**

Soil Listing (selected nodes)

Area (acres)	Soil Group	Subcatchment Numbers
0.000	HSG A	
0.000	HSG B	
11.500	HSG C	2S, 4S
0.000	HSG D	
0.000	Other	
11.500		**TOTAL AREA**

Ground Covers (selected nodes)

HSG-A (acres)	HSG-B (acres)	HSG-C (acres)	HSG-D (acres)	Other (acres)	Total (acres)	Ground Cover	Subcatchment Numbers
0.000	0.000	2.270	0.000	0.000	2.270	>75% Grass cover, Good	2S, 4S
0.000	0.000	4.350	0.000	0.000	4.350	Paved parking	2S, 4S
0.000	0.000	4.880	0.000	0.000	4.880	Roofs	2S, 4S
0.000	**0.000**	**11.500**	**0.000**	**0.000**	**11.500**	**TOTAL AREA**	

Time span=5.00-20.00 hrs, dt=0.05 hrs, 301 points
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment2S: Proposed South Runoff Area=7.800 ac 76.67% Impervious Runoff Depth>1.76"
Tc=6.0 min CN=92 Runoff=24.34 cfs 1.142 af

Subcatchment4S: Proposed North Runoff Area=3.700 ac 87.84% Impervious Runoff Depth>2.02"
Tc=6.0 min CN=95 Runoff=12.76 cfs 0.624 af

Total Runoff Area = 11.500 ac Runoff Volume = 1.765 af Average Runoff Depth = 1.84"
19.74% Pervious = 2.270 ac 80.26% Impervious = 9.230 ac

Summary for Subcatchment 2S: Proposed South

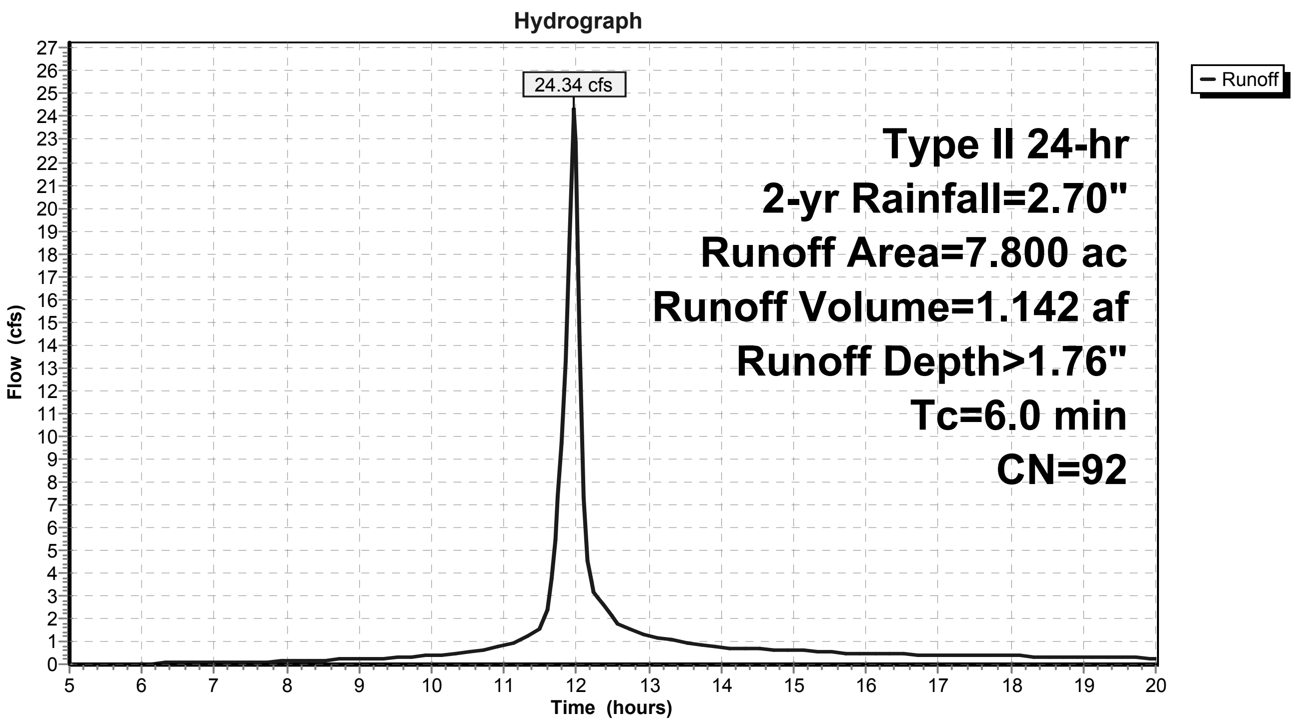
Runoff = 24.34 cfs @ 11.97 hrs, Volume= 1.142 af, Depth> 1.76"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Type II 24-hr 2-yr Rainfall=2.70"

Area (ac)	CN	Description
2.470	98	Paved parking, HSG C
3.510	98	Roofs, HSG C
1.820	74	>75% Grass cover, Good, HSG C
7.800	92	Weighted Average
1.820		23.33% Pervious Area
5.980		76.67% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 2S: Proposed South



Summary for Subcatchment 4S: Proposed North

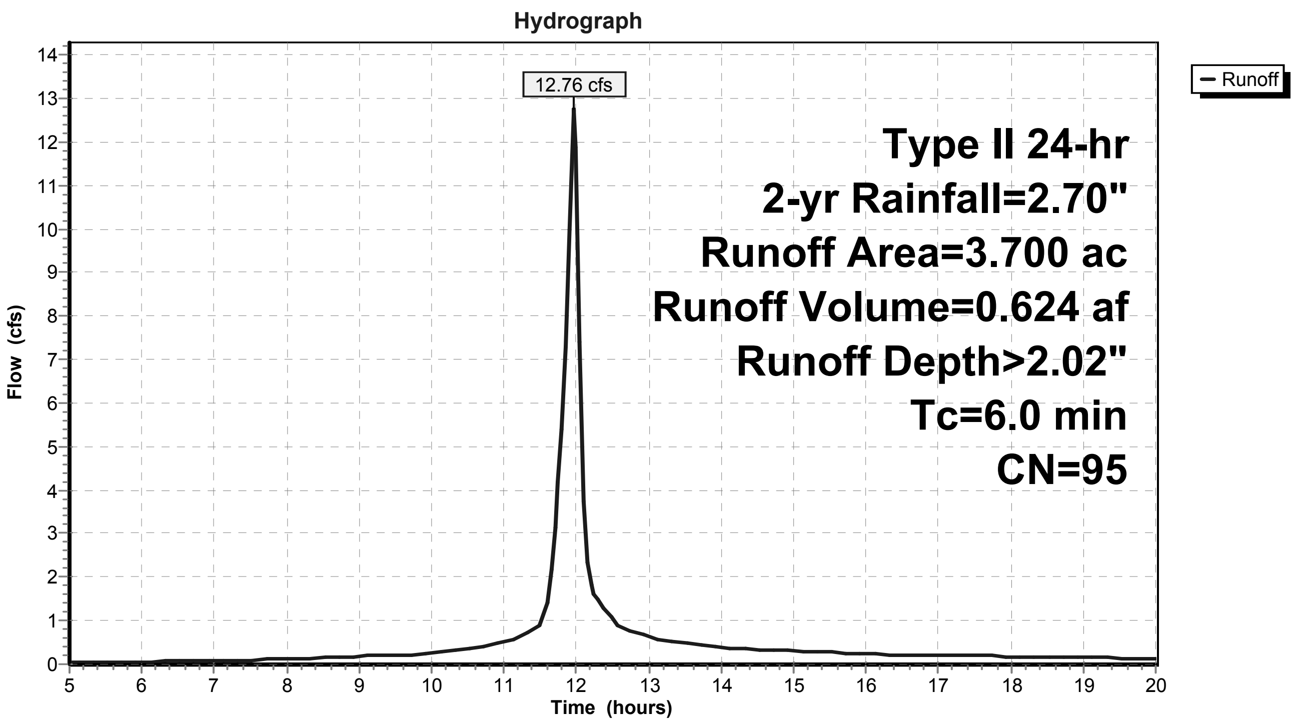
Runoff = 12.76 cfs @ 11.96 hrs, Volume= 0.624 af, Depth> 2.02"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Type II 24-hr 2-yr Rainfall=2.70"

Area (ac)	CN	Description
1.880	98	Paved parking, HSG C
1.370	98	Roofs, HSG C
0.450	74	>75% Grass cover, Good, HSG C
3.700	95	Weighted Average
0.450		12.16% Pervious Area
3.250		87.84% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 4S: Proposed North



Time span=5.00-20.00 hrs, dt=0.05 hrs, 301 points
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment2S: Proposed South Runoff Area=7.800 ac 76.67% Impervious Runoff Depth>2.92"
Tc=6.0 min CN=92 Runoff=39.20 cfs 1.900 af

Subcatchment4S: Proposed North Runoff Area=3.700 ac 87.84% Impervious Runoff Depth>3.21"
Tc=6.0 min CN=95 Runoff=19.69 cfs 0.990 af

Total Runoff Area = 11.500 ac Runoff Volume = 2.890 af Average Runoff Depth = 3.02"
19.74% Pervious = 2.270 ac 80.26% Impervious = 9.230 ac

Summary for Subcatchment 2S: Proposed South

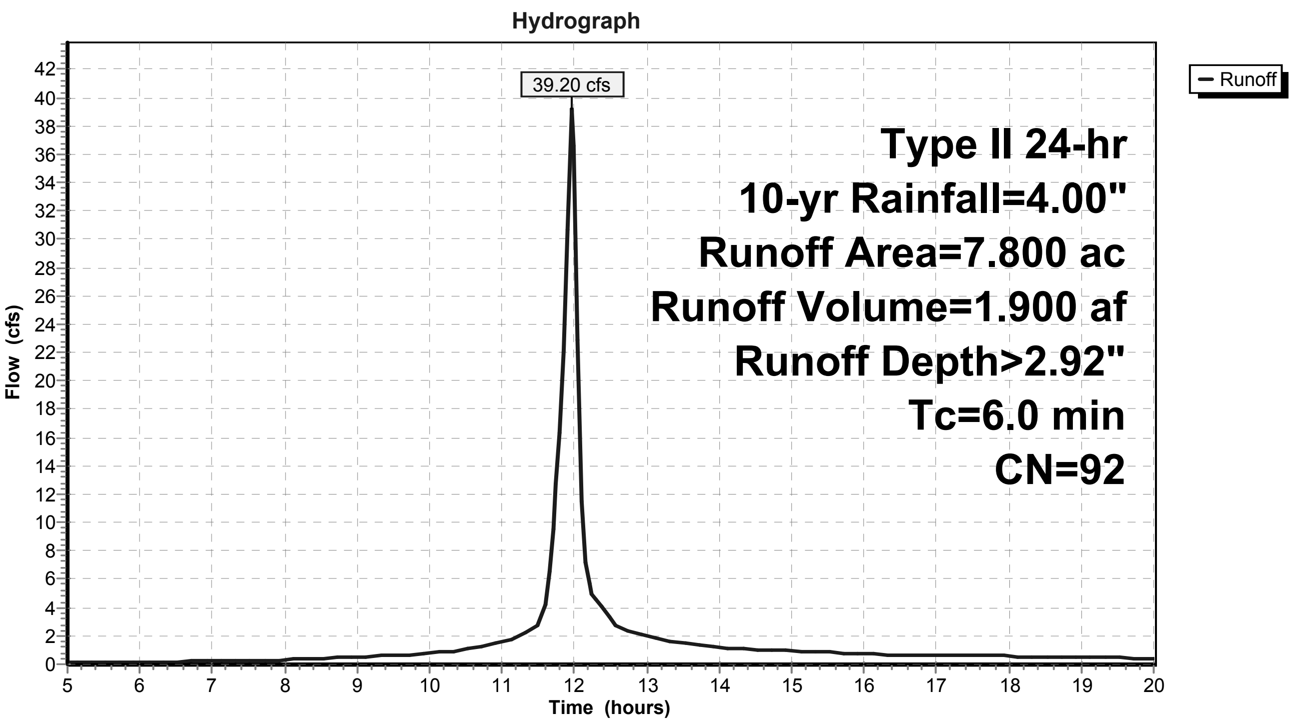
Runoff = 39.20 cfs @ 11.96 hrs, Volume= 1.900 af, Depth> 2.92"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Type II 24-hr 10-yr Rainfall=4.00"

Area (ac)	CN	Description
2.470	98	Paved parking, HSG C
3.510	98	Roofs, HSG C
1.820	74	>75% Grass cover, Good, HSG C
7.800	92	Weighted Average
1.820		23.33% Pervious Area
5.980		76.67% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 2S: Proposed South



Summary for Subcatchment 4S: Proposed North

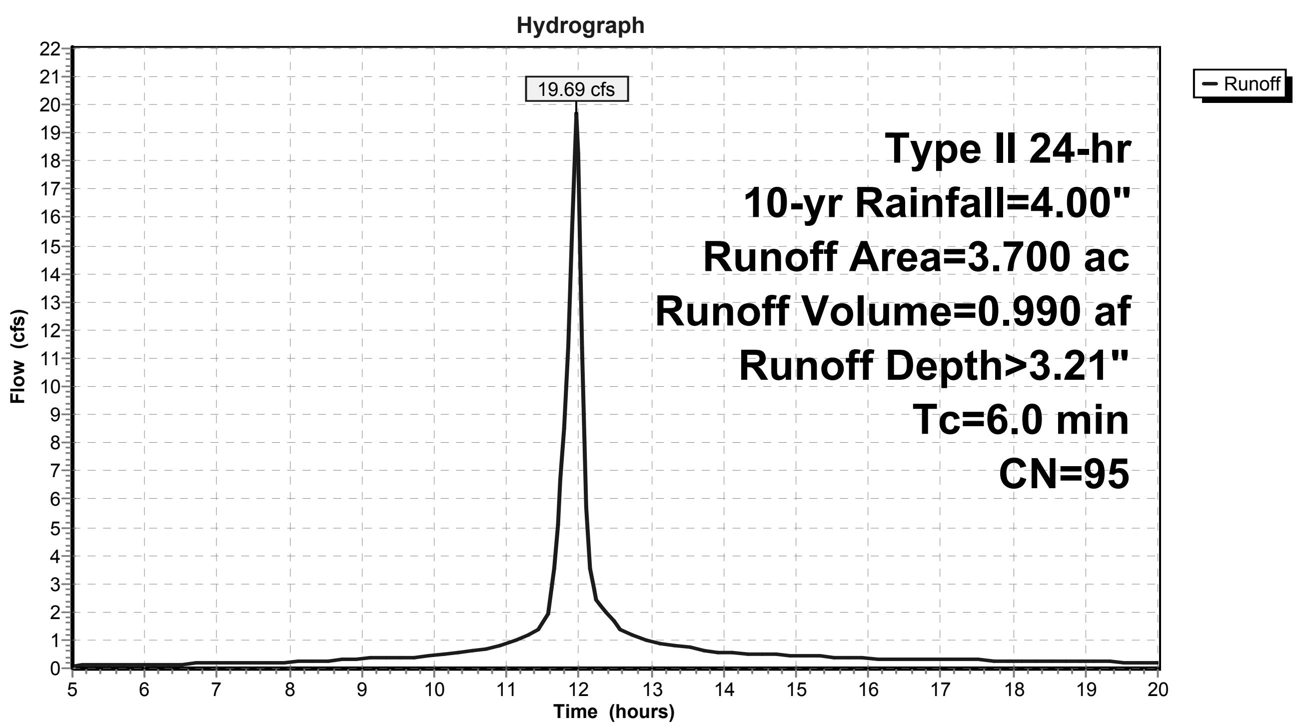
Runoff = 19.69 cfs @ 11.96 hrs, Volume= 0.990 af, Depth> 3.21"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Type II 24-hr 10-yr Rainfall=4.00"

Area (ac)	CN	Description
1.880	98	Paved parking, HSG C
1.370	98	Roofs, HSG C
0.450	74	>75% Grass cover, Good, HSG C
3.700	95	Weighted Average
0.450		12.16% Pervious Area
3.250		87.84% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 4S: Proposed North



Time span=5.00-20.00 hrs, dt=0.05 hrs, 301 points
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment2S: Proposed South Runoff Area=7.800 ac 76.67% Impervious Runoff Depth>4.38"
Tc=6.0 min CN=92 Runoff=57.28 cfs 2.848 af

Subcatchment4S: Proposed North Runoff Area=3.700 ac 87.84% Impervious Runoff Depth>4.67"
Tc=6.0 min CN=95 Runoff=28.12 cfs 1.440 af

Total Runoff Area = 11.500 ac Runoff Volume = 4.287 af Average Runoff Depth = 4.47"
19.74% Pervious = 2.270 ac 80.26% Impervious = 9.230 ac

Summary for Subcatchment 2S: Proposed South

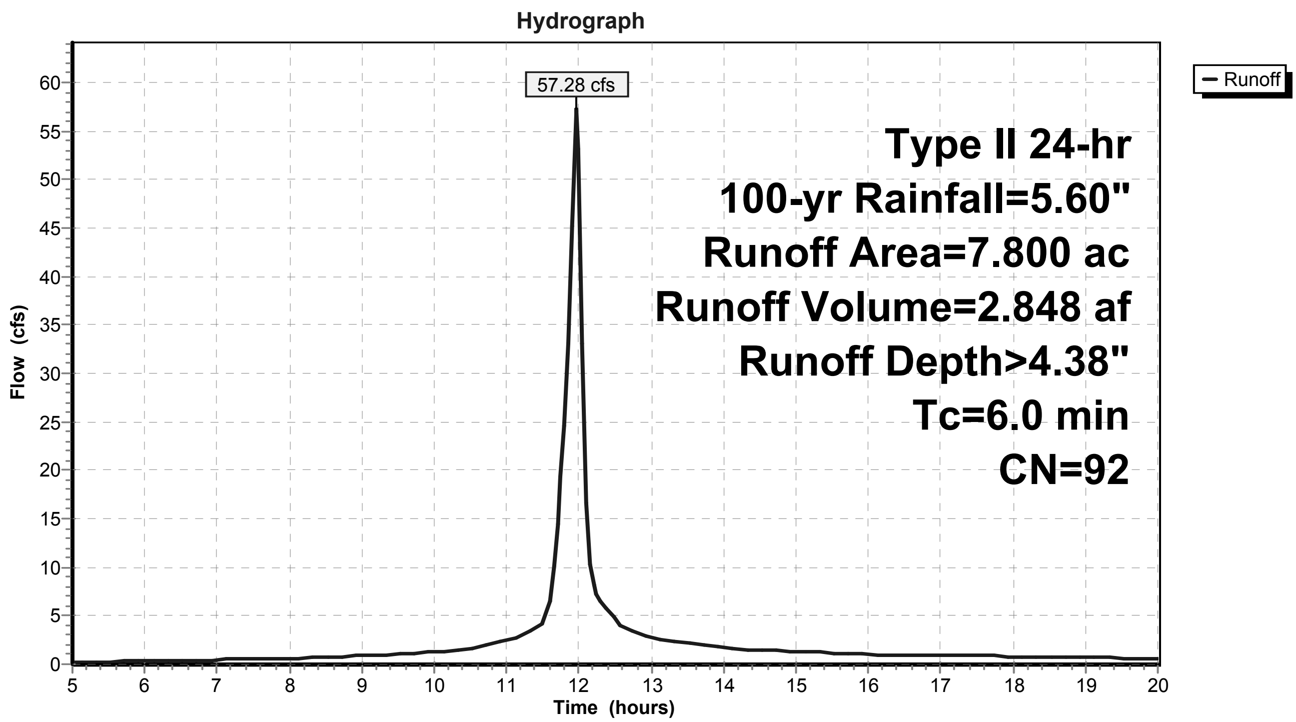
Runoff = 57.28 cfs @ 11.96 hrs, Volume= 2.848 af, Depth> 4.38"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Type II 24-hr 100-yr Rainfall=5.60"

Area (ac)	CN	Description
2.470	98	Paved parking, HSG C
3.510	98	Roofs, HSG C
1.820	74	>75% Grass cover, Good, HSG C
7.800	92	Weighted Average
1.820		23.33% Pervious Area
5.980		76.67% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 2S: Proposed South



Summary for Subcatchment 4S: Proposed North

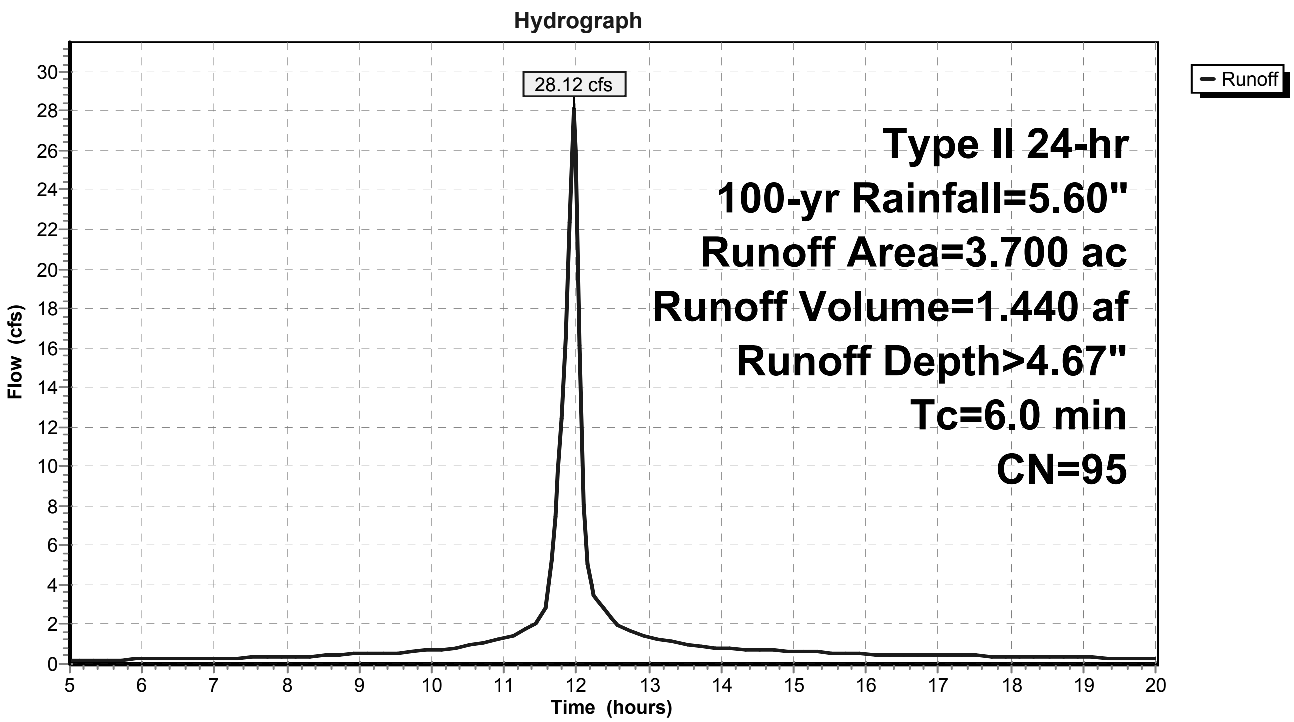
Runoff = 28.12 cfs @ 11.96 hrs, Volume= 1.440 af, Depth> 4.67"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
Type II 24-hr 100-yr Rainfall=5.60"

Area (ac)	CN	Description
1.880	98	Paved parking, HSG C
1.370	98	Roofs, HSG C
0.450	74	>75% Grass cover, Good, HSG C
3.700	95	Weighted Average
0.450		12.16% Pervious Area
3.250		87.84% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment 4S: Proposed North



Appendix 4

Post Development Conditions (Water Quality)

Data file name: Z:\Projects\2014\426.00-WI\DESIGN\SWMP\SLAMM\2015-06-03 BLH.mdb
WinSLAMM Version 10.1.6
Rain file name: C:\WinSLAMM Files\Rain Files\WI Milwaukee 69.RAN
Particulate Solids Concentration file name: C:\WinSLAMM Files\v10.1 WI_AVG01.pscx
Runoff Coefficient file name: C:\WinSLAMM Files\WI_SL06 Dec06.rsvx
Residential Street Delivery file name: C:\WinSLAMM Files\WI_Res and Other Urban Dec06.std
Institutional Street Delivery file name: C:\WinSLAMM Files\WI_Com Inst Indust Dec06.std
Commercial Street Delivery file name: C:\WinSLAMM Files\WI_Com Inst Indust Dec06.std
Industrial Street Delivery file name: C:\WinSLAMM Files\WI_Com Inst Indust Dec06.std
Other Urban Street Delivery file name: C:\WinSLAMM Files\WI_Res and Other Urban Dec06.std
Freeway Street Delivery file name: C:\WinSLAMM Files\Freeway Dec06.std
Apply Street Delivery Files to Adjust the After Event Load Street Dirt Mass Balance: False
Pollutant Relative Concentration file name: C:\WinSLAMM Files\WI_GEO03.ppdx
Cost Data file name:
Seed for random number generator: -42
Study period starting date: 03/28/69   Study period ending date: 12/06/69
Date: 06-08-2015   Time: 12:59:04
Site information:

LU# 1 - Industrial: Grass North   Total area (ac): 0.450
     45 - Large Landscaped Areas 1: 0.450 ac.   Normal Clayey   Low Density

LU# 2 - Industrial: Grass South   Total area (ac): 1.820
     45 - Large Landscaped Areas 1: 1.820 ac.   Normal Clayey   Low Density

LU# 3 - Industrial: Pavement North   Total area (ac): 1.880
     13 - Paved Parking 1: 1.880 ac.   Connected   Connected

LU# 4 - Industrial: Pavement South   Total area (ac): 2.470
     13 - Paved Parking 1: 2.470 ac.   Connected   Connected

Control Practice 1: Wet Detention Pond CP# 1 (DS) - North Underground Chamber
Particle Size Distribution file name: Not needed - calculated by program
Initial stage elevation (ft):   3.5
Peak to Average Flow Ratio: 3.8
Maximum flow allowed into pond (cfs): No maximum value entered
Outlet Characteristics:
Outlet type: Orifice 1
1. Orifice diameter (ft):   1
2. Number of orifices: 1
3. Invert elevation above datum (ft):   3.5
Outlet type: Broad Crested Weir

1. Weir crest length (ft): 16
2. Weir crest width (ft): 0.5
3. Height of weir opening (cfs): 0
4. Height from datum to bottom of weir opening: 5

Pond stage and surface area

Entry Number	Stage (ft)	Pond Area (acres)	Natural Seepage (in/hr)	Other Outflow (cfs)
0	0.00	0.0000	0.00	0.00
1	0.01	0.0069	0.00	0.00
2	1.00	0.0069	0.00	0.00
3	2.00	0.0069	0.00	0.00
4	3.00	0.0069	0.00	0.00
5	4.00	0.0069	0.00	0.00
6	5.00	0.0069	0.00	0.00
7	6.00	0.0069	0.00	0.00
8	7.00	0.0069	0.00	0.00

Control Practice 2: Wet Detention Pond CP# 2 (DS) - South Chamber

Particle Size Distribution file name: Not needed - calculated by program
Initial stage elevation (ft): 3.5
Peak to Average Flow Ratio: 3.8
Maximum flow allowed into pond (cfs): No maximum value entered
Outlet Characteristics:

Outlet type: Orifice 1

1. Orifice diameter (ft): 1
2. Number of orifices: 1
3. Invert elevation above datum (ft): 3.5

Outlet type: Broad Crested Weir

1. Weir crest length (ft): 16
2. Weir crest width (ft): 5
3. Height of weir opening (cfs): 0
4. Height from datum to bottom of weir opening: 6

Pond stage and surface area

Entry Number	Stage (ft)	Pond Area (acres)	Natural Seepage (in/hr)	Other Outflow (cfs)
0	0.00	0.0000	0.00	0.00
1	0.01	0.0136	0.00	0.00
2	1.00	0.0136	0.00	0.00
3	2.00	0.0136	0.00	0.00
4	3.00	0.0136	0.00	0.00
5	4.00	0.0136	0.00	0.00
6	5.00	0.0136	0.00	0.00
7	6.00	0.0136	0.00	0.00
8	7.00	0.0136	0.00	0.00

Control Practice 3: Other Device CP# 1 (DS) - DS Other Device # 2

Fraction of drainage area served by device (ac) = 1.00
Concentration reduction fraction = 1.00
Runoff volume reduction fraction = 0

Control Practice 4: Other Device CP# 2 (DS) - DS Other Device # 3
Fraction of drainage area served by device (ac) = 1.00
Concentration reduction fraction = 1.00
Runoff volume reduction fraction = 0

SLAMM for Windows Version 10.1.6
(c) Copyright Robert Pitt and John Voorhees 2012
All Rights Reserved

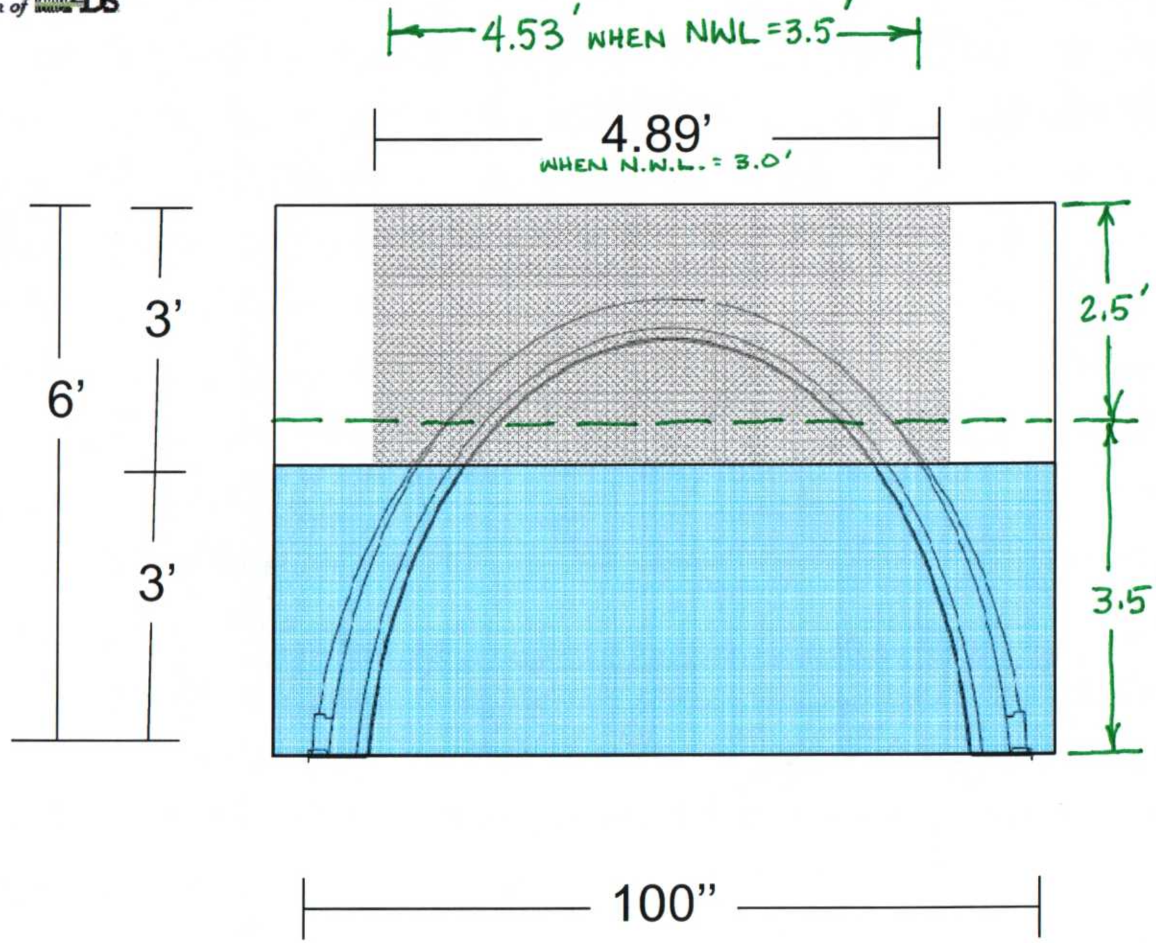
Data file name: Z:\Projects\2014\426.00-WI\DESIGN\SWMP\SLAMM\2015-06-03 BLH.mdb
Data file description:
Rain file name: C:\WinSLAMM Files\Rain Files\WI Milwaukee 69.RAN
Particulate Solids Concentration file name: C:\WinSLAMM Files\v10.1 WI_AVG01.pscx
Runoff Coefficient file name: C:\WinSLAMM Files\WI_SL06 Dec06.rsvx
Residential Street Delivery file name: C:\WinSLAMM Files\WI_Res and Other Urban Dec06.std
Institutional Street Delivery file name: C:\WinSLAMM Files\WI_Com Inst Indust Dec06.std
Commercial Street Delivery file name: C:\WinSLAMM Files\WI_Com Inst Indust Dec06.std
Industrial Street Delivery file name: C:\WinSLAMM Files\WI_Com Inst Indust Dec06.std
Other Urban Street Delivery file name: C:\WinSLAMM Files\WI_Res and Other Urban Dec06.std
Freeway Street Delivery file name: C:\WinSLAMM Files\Freeway Dec06.std
Pollutant Relative Concentration file name: C:\WinSLAMM Files\WI_GEO03.ppdx
Model Run Start Date: 03/28/69 Model Run End Date: 12/06/69
Date of run: 06-08-2015 Time of run: 12:58:38
Total Area Modeled (acres): 6.620
Years in Model Run: 0.67

	Runoff Volume (cu ft)	Percent Runoff Volume Reduction	Particulate Solids Conc. (mg/L)	Particulate Solids Yield (lbs)	Percent Particulate Solids Reduction
Total of all Land Uses without Controls:	351275	-	248.9	5458	-
Outfall Total with Controls:	350725	0.16%	148.3	3246	40.53%
Annualized Total After Outfall Controls:	526810			4876	

StormTech

Detention • Retention • Water Quality

Division of ADS



EQUIVALENT WIDTH WHEN N.W.L. = 3.5' (PER P.V. & ASSOCIATES SUGGESTED METHOD)

CHAMBER VOLUME FULL = 162.62 CF

CHAMBER VOLUME @ 3.5' = 116.94 CF

VOLUME ABOVE N.W.L @ 3.5' = 45.06 CF

÷ 4.025' LENGTH OF CHAMBER

11.349 SF CROSS-SECTIONAL AREA ABOVE N.W.L.

÷ 2.5 HEIGHT OF STORAGE (DIVIDE TO OBTAIN EQUIVALENT WIDTH)

4.53' EQUIVALENT WIDTH

MC-4500 Chamber

StormTech®
Detention • Retention • Water Quality

Subsurface Stormwater Management℠

StormTech™ MC-4500 Chamber

Designed to meet the most stringent industry performance standards for superior structural integrity while providing designers with a cost-effective method to save valuable land and protect water resources. The StormTech system is designed primarily to be used under parking lots thus maximizing land usage for commercial and municipal applications.

StormTech MC-4500 Chamber (not to scale)

Nominal Chamber Specifications

Size (L x W x H)	52" (1321 mm) x 100" (2540 mm) x 60" (1524 mm)
Chamber Storage	106.5 ft³ (3.01 m³)
Min. Installed Storage*	162.6 ft³ (4.60 m³)
Nominal Weight	120 lbs (54.4 kg)

* This assumes a minimum of 12" (305 mm) of stone above, 9" (229 mm) of stone below chambers, 9" (229 mm) of stone between chambers/end caps and 40% stone porosity.

StormTech MC-4500 End Cap (not to scale)

Nominal End Cap Specifications

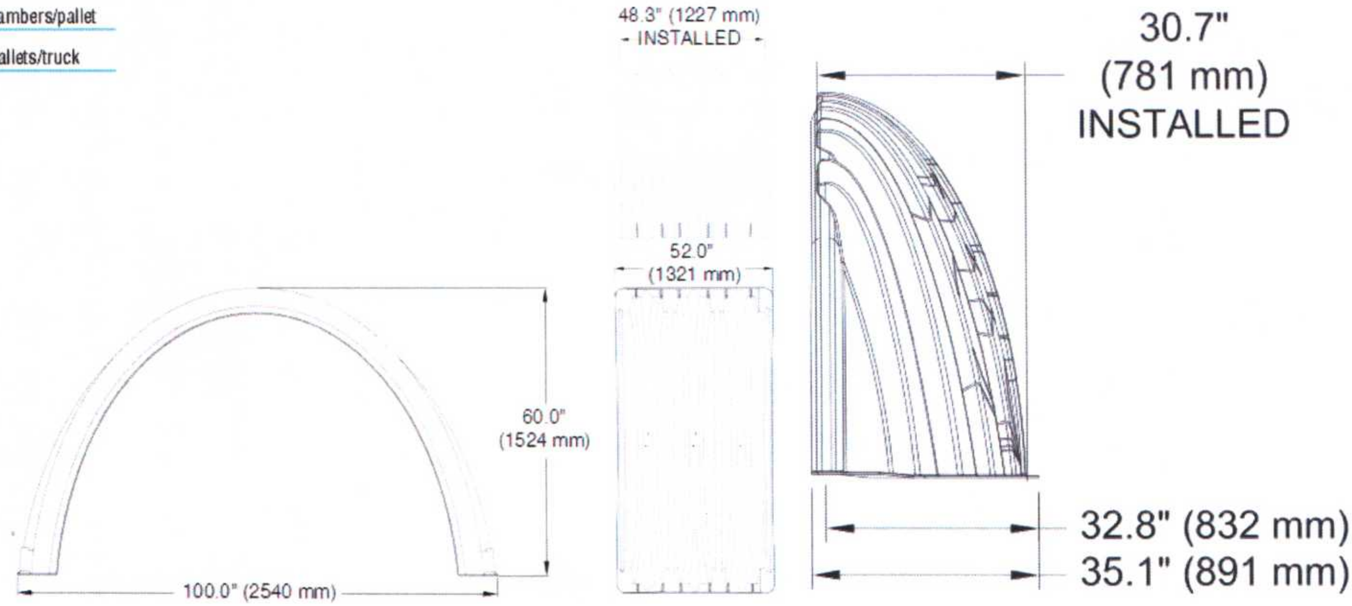
Size (L x W x H)	35.1" (891 mm) x 90.2" (2291 mm) x 59.4" (1509 mm)
End Cap Storage	35.7 ft³ (1.01 m³)
Min. Installed Storage*	108.7 ft³ (3.08 m³)
Nominal Weight	120 lbs (54.4 kg)

*This assumes a minimum of 12" (305 mm) of stone above, 9" (229 mm) of stone below, 12" (305 mm) of stone perimeter, 9" (229 mm) of stone between chambers/end caps and 40% stone porosity.

Shipping

8 chambers/pallet

11 pallets/truck



MC4500 chambers at an equivalent area of 18.233 SF/EA

MC4500 End Caps at an equivalent area of 10.82 SF/EA

*Total Equivalent area for each MC4500 underground chamber system with 3.5' of dead storage

Tables 8 and **9** provide cumulative storage volumes for the MC-4500 chamber and end cap. These tables can be used to calculate the stage-storage relationship for the retention or detention system. Digital spreadsheets in which the number of chambers and end caps can be input for quick cumulative storage calculations are available at www.stormtech.com. For assistance with site-specific calculations or input into routing software, contact the StormTech Technical Services Department.

TABLE 8 – MC-4500 Incremental Storage Volume Per Chamber

Assumes 40% stone porosity. Calculations are based upon a 9" (229 mm) stone base under the chambers, 12" (305 mm) of stone above chambers, and 9" (229 mm) spacing between chambers.

	Depth of Water in System Inches (mm)	Cumulative Chamber Storage ft^3 (m^3)	Total System Cumulative Storage ft^3 (m^3)
TOTAL SYSTEM VOLUME	81 (2057)	0	162.62 (4.605)
	80 (2032)	0	161.40 (4.570)
	79 (2007)	Stone 0	160.18 (4.536)
	78 (1981)	Cover 0	158.96 (4.501)
	77 (1956)	0	157.74 (4.467)
	76 (1930)	0	156.52 (4.432)
	75 (1905)	0	155.30 (4.398)
	74 (1880)	0	154.09 (4.363)
	73 (1854)	0	152.87 (4.329)
	72 (1829)	0	151.65 (4.294)
	71 (1803)	0	150.43 (4.260)
	70 (1778)	0	149.21 (4.225)
	69 (1753)	106.51 (3.016)	147.99 (4.191)
	68 (1727)	106.47 (3.015)	146.75 (4.156)
	67 (1702)	106.35 (3.012)	145.46 (4.119)
	66 (1676)	106.18 (3.007)	144.14 (4.082)
	65 (1651)	105.98 (3.001)	142.80 (4.044)
	64 (1626)	105.71 (2.993)	141.42 (4.005)
	63 (1600)	105.25 (2.981)	139.93 (3.962)
	62 (1575)	104.59 (2.962)	138.31 (3.917)
	61 (1549)	103.79 (2.939)	136.61 (3.869)
	60 (1524)	102.88 (2.913)	134.85 (3.819)
	59 (1499)	101.88 (2.885)	133.03 (3.767)
	58 (1473)	100.79 (2.854)	131.16 (3.714)
	57 (1448)	99.63 (2.821)	129.24 (3.660)
	56 (1422)	98.39 (2.786)	127.28 (3.604)
	55 (1397)	97.10 (2.749)	125.28 (3.548)
	54 (1372)	95.73 (2.711)	123.25 (3.490)
	53 (1346)	94.32 (2.671)	121.18 (3.431)
	52 (1321)	92.84 (2.629)	119.08 (3.372)
3.5' FULL VOLUME	51 (1295)	91.32 (2.586)	116.94 (3.311)
	50 (1270)	89.74 (2.541)	114.78 (3.250)
	49 (1245)	88.12 (2.495)	112.59 (3.188)
	48 (1219)	86.45 (2.448)	110.37 (3.125)
	47 (1194)	84.75 (2.400)	108.13 (3.062)
	46 (1168)	83.00 (2.350)	105.86 (2.998)
3' FULL VOLUME	45 (1143)	81.21 (2.300)	103.56 (2.933)
	44 (1118)	79.38 (2.248)	101.25 (2.867)
	43 (1092)	77.52 (2.195)	98.91 (2.801)
	42 (1067)	75.62 (2.141)	96.55 (2.734)
	41 (1041)	73.69 (2.087)	94.18 (2.667)
	40 (1016)	71.72 (2.031)	91.78 (2.599)
	39 (991)	69.73 (1.974)	89.36 (2.531)
	38 (965)	67.70 (1.917)	86.93 (2.462)
	37 (948)	65.65 (1.859)	84.48 (2.392)
	36 (914)	63.57 (1.800)	82.01 (2.322)
	35 (889)	61.46 (1.740)	79.53 (2.252)
	34 (864)	59.32 (1.680)	77.03 (2.181)
	33 (838)	57.17 (1.619)	74.52 (2.110)
	32 (813)	54.98 (1.557)	71.99 (2.038)
	31 (787)	52.78 (1.495)	69.45 (1.966)
	30 (762)	50.55 (1.431)	66.89 (1.894)
	29 (737)	48.30 (1.368)	64.32 (1.821)
	28 (711)	46.03 (1.303)	61.74 (1.748)
	27 (686)	43.74 (1.239)	59.15 (1.675)
	26 (680)	41.43 (1.173)	56.55 (1.601)
	25 (610)	39.11 (1.107)	53.93 (1.527)
	24 (609)	36.77 (1.041)	51.31 (1.453)
	23 (584)	34.41 (0.974)	48.67 (1.378)
	22 (559)	32.03 (0.907)	46.03 (1.303)
	21 (533)	29.64 (0.839)	43.38 (1.228)
	20 (508)	27.23 (0.771)	40.71 (1.153)
	19 (483)	24.81 (0.703)	38.04 (1.077)
	18 (457)	22.38 (0.634)	35.37 (1.001)
	17 (432)	19.94 (0.565)	32.68 (0.925)
	16 (406)	17.48 (0.495)	29.99 (0.849)
	15 (381)	15.01 (0.425)	27.29 (0.773)
	14 (356)	12.53 (0.355)	24.58 (0.696)
	13 (330)	10.05 (0.284)	21.87 (0.619)
	12 (305)	7.55 (0.214)	19.15 (0.542)
	11 (279)	5.04 (0.143)	16.43 (0.465)
	10 (254)	2.53 (0.072)	13.70 (0.388)
	9 (229)	0	10.97 (0.311)
	8 (203)	0	9.75 (0.276)
	7 (178)	0	8.53 (0.242)
	6 (152)	Stone 0	7.31 (0.207)
	5 (127)	Foundation 0	6.09 (0.173)
	4 (102)	0	4.87 (0.138)
	3 (76)	0	3.66 (0.104)
	2 (51)	0	2.44 (0.069)
	1 (25)	0	1.22 (0.035)

NOTE: Add 1.22 ft^3 (0.035 m^3) of storage for each additional inch (25 mm) of stone foundation. Contact StormTech for cumulative volume spreadsheets in digital format.

5.0 Cumulative Storage Volumes

TABLE 9 – MC-4500 Incremental Storage Volume Per End Cap

Assumes 40% stone porosity. Calculations are based upon a 9" (229 mm) stone base under the end caps, 12" (305 mm) of stone above end caps, 9" (229 mm) of spacing between end caps and 12" (305 mm) of stone perimeter.

Depth of Water in System Inches (mm)	Cumulative Chamber Storage ft³ (m³)	Total System Cumulative Storage ft³ (m³)
81 (2057)	0	108.69 (3.078)
80 (2032)	0	107.62 (3.047)
79 (2007)	Stone 0	106.54 (3.017)
78 (1981)	Cover 0	105.46 (2.986)
77 (1956)	0	104.38 (2.956)
76 (1930)	0	103.31 (2.925)
75 (1905)	0	102.23 (2.895)
74 (1880)	0	101.15 (2.864)
73 (1854)	0	100.07 (2.834)
72 (1829)	0	99.00 (2.803)
71 (1803)	0	97.92 (2.773)
70 (1778)	0	96.84 (2.742)
69 (1753)	35.71 (1.011)	95.76 (2.712)
68 (1727)	35.71 (1.011)	94.69 (2.681)
67 (1702)	35.70 (1.011)	93.60 (2.651)
66 (1676)	35.67 (1.010)	92.51 (2.620)
65 (1651)	35.62 (1.009)	91.40 (2.588)
64 (1626)	35.56 (1.007)	90.29 (2.557)
63 (1600)	35.47 (1.004)	89.16 (2.525)
62 (1575)	35.36 (1.001)	88.01 (2.492)
61 (1549)	35.21 (0.997)	86.85 (2.459)
60 (1524)	35.05 (0.992)	85.67 (2.426)
59 (1499)	34.86 (0.987)	84.48 (2.392)
58 (1473)	34.64 (0.981)	83.27 (2.358)
57 (1448)	34.40 (0.974)	82.05 (2.323)
56 (1422)	34.13 (0.966)	80.81 (2.288)
55 (1397)	33.83 (0.958)	79.55 (2.253)
54 (1372)	33.51 (0.949)	78.28 (2.217)
53 (1346)	33.16 (0.939)	77.00 (2.180)
52 (1321)	32.79 (0.928)	75.70 (2.144)
51 (1295)	32.39 (0.917)	74.38 (2.106)
50 (1270)	31.98 (0.906)	73.06 (2.069)
49 (1245)	31.54 (0.893)	71.71 (2.031)
48 (1219)	31.07 (0.880)	70.36 (1.992)
47 (1194)	30.59 (0.866)	68.99 (1.954)
46 (1168)	30.09 (0.852)	67.61 (1.915)
45 (1143)	29.56 (0.837)	66.22 (1.875)
44 (1118)	29.02 (0.822)	64.81 (1.835)
43 (1092)	28.45 (0.806)	63.40 (1.795)
42 (1067)	27.87 (0.789)	61.97 (1.755)
41 (1041)	27.27 (0.772)	60.53 (1.714)
40 (1016)	26.65 (0.755)	59.08 (1.673)
39 (991)	26.01 (0.736)	57.62 (1.632)
38 (965)	25.35 (0.718)	56.15 (1.590)
37 (948)	24.68 (0.699)	54.67 (1.548)
36 (914)	23.99 (0.679)	53.18 (1.506)
35 (889)	23.28 (0.659)	51.68 (1.463)
34 (864)	22.56 (0.639)	50.17 (1.421)
33 (838)	21.82 (0.618)	48.64 (1.377)
32 (813)	21.06 (0.596)	47.11 (1.334)
31 (787)	20.29 (0.575)	45.57 (1.290)
30 (762)	19.50 (0.552)	44.02 (1.247)
29 (737)	18.70 (0.530)	42.46 (1.202)
28 (711)	17.88 (0.506)	40.89 (1.158)
27 (686)	17.04 (0.483)	39.31 (1.113)
26 (680)	16.19 (0.459)	37.73 (1.068)
25 (610)	15.33 (0.434)	36.14 (1.023)
24 (609)	14.46 (0.410)	34.53 (0.978)
23 (584)	13.58 (0.384)	32.93 (0.932)
22 (559)	12.68 (0.359)	31.31 (0.887)
21 (533)	11.77 (0.333)	29.69 (0.841)
20 (508)	10.85 (0.307)	28.06 (0.794)
19 (483)	9.91 (0.281)	26.42 (0.748)
18 (457)	8.97 (0.254)	24.77 (0.702)
17 (432)	8.01 (0.227)	23.12 (0.655)
16 (406)	7.04 (0.199)	21.46 (0.608)
15 (381)	6.07 (0.172)	19.80 (0.561)
14 (356)	5.08 (0.144)	18.13 (0.513)
13 (330)	4.08 (0.116)	16.45 (0.466)
12 (305)	3.07 (0.087)	14.77 (0.418)
11 (279)	2.06 (0.058)	13.09 (0.371)
10 (254)	1.03 (0.029)	11.39 (0.323)
9 (229)	0	9.70 (0.275)
8 (203)	0	8.62 (0.244)
7 (178)	0	7.54 (0.214)
6 (152)	Stone 0	6.46 (0.183)
5 (127)	Foundation 0	5.39 (0.153)
4 (102)	0	4.31 (0.122)
3 (76)	0	3.23 (0.092)
2 (51)	0	2.15 (0.061)
1 (25)	0	1.08 (0.031)

NOTE: Add 1.08 ft³ (0.031 m³) of storage for each additional inch (25 mm) of stone foundation. Contact stormtech for cumulative volume spreadsheets in digital format.

May 11, 2015

Premier Design Build Group
1000 West Irving Park Road, Suite 200
Itasca, IL 60143

Attn: Mr. Alan C. Zocher
President

Re: Geotechnical Exploration Report
Proposed Warehouse Development
901 Northview Road
Waukesha, WI
PSI Report No. 00521212

Dear Mr. Zocher:

Professional Service Industries, Inc. (PSI) is pleased to submit our Geotechnical Exploration Report for the Proposed Industrial Development in Mt. Pleasant, Wisconsin. This report includes the results of field and laboratory testing, recommendations for foundations, floor slabs and pavements, as well as general site development recommendations.

PSI appreciates the opportunity to perform this geotechnical study and we look forward to continued participation during the design and construction phases of this project. If you have any questions pertaining to this report, or if we may be of further service, please contact our office.

Respectfully submitted,

PROFESSIONAL SERVICE INDUSTRIES, INC.

Colin T. Henderson, E.I.T.
Staff Engineer

Paul J. Koszarek, P.E.
Department Manager
Geotechnical Services

WISCONSIN PAUL J. KOSZAREK E-36708 WAUKESHA WIS. PROFESSIONAL ENGINEER

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PROJECT INFORMATION

Project Authorization

The following Table summarizes, in chronological order, the Project Authorization History for the services performed and represented in this report by Professional Service Industries, Inc. (PSI):

DOCUMENT AND REFERENCE NUMBER	DATE	SOURCE OF REQUEST	AUTHOR OR AGENT
Request for Proposal	3/4/2015	Premier Design Build Group	Mr. Alan Zocher President
PSI Proposal Number: 146889	3/9/2015	PSI	Paul J. Koszarek, P.E. David M. Barndt, P.E.
Change Order No.: 1	4/20/2015	PSI	Paul J. Koszarek, P.E.

Project Description

Briefly, PSI understands that the project includes construction of a new warehouse development, located at 901 Northview Road, in Waukesha, Wisconsin. The following Table lists the material and information provided for this project:

DESCRIPTION OF MATERIAL	PROVIDER/SOURCE	DATE
Proposed Site Plan	Premier Design Build Group	3/4/2015
CAD Drawing with existing contours	Pinnacle Engineering Group	4/3/2015

From the information provided by the client, it is understood that the proposed project will consist of one single-story structure, without a basement, about 213,949 square feet in plan area. The building construction is anticipated to be mostly steel framed with precast walls, with a concrete slab-on-grade floor. Structural loads were not provided; however, based upon PSI's experience with similar structures, it is estimated that maximum wall and column loads will be on the order of 4 kips per foot and 150 kips, respectively.

Based upon the information provided by Premier Design Build Group, it is understood that the finished first floor elevation will be 139 feet (Local). The existing grades within the proposed building footprint vary from about 133 to 140 feet (Local). Therefore, fills on the order of about 1± to 7± feet, and cuts of up to about 1± foot will be required within the proposed building pad area.

Additional site work will include construction of new driveways, parking areas, and loading docks, and the demolition of the existing AMF Bowling Facility. Grades for the new pavements are anticipated to be within 2± feet of existing grade. The loading docks are planned on the east and west sides of the building. Final grades for the loading grades are anticipated to be about 4 feet lower than the finished floor elevation of the building.

The following Table lists the structural loads and site features that are required for or are the design basis for the conclusions contained in this report:

STRUCTURAL LOAD/PROPERTY	REQUIREMENT/DESIGN BASIS	
BUILDING		
Maximum Column Loads	150 kips	B
Maximum Wall Loads	4 kips per lineal foot (klf)	B
Finished Floor Elevation and Style	139 feet (Local)/Slab-on-Grade; without a basement	R
Maximum Floor Loads and Size	150 pounds per square foot (psf)	B
Settlement Tolerances	1-inch total; ¾-inch differential between adjacent columns	B
PAVEMENTS		
Pavement 18-kip ESAL (cycle & duration)	Light Duty Parking Lot– 30,000 ESAL; with a life expectancy of 20 years	B
Pavement 18-kip ESAL (cycle & duration)	Heavy Duty Trucking Routes-300 Tractor Trailer Semi-Trucks/week, with life expectancy of 20 years	B
GRADING		
Planned Grade Variations at Surface of Site in Building Pad Area	1± to 7± feet of fill; 1± feet of cut	R
Planned Grade Variations at Surface of Site in Parking Lot and Driveway Areas	2± feet of cut/fill	B
Planned Grade Variations at Surface of Site in Loading Dock Area	1± to 5± feet of cut	R

R = Reported to PSI by Others
B = Report has been prepared based on this parameter or loading in the absence of client supplied information at the time of this report

The geotechnical recommendations presented in this report are based on the available project information, building location, and the subsurface materials described in this report. If the noted information is incorrect, please inform PSI in writing so that we may amend the recommendations presented in this report if appropriate and if desired by the client. PSI will not be responsible for the implementation of its recommendations when it is not notified of changes in the project.

Purpose and Scope of Services

The purpose of this study was to explore the subsurface conditions at the site and develop geotechnical design criteria regarding foundations, floor slabs and pavements for the proposed project. Subgrade preparation recommendations and construction considerations are also provided. PSI's scope of services included drilling a planned total of 20 soil borings, select laboratory testing, and preparation of this geotechnical report.

The scope of services did not include an environmental assessment for determining the presence or absence of wetlands, or hazardous or toxic materials in the soil, bedrock, surface water, groundwater, or air on or below, or around this site. Any statements in this report or on the boring logs regarding odors, colors, and unusual or suspicious items or conditions are strictly for informational purposes.

SITE AND SUBSURFACE CONDITIONS

Site Location and Description

The project site is located at 901 Northview Road, at the southwest corner of the intersection of Northview Road and Aviation Drive, in Waukesha, Wisconsin. The site is currently occupied the AMF Bowling Facility structure with an associated paved parking lot. The site is currently vacant and utilized for agricultural purposes. The site is fronted by Northview Road to the north, Aviation Drive to the east, paved parking and a detention pond to the south, and a strip of green space followed by multiple commercial developments to the west.

The topography of site was rolling in nature, with site grades ranging from about 116 to 142 feet (Local) at the boring locations, generally sloping to the south and north from the north side of the existing AMF Bowling facility building. The site Latitude and Longitude is 43.031533ºN and 88.238828ºW, respectively.

Subsurface Conditions

The subsurface conditions were planned to be explored with 20 soil test borings (B-1 through B-20). The borings were planned to extend to a depth of 10 feet in the loading dock and pavement areas (B-13 through B-20), and to 20 feet within the building pad (B-1 through B-12). However, auger refusal was encountered on probable cobbles, boulders, or bedrock, prior to reaching planned depths at B-1 through B-3, at depths ranging from about 15 to 16 feet below existing ground surface. In addition, auger refusal was encountered within the fill at B-10 at a depth of 20 feet on probable boulders, which was originally believed to be bedrock at the time of exploration. Borings B-18, B-11, and B-12 were extended beyond the planned depths to depths ranging from 12.5 to 40 feet due to fill and buried topsoil materials encountered at the planned depth. The following Table indicates the general locations, elevations and completion depths to which the borings were performed.

BORING NUMBER	GENERAL LOCATION	BORING ELEVATION (FEET)	COMPLETION DEPTH (FEET)
B-1	Building Pad	138	15
B-2	Building Pad	138	16
B-3	Building Pad	138	15.5
B-4	Building Pad	139	20
B-5	Building Pad	139	20
B-6	Building Pad	138	20
B-7	Building Pad	137	20
B-8	Building Pad	136	20
B-9	Building Pad	136	20
B-10	Building Pad	133	20
B-11	Building Pad	135	35
B-12	Building Pad	132	40
B-13	North Parking Lot (Pavement)	136	10
B-14	North Parking Lot (Pavement)	137	10
B-15	West Loading Dock Area	140	10
B-16	East Loading Dock Area	139	10
B-17	West Loading Dock Area	136	10
B-18	East Loading Dock Area	135	12.5
B-19	South Drive Area (Pavement)	134	10
B-20	South Drive Area (Pavement)	132	10

The borings were field located by PSI by measuring from fixed points at the site. The approximate boring locations can be found in the Appendix of this report. The surface elevations shown on the logs were determined by the drill crew utilizing conventional leveling techniques. The floor slab of the existing AMF Bowling Facility building, located near the center of the site, was utilized as a benchmark, with a known elevation of 139.93 feet (local). The elevations are considered accurate to within about one foot. The borings were advanced utilizing hollow-stem auger drilling methods and soil samples were routinely obtained during the drilling process. Drilling and sampling techniques were accomplished generally in accordance with ASTM procedures. Upon completion, the borings were backfilled with bentonite chips and the upper 4± were patched with cold-mix asphalt at locations where pavement was present.

Representative soil samples were obtained from the soil borings and were returned to PSI's laboratory where they were visually classified using the Unified Soil Classification System (USCS) as a guideline. Further, PSI conducted limited laboratory testing on select soil samples to aid in identifying and describing the physical characteristics of the soils and to aid in defining the site soil stratigraphy. The results of the field exploration and laboratory tests were used in PSI's engineering analysis and in the formulation of our engineering recommendations.

Warehouse Building Borings (B-1 through B-12)

The surface materials present at the warehouse building borings consisted of about 3 to 12 inches of asphalt, but more typically in the range of about 3 to 6 inches. The asphalt was underlain by about 4 to 8 inches of aggregate base. No discernable base layer was observed at B-10 and B-11. The surface materials at B-8 through B-12 were underlain by fill soils consisting of brown to dark brown silty or lean clay, silt, sandy silt or silt with gravel to depths ranging from about 6 to 26 feet (108 to 130± feet). **It should be noted that traces of brick and concrete rubble and/or wood pieces were present within the fill at B-8, B-10, and B-11.** Moistures contents of the fill soils ranged from about 4% to 29%, indicating a moist to very moist condition. The fill soils were medium dense to very dense, with N-values ranging from about 3 blows per foot to 50 blows per 3 inches of sampler penetration, but more typically in the range of about 3 to 42 blows per foot. It should be noted that the N-values of fill soils may have been elevated due to the presence of rubble and the frozen nature of the soils within the upper 3.5 feet of the borings, and the possible presence of boulders at B-10 where auger refusal was encountered within the fill at a depth of about 20 feet (113± feet).

A layer of possible buried topsoil was present below the fill at B-11 and B-12 to depths of about 26 to 28 feet below existing ground surface (106 to 107± feet).

Loss-on-ignition testing was performed on the fill at B-10 (6 to 7.5 feet) and B-12 (1 to 2.5 feet), and the buried topsoil at B-11 (26 to 27.5 feet) and B-12 (23.5 to 25 feet). The moisture contents within these soils were observed to be in the range of 11% to 32%, indicating a moist to very moist condition. The loss-on-ignition (LOI) testing performed on the subjected samples revealed organic contents ranging from approximately 4.3% to 11.2%. Typically, soils with organic contents greater than 5% are considered to be "organic". The individual results of each sample are shown in the boring logs in the appendix.

The soils underlying the fill at B-8 and B-9, and the surface materials at the rest of the locations, consisted of native granular and fine-grained soils to the maximum depths explored. The granular and fine-grained soils were comprised of brown silt/sandy silt, silt/sandy silt with gravel, or silty sand and gravel with probable cobbles and boulders. Moistures contents of the granular soils ranged from about 2% to 16%, indicating a damp to very moist condition, but were more typically in the range of 3% to 9%. The granular soils were in a loose to very dense condition, with N-values ranging from about 6 blows per foot to 50 blows per 1 inch of sampler penetration. Extremely difficult drilling was experienced at most of the borings, especially where probable cobbles and boulders were present.

Auger refusal on probable cobbles, boulders, or bedrock was experienced above the planned depths at B-1 through B-3 at depths ranging from about 15 to 16 feet (122 to 123± feet). Additionally, auger refusal was encountered on probable boulders within the fill at B-

10 at a depth of about 20 feet (113± feet). The approximate refusal depths and elevations at the boring locations are outlined in the following table.

BORING NUMBER	REFUSAL DEPTH (FEET)	REFUSAL ELEVATION (FEET [LOCAL])
B-1	15	123±
B-2	16	122±
B-3	15.5	122.5±
B-10	20	113±

As an exception to the foregoing was observed at B-9 where brown lean clay soils were present below the native granular soils at a depth of about 17 feet (119± feet), and extended to the boring termination depth. The moisture content of the clay soil was approximately 16%, indicating a moist condition. The clay soils were very stiff in consistency, with pocket penetrometer values and measured Rimac unconfined compressive strength values ranging from about 2.89 to 3.0 tons per square foot (tsf).

Loading Dock and Pavement Borings (B-13 through B-20)

Boring B-13 through B-20 were performed within the pavement and loading dock areas. The surface materials present at B-15 through B-18 consisted of about 3 to 4 inches of asphalt underlain by about 5 to 14 inches of aggregate base. About 3 to 5 inches of dark brown organic silt topsoil was present at the surface of B-13, B-14, B-19 and B-20. Fill and possible fill soils, consisting of silty clay and silt/sandy with gravel, were present below the surface materials at B-17 through B-20 to depths ranging from about 5 feet to the boring termination depths (122 to 131± feet). Moistures contents of the fill soils ranged from about 5% to 18%, indicating a moist to very moist condition. The cohesive fill soils were very stiff, with measured Rimac unconfined compressive strength values and estimated hand penetrometer values ranging from about 0.5 to 2.75 tons per square foot (tsf). The granular fill soils exhibited N-values of about 10 to 36 blows per foot, indicating a medium dense to very dense condition. It should be noted that the N-values of fill soils may have been elevated due to the presence of rubble and the frozen nature of the soils within the upper 3.5 feet of the borings.

A layer of buried topsoil comprised of dark brown organic silty clay was present below the fill at B-18 to a depth of about 11 feet (124± feet). Loss-on-ignition testing was performed on the dark brown organic silty clay buried topsoil at B-18 at the 8.5 to 10 foot sample interval. The buried topsoil was in a very moist condition, with a moisture content of approximately 25%. The loss-on-ignition (LOI) testing performed within the organic silty clay revealed an organic content of approximately 5.6%. Typically, soils with organic contents greater than 5% are considered to be “organic”.

The underlying soils below the fill at B-17, the buried topsoil at B-18, and the surface materials at the rest of locations, consisted of native granular and fine-grained soils to the maximum depths explored. The granular and fine-grained soils were comprised of silt, silt with gravel, or silty sand and gravel with probable cobbles and boulders. Moistures contents of the granular soils ranged from about 2% to 16%, indicating moist to very moist condition, but were more typically in the range of 3% to 9%. The granular soils were in a medium dense to very dense condition, with N-values ranging from about 20 blows per foot to 50 blows per ½ inch of sampler penetration. Extremely difficult drilling was experienced at most of the borings, especially where probable cobbles and boulders were present.

The above subsurface description is of a generalized nature to highlight the major subsurface stratification features and material characteristics. The boring logs included in the appendix should be reviewed for specific information at individual boring locations. These records include soil descriptions, stratifications, penetration resistances, locations of the samples and laboratory test data. The stratifications shown on the boring logs represent the conditions only at the actual boring locations. Variations may occur and should be expected between boring locations. The stratifications represent the approximate boundary between subsurface materials and the actual transition may be gradual. Water level information obtained during field operations is also shown on these boring logs. The samples that were not discarded during classification or altered by laboratory testing will be retained for 60 days from the date of this report and then will be discarded.

Groundwater Information

Groundwater was observed within B-10 through B-12 by PSI during drilling at depths of about 16 to 36 feet (96 to 117± feet). Upon completion of drilling operations, groundwater was encountered at B-10 and B-12 at depths of about 13 to 27 feet (105 to 120± feet) above the caved soils. Fluctuations in the groundwater level should be anticipated throughout the year depending on variations in climatological conditions and other factors not apparent at the time the Borings were performed. The possibility of groundwater level fluctuation and perched water conditions should be considered when developing the design and construction plans for the project.

EVALUATION AND RECOMMENDATIONS

Geotechnical Discussion

There are seven primary geotechnical related concerns at this site, which will affect the design of the building as well as the earthwork operations for this project. The following summarizes this concern:

1) ***Existing undocumented fill and possible fill materials were encountered within B-8 through B-12 and B-17 through B-20 (as well as buried topsoil at B-11, B-12, and B-18) performed within or near the southern half of proposed building pad area extending to depths ranging from about 5 to 28 feet (106 to 131± feet) below existing ground surface. It should be anticipated that the depth and consistency of the existing fill materials may change from within the building.***

In view of the subsurface conditions encountered in the test borings, together with the structural loading criteria and development grades anticipated, conventional spread footings, along with conventional slab-on-grade construction, can be used for support of the north half of the proposed structure. However, deposit of fill and buried topsoil was observed within B-8 through B-12 and B-17 through B-20 performed within or near the southern half of the proposed structure to depths of approximately 5 to 28 feet (106 to 131± feet) below existing grade. **Due to the variability and undocumented nature of the existing fill at these locations within the southern half of the building, the existing fill soils are not considered suitable for support of foundations in their current condition.** The following table shows the approximate elevations of suitable bearing native soils at the building boring locations and estimated undercut depths below frost depth.

BORING NO.	ELEVATION TO TOP OF SUITABLE NATIVE NON-ORGANIC SOILS (FEET [LOCAL])	APPROXIMATE UNDERCUT DEPTHS (FEET)
B-8	130±	5
B-9	128±	7
B-10	113±	22
B-11	107±	28
B-12	106±	29
B-17	131±	4
B-18	124±	11

Based on the relatively deep undercuts that would be required for the approximate south half of the site, it is recommended that the foundations be supported by improving the fill and buried topsoil soils in place by using a series of Geopiers®, also known as rammed aggregate piers. Once the Geopier® system is installed, the foundations can be supported by compacted engineered fill used to raise the site to planned grades. The Geopier® system typically is completed by excavating a series of holes typically terminating below the poor soils and then backfilling the holes with crushed stone in lifts. Each lift of stone would be compacted using a specially designed vibratory ram which imparts vibration radially (both vertically and horizontally). This will create a series of very stiff columns of compacted stone while also improving the load carrying capacity of the poor soils. Typically, once the piers are installed, a typical shallow foundation designed for between 3,000 and 4,000 psf can be placed on top of the piers to support the building. Typically, these piers are installed on a design/building basis; therefore, design values for both the new allowable bearing capacity for foundation design would be provided by the rammed aggregate pier contractor.

Recognizing that complete removal and replacement of the existing fill soils would be very expensive throughout the proposed floor slab areas, parking lots, and drive areas, the planned floor slabs and pavements could be placed directly upon the existing fill, provided that some risk of consolidation/settlement within the floor slab and pavement areas, and resultant distress, can be accepted. The existing fill soils should be properly prepared as indicated in the following Site Preparation section. At least some overexcavation of unsuitable soils will likely be necessary. It must be recognized that differential settlements and poorer pavement performance may result. Design of a conventional flexible (asphalt) pavement on the existing subgrade soils will generally require a somewhat thicker pavement section and an increased maintenance program throughout the pavement design life. A discussion of the foundation design parameters, as well as the support conditions for the floor slabs and pavements is included in later sections.

2) ***Settlement monitoring is recommended for area being supported by Rammed Aggregate Piers;***

PSI recommends that a settlement monitoring program be implemented immediately after finished subgrade is obtained within the southern portion of the building pad where Geopiers are being installed. The settlement plates should be installed in a grid pattern with a minimum spacing of 1 per 20,000 square feet with a minimum of three plates being installed. A detail of the recommended settlement plate construction is included in the appendix of this report. The settlement plate should be installed level and bearing approximately 12 inches below finished subgrade elevation and then backfilled with sand. The area surrounding the settlement plates should be protected as to not allow for construction equipment to disturb them during the monitoring period. The X, Y and Z coordinates of the top of the monitoring pole should be recorded by a registered surveyor until three consecutive readings have been less than 0.009 feet from the previous day's readings. It is anticipated that this process should take approximately 1 to 2 weeks. Results of the survey should be forwarded to PSI for analysis to determine when foundation construction may begin.

3) ***It should be anticipated that the near surface soils will be in a very moist or wet condition upon stripping topsoil and will therefore require drying time to regain stability.***

The process of drying these soils can be accelerated by mechanical means such as disking. Care should be taken that while these soils are in a very moist condition that construction traffic be limited or not allowed in order to maintain a stable working platform. Similar drying procedures should be used after precipitation events as well.

4) ***Based upon PSI's experience with the soils encountered at this site, it is likely that cobbles and boulders will be unearthed during excavation activities.***

It should be anticipated that cobbles and boulders will be encountered during excavation activities. These obstructions will require complete removal if observed at or above the bottom of the footing slab elevations. Voids created beneath the bottom of the footing elevation will need to be filled with engineered fill as described within the Site Preparation section of this report.

5) ***It should be anticipated that loose granular soils will be present at the estimated footing depths at some locations.***

Due to the granular nature of the bearing soils, it will be necessary to recompact the soils using vibratory compactors upon completion of excavation activities.

6) ***Groundwater was encountered within the proposed building pad at B-10 through B-12 at depths ranging from about 16 to 36 feet (96 to 117± feet)*** **below existing ground surface*.*** ***Based on the relatively deep fill soils encountered at these location performed within the proposed building pad, some of the recommended over-excavations will encroach upon or extend below the groundwater.***

Based upon the borings performed, and the estimated groundwater levels, some of the foundation subgrade over-excavations will encroach upon or extend below the groundwater and into the existing fill and native soils. This will likely result in substantial sloughing and caving, and the potential for significant subgrade instability. In areas where Geopiers are used for foundation support, temporary casing may be required to maintain stability for Geopier foundations extending below the groundwater and into wet granular soils. For utility areas, some overexcavation of softened or loosened soils, in conjunction with the use of a crushed stone working mat, may be necessary to establish a stable bearing subgrade. Significant widening and/or bracing of excavations will generally be required. Significant groundwater related difficulty may be encountered in at least some areas.

Test pits should be completed in order that the Geopier engineer can accurately ascertain if temporary casing is required or not.

7) ***The existing AMF Bowling Facility that is planned for demolition is located on the site within the proposed building area.***

Old building foundations, building remnants, associated underground utilities, light poles, or unsuitable backfill materials, should be completely removed from within and a minimum of 10 feet beyond the new building pad area. The resulting excavation should then be backfilled with engineered fill as outlined in the Site Preparation section of this report. Complete removal of foundations, foundation walls or concrete floor slabs need not be removed from within parking and green areas; however, PSI recommends they be removed to a minimum depth of 2 feet below subgrade (bottom of base course elevation) to provide a uniform subgrade condition. Basement slabs located below 2 feet from planned subgrade elevation may be left in place; however they should be broken into maximum 6 inch pieces to facilitate drainage.

The following geotechnical related recommendations have been developed on the basis of the subsurface conditions encountered and PSI's understanding of the proposed development. PSI has presented these recommendations with the understanding that the owner is willing to accept an elevated risk of settlement and utilize the existing fill for support of the building's floor slabs in lieu of the much higher cost of removal and replacement of these materials. Should changes in the project criteria occur, a review must be made by PSI to determine if modifications to our recommendations will be required.

Site Preparation

Prior to the placement of new fill or preparation of the construction area subgrade, PSI recommends that the existing surficial organic matter, trees including root bulbs, frozen soils, topsoil, and surficial pavement materials be removed from within and a minimum of 10 feet beyond the building and paved areas. Unsuitable soils encountered should be selectively undercut and/or stabilized in place. A representative of a qualified geotechnical engineer should determine the need for and depth of removal or stabilization at the time of construction.

Special care should be given in the removal of the existing AMF Bowling Facility structure at the project site. PSI recommends that the existing foundations, walls, floor slabs, as well as any foundation elements from any previous structures (such as parking lot light poles), be removed in their entirety from beneath and a minimum of 10 feet beyond the new building pad area and properly disposed of off-site. In pavement areas, the walls/footings/slabs should be removed to a depth of at least 2 feet below planned bottom of footing elevation. Existing walls/footing/slabs could remain in place below a depth of 2 feet from bottom of base course elevation; however, the slabs are recommended to be broken into pieces having a maximum dimension of 6 inches in any direction. The removal and/or breaking of buried structures should be observed by a representative of a geotechnical engineer. Voids caused by the removal of the debris should be replaced with compacted fill as outlined below.

Some stabilization or selective undercutting of the soils may be required depending upon the moisture conditions at the time of construction. If unstable soils are observed in an area, they should be stripped from that area until more stable soils are observed or stabilized in place. If allowed to dry these soils could be used as engineered fill provided they are placed and compacted as outlined below. A representative of a qualified geotechnical engineer should determine the need for and actual stabilization technique at the time of construction.

Following the overexcavation of the pavement and old fill materials, and/or the installation of Geopiers® within the south half of the proposed building, the structural areas should be proofrolled. If the proofroll is not feasible, the native soils should be inspected by a representative of the geotechnical engineer prior to placement of backfill. The proofroll should be conducted prior to placement of new fill to raise site grades. Proofrolling should be performed with a fully-loaded tandem axle dump truck or rubber tired vehicle of similar size and weight, typically a 9 tons/axle truck where cohesive soils are present. Soils that are observed to rut or deflect excessively under the moving load (typically > 1"), should be undercut and replaced with properly compacted engineered fill. The proofrolling and undercutting activities should be documented by a representative of a qualified geotechnical engineer and should be performed during a period of dry weather. The subgrade soils should be scarified and compacted to at least 95 percent of the maximum dry density and within 3 percent of the optimum moisture content as obtained by the modified Proctor test ASTM D 1557. The depth of scarification should not be less than six inches below the surface. Drying or wetting of the subgrade soils, typically to within 3% of the optimum moisture content, may be advised to facilitate compaction.

Newly placed engineered fill required to establish site grades should be free of organic, frozen, or other deleterious materials, have a maximum particle less than 3 inches. Clay fills should have a liquid limit less than 45 and plasticity index less than 25 and greater than 11. Other soils with Atterberg limits outside those recommended should be reviewed by the geotechnical for their intended use. If a fine-grained clay soil is used for fill, close moisture content control will be required to achieve the recommended degree of compaction. Engineered fill should be compacted to at least 95 percent of the maximum dry density and within 3 percent of the optimum moisture content as determined by the modified Proctor ASTM Designation D 1557. Also, PSI recommends that a qualified geotechnical engineer test and document the engineered fill materials prior to placement.

As stated, engineered fill should be placed in maximum lifts of eight inches of loose material and should be compacted within 3% of the optimum moisture content value as determined by the modified Proctor test (ASTM D 1557). If water is to be added, it should be uniformly applied and thoroughly mixed into the soil by disking or scarifying. Each lift of compacted engineered fill should be observed and tested by a representative of PSI prior to placement of subsequent lifts. The minimum lateral extent of the overexcavation of poor soil and subsequent placement and compaction of engineered fill should be equal to or greater than the depth of overexcavation below finished floor elevation or 10 feet, whichever is greater.

Foundation Recommendations

Provided that the building pad has been prepared as recommended in this report, the approximate north half of the building can be supported by conventional continuous wall and column footings. Based on a planned finish floor elevation of 139, with interior and exterior footings will bear at approximately 137.5 feet and 135 feet, respectively. For the north half of the building, it is anticipated that shallow foundations placed at normal frost depths will bear within suitable native fine-grained or granular soils or newly placed engineered fill.

Based on the results of the borings, PSI recommends that column and wall footings in the approximate north half of building **bearing upon suitable native soils, upon compacted structural fill or lean concrete slurry used to replace unsuitable materials** be designed for a maximum net allowable soil bearing pressure of **3,000 pounds per square foot (psf)** based on dead load plus design live load. Minimum dimensions of 18 inches for continuous footings and 30 inches for any column footings should be used in foundation design to minimize the possibility of a local bearing capacity failure, even if the allowable bearing pressure recommended herein is not fully utilized.

The table below shows the anticipated undercut depths below frost depth in the north half of the building utilizing the recommended 3,000 psf allowable bearing pressures at the building and loading dock boring locations to the nearest half of a foot.

BORING NO.	ELEVATION OF SUITABLE SOILS WITH 3,000 PSF BEARING PRESSURE (FEET [LOCAL])	ANTICIPATED UNDERCUT BELOW FROST DEPTH WITH 3,000 PSF
NORTH HALF OF BUILDING PAD		
B-1	137±	0
B-2	137±	0
B-3	137±	0
B-4	138±	0
B-5	138.5±	0
B-6	137±	0
B-7	136.5±	0
B-13	135.5+	0*
B-14	136.5±	0
B-15	139.5±	0
B-16	138±	0

*No undercutting because the elevation is already below bottom of footing after stripping surface materials.

Based on PSI's boring data, it is recommended that the southern half of the building be supported by a RAP system (Geopiers). Additional borings to the north of Borings B-17, B-8, B-9 and B18 would be required in order to more accurately define the northern extent of the Geopier system. The actual bearing pressure used to design the footings in this portion of the building should be specified by the RAP company. Typically, once a RAP system is installed, conventional continuous wall and column footings can be used to support the structure that are designed for between 3,000 psf and 4,000 psf bearing capacity. Test pits should be completed in order that the Geopier engineer can accurately ascertain how far the Geopier® system should extend and where shallow conventional spread foundations should begin within the building pad.

In addition, brick and concrete rubble and other debris may be present within the fill at some locations, and difficult drilling should be expected. It may be necessary to offset pier locations. PSI recommends that test pits be completed in order better define the magnitude of the debris in the presence of the Geopier Engineer. Additionally, the test pits will also be a useful tool in determining if temporary casing will be required during installation of the piers. If both of these items could be eliminated from the Geopier estimate, this could significantly reduce the cost of the piers.

Minimum plan dimensions of 30 inches for column footings and 18 inches for continuous wall footings should be also be used in foundation design utilizing Geopiers® to provide an acceptable factor of safety against a local bearing capacity failure and provide adequate room for cleaning of the bearing surface.

After opening, **PSI recommends that the soils at foundation bearing elevation be recompacted with a vibratory compactor**, then observed and tested by a representative of PSI prior to concrete placement, to evaluate the suitability and uniformity of the bearing materials for support of the design foundation loads. Once the support soils are observed and tested, the concrete should be placed as quickly as possible to avoid exposure of the footing bottoms to wetting and drying. Surface run-off water should be drained away from the excavations and not be allowed to pond. The foundation concrete should be placed during the same day the excavation is made. If it is required that footing excavations be left open for more than one day, they should be protected to reduce evaporation or entry of moisture.

Newly placed engineered fill present below the bottom of the footing excavations should be evaluated by in-place field density tests during construction. The in-place field density may be evaluated on samples obtained by driving thin-wall Shelby tubes in the bottom of footing excavations to a minimum depth of 3 feet or 1 footing width, whichever is greater, below the base of the excavation. In the test probes, the fill density should be evaluated at the surface and every 12 inches for the entire probe depth. Engineered fill below footings should have an in-place density of at least 95% of maximum density and a moisture content within 3% of the optimum as determined by ASTM D 1557. An alternate method for evaluating the acceptability of the fill and an acceptable method to evaluate the natural fine-grained and granular soils under the footing would involve hand auger and static cone or dynamic cone penetrometer testing below the footing bearing level. Each isolated footing should include at least 1 test probe. Test probes should be performed every 20-lineal feet in continuous footings. Based on the recommended 3,000 psf net allowable bearing pressure, suitable bearing fine-grained or granular soils (native of compacted structural fill) should have a minimum dynamic cone penetrometer value commensurate with a Standard Penetration Test N-Value of 9 blows per foot based on the recommended 3,000 psf net allowable bearing capacity.

Where unsuitable bearing soils are encountered in a footing excavation, the excavation should be deepened to competent bearing soil, and the footing could be lowered or an overexcavation and backfill procedure could be performed. An overexcavation and backfill treatment would require widening the deepened excavation in all directions at least 6 inches beyond the edge of the footing for each 12 inches of overexcavation depth. The overexcavation should then be backfilled up to footing base elevation in maximum 8 inches thick loose lifts with suitable granular fill material compacted to at least 95 percent of maximum dry density and within 3% of the optimum moisture content as determined by modified Proctor, ASTM Designation D 1557.

As an alternative to supporting the footings at deeper elevations or on a new observed and tested compacted engineered fill, footings may also be designed to bear upon a lean concrete or controlled low strength material (CLSM) base founded upon suitable bearing natural soils as recommended above. If this option is chosen, the footing excavation should extend a minimum of 6 inches beyond each face of the footing.

Exterior footings and footings in unheated areas should be located at a depth of at least 48 inches below the final exterior grade to provide adequate frost protection. If the building is to be constructed during the winter months or if footings will likely be subjected to freezing temperatures after foundation construction, then the footings and concrete should be adequately protected from freezing.

Loading Dock Wall Design Considerations

The new loading docks on the east and west walls of the new structure will be required to resist lateral earth pressures. The actual earth pressure on the walls will vary according to material types and backfill materials used, how the backfill is compacted and the grade above the top of wall. If the below grade wall is restrained from movement in each direction, the at-rest condition applies. However, if the below grade wall is not restrained, then the active pressures would be applicable. The following design parameters are recommended:

PARAMETER	RECOMMENDED VALUE[3]
Backfill Unit Weight	125 pcf
"Active" Coefficient of Lateral Earth Pressure, K_a	0.33
"Active" Equivalent Fluid Pressure	42 psf/ft of depth
Coefficient of "Passive" Pressure, K_p[1]	3.0
"Passive" Equivalent Fluid Pressure	375 psf/ft of depth
"At-Rest" Coefficient of Lateral Earth Pressure, K_o	0.50
"At-Rest" Equivalent Fluid Pressure[2]	63 psf/ft of depth
Coefficient of Sliding	0.32

Notes: Ultimate passive pressure typically requires large strains to be fully mobilized, therefore, PSI recommends using 50% or less of the ultimate passive pressure to limit the strain on the structure. The values in the Table are ultimate values and do not include a factor of safety

The above values do not include the influence of foundation or surface loads in or adjacent to the wall backfill, or the effects of hydrostatic pressures. The magnitude of this and other surcharge loads, acting within the zone that begins at the base of a new foundation and extends upward and outward at a 1H: 1V ratio can be determined by multiplying the load by the appropriate lateral earth pressure coefficient.

Passive resistance should be neglected to a depth of four feet below exterior grade due to seasonal softening from freeze-thaw. In addition, the passive earth pressure values given above are based upon the concrete for the structure being placed in direct contact with the naturally deposited soils. If forms will be used to cast the concrete structure, fill material within the excavations surrounding the structure must be placed in layers that are less than eight inches (measured loose) and at a moisture content within three percent of the optimum moisture content determined by the modified Proctor compaction test (ASTM D1557). The fill material should be compacted to a minimum of 95% of the maximum dry density as determined by the modified Proctor test.

In order to intercept groundwater and limit lateral earth pressures, free-draining gravel or crushed stone backfill is recommended to be placed adjacent to the below-grade walls. The gravel or crushed stone backfill should have less than five percent passing the No. 200 sieve and a maximum particle size less than three inches. The width of the gravel or crushed stone layer should be equal to half the height of the below-grade wall or 4 feet, whichever is less. If clayey or silty soils are placed against or near the below-grade walls, or if the gravel or crushed stone layer is less than the recommended width or height, water-related and structural problems may develop. A one-foot-thick layer of relatively impervious clay is recommended to be placed above backfill that will be exposed to precipitation to minimize surface water infiltration. In addition, the ground surface should be sloped to drain surface water away from the structures.

PSI recommends that backfill directly behind the walls be compacted with light, hand-held compactors. Heavy compactors and grading equipment should not be allowed to operate within five to 10 feet of the walls during backfilling to avoid developing excessive temporary or long-term lateral soil pressures. PSI recommends that a representative of a qualified geotechnical engineer be present to monitor foundation excavations and fill placement.

Floor Slab Recommendations

The warehouse building floor slab could be supported upon the observed undocumented fill or native fine-grained or granular soils (based on the owner assuming the inherent risk of experiencing settlement related distress by utilizing existing undocumented fill for structural support of the new floor slab) that have been observed and tested, or newly placed compacted engineered fill provided the subgrade is prepared as outlined in the Site Preparation Section of this report. PSI has also recommended a settlement monitoring program for the southern half of the building pad that should be followed which will continue to lower the risk factor of experiencing settlement related distress in the floor slab.

PSI recommends that a subgrade modulus (k) of 150 pounds per cubic inch (pci) be used for design considerations based on a 12 inch diameter plate load test. However, depending on how the slab loads are applied, the value will have to be geometrically modified. The value should be adjusted for larger areas using the following expression for cohesive and cohesionless soil:

Modulus of Subgrade Reaction, $k_S = \left(\frac{k}{B}\right)$ for cohesive soil and

$k_S = k\left(\frac{B+1}{2B}\right)^2$ for cohesionless soil

where: k_S= coefficient of vertical subgrade reaction for loaded area,
k= coefficient of vertical subgrade reaction for 113 square inches area
B= width of area loaded, in feet

PSI recommends that a minimum four-inch thick free draining granular mat be placed beneath the floor slab to enhance drainage. Polyethylene sheeting should be placed to act as a vapor retarder where the floor will be in contact with moisture sensitive equipment or products such as tile, wood, carpet, etc., as directed by the design engineer. The decision to locate the vapor retarder in direct contact with the slab or beneath the layer of granular fill should be made by the design engineer after considering the moisture sensitivity of subsequent floor finishes, anticipated project conditions and the potential effects of slab curling and cracking. The floor slabs should have an adequate number of joints to reduce cracking resulting from differential movement and shrinkage.

Seismic Site Class

The 2009 International Building Code requires a site class for the calculation of earthquake design forces. This class is a function of soils type (i.e. depth of soil and strata types). Based on the estimated density of the soils observed within the Boring locations, **Site Class "C"** is recommended.

Pavement Recommendations

PSI understands that a new heavy duty-truck driveway and parking area is planned to the south, east and west of the proposed structure, while a light duty car parking lot is planned to the north of the proposed structure. Based upon the soils observed on site, PSI anticipated the subgrade soils within the pavement area to consist of native fine-grained and granular soils, fill soils consisting of silt with gravel or clay, or newly placed and compacted engineered fill. PSI recommends that the subgrade soils for the pavements be prepared in accordance with the Site Preparation section of this report.

A detailed traffic analysis was not performed as part of this exploration; however, based upon the proposed construction, the light and heavy duty pavement sections shown below are based on a 20 year design life. Specific traffic loading design details were not known at the time of this report. However, for the purpose of this analysis, a projected average traffic loading 300 tractor-trailer semis per week was estimated in heavy duty areas and a total of 30,000 equivalent 18,000 pound single axle loads (ESAL) was estimated in light duty parking areas. When traffic loading details are finalized, they must be discussed with PSI to determine if a re-evaluation of the recommendations contained herein is warranted. The existing soils encountered below the surficial topsoil and pavement materials are considered fair to poor subgrade materials, having a minimum CBR value of 3 according to the Wisconsin Asphalt Pavement Association Design Guide. Engineered fill material used to raise existing grades within parking and drive areas should meet or exceed this CBR value.

The following design factors were used in developing the recommended pavement sections:

- Design Life: 20 years
- Design Traffic (Heavy Duty): 1,270,664 ESALs (rigid); 736,110 ESALs (flexible)
- Design Traffic (Light Duty): 30,000 ESAL
- Resilient Modulus (M_R): 3,000 psi
- Modulus of Subgrade Reaction: 125 pci
- Reliability: 85%
- Initial Serviceability: 4.5 (rigid); 4.2 (flexible)

- Terminal Serviceability: 2.0
- Standard Deviation: 0.35 (rigid), 0.45 (flexible)
- Load Transfer Coefficient J: 3.2
- Concrete Modulus of Rupture: 600 psi
- Structural Coefficient Hot Mix Asphalt: 0.44
- Structural Coefficient Aggregate Base: 0.14

If during the final design phase these values are determined to be incorrect, PSI must be contacted to provide revised pavement recommendations. Based upon the soil Borings, laboratory data and provided the subgrade soils are prepared as outlined in this report, the following flexible pavement section is recommended for parking stalls (light duty) and drive lanes for heavy trucks (heavy duty).

Light Duty Asphalt Pavement Section-Automobile Parking Lot

Granular Base Course Thickness	8 inches*
HMA Thickness	3 ¼ inches

***If front end loader is used for snow removal, a BX1200 should be placed below the light duty base course layer or an additional 2 inches of base course added to the recommended section**

Heavy Duty Asphalt Pavement Section

Granular Base Course Thickness	9 inches
HMA Thickness	5 inches

Heavy Duty Concrete Pavement Section

Granular Base Course Thickness	7 inches
Minimum 4,000 psi Concrete Thickness	7 inches

The granular base course should consist of well-graded crushed stone meeting the requirements from Section 305 of the State of Wisconsin Standard Specifications for Construction for a 1¼" dense graded base. The granular base course material should be placed and compacted to a minimum of 95% of maximum density as determined by ASTM D 1557 (modified Proctor) and within +/-3% of the optimum moisture content value. Also, a representative of a qualified geotechnical engineer must test the base course material prior to, and during, placement.

Asphaltic binder and surface courses should meet the requirements from Section 460 of the State of Wisconsin Standard Specifications for Construction. Asphaltic courses should be placed and compacted to the minimum required density contained within section 460 of the Standard Specifications. An adequate number of in-place density tests should be performed during construction to document the placement compaction.

The pavements should be sloped to provide positive surface drainage. Water should not be allowed to pond on or adjacent to the pavement as this could saturate the subgrade and cause premature pavement deterioration. The granular base course should be protected from water inflow along drainage paths. Additionally, the granular base course should extend beyond the edges of the pavement in low areas to allow any water that enters the base course stone a path for exit.

The parking areas are recommended to be constructed with attention to final grades to facilitate drainage. Otherwise, a storm sewer system may be appropriate to carry away storm run-off water. Construction of the subgrade and pavements should be in accordance with the project specifications.

A flexible pavement system is not recommended in dumpster pad areas and areas where heavy trucks will turn frequently or will be parked. Within these areas, consideration should be given for use of a rigid pavement. The concrete must be properly designed to withstand large point loads incurred by truck tires.

Subgrade Drainage Recommendations

In order to achieve the full design life of the pavement, PSI recommends subsurface drains be installed. If placed properly, subsurface drains will greatly the amount of trapped water under the pavement surfaces. Trapped water leads to subgrade degradation and increases pavement heave during winter months.

Minimally, these drains should be placed in low spots in the pavement, at the toe of slopes that are draining toward pavement surfaces, under landscape islands and where undercuts have been filled with granular fill and as finger drains extending for a distance of at least 10 feet from the edge of catch basins. The drain system should consist of minimum three-inch-diameter perforated drainpipes surrounded by at least 6 inches of clean crushed ¾” to 1” limestone. The granular fill should be filter protected by wrapping the clean stone fill in a 6 oz. non-woven geotextile filter fabric. The top of the draintile trench should coincide with the pavement base course layer. Additionally, the draintile should be installed with a positive slope (Minimum ½%-1%) throughout the length of the tile. The drains should connect to the nearest storm sewer catch basin.

CONSTRUCTION CONSIDERATIONS

PSI should be retained to provide observation and testing of construction activities involved in the foundation, earthwork, and related activities of this project. PSI will not accept any responsibility for any conditions that deviated from those described in this report, nor for the performance of the foundation or pavement if we are not engaged to also provide construction observation and testing for this project.

Moisture Sensitive Soils/Weather-Related Concerns

The soils encountered at this site are expected to be sensitive to disturbances caused by construction traffic and changes in moisture content. Increases in the moisture content of the soil can cause significant reduction in the soil strength and support capabilities. In addition, soils that become wet may be slow to dry and thus significantly retard the progress of grading and compaction activities. It will, therefore, be advantageous to perform earthwork and foundation construction activities during dry weather.
Water should not be allowed to collect in the foundation excavation, on floor slab or pavement areas, or on prepared subgrades during or after construction. Areas should be sloped to facilitate removal of collected rainwater, groundwater, or surface runoff. Positive site drainage should be provided to reduce infiltration of surface water around the perimeter of buildings, beneath floor slabs, and within pavement areas. The grades should be sloped away from buildings and surface drainage should be collected and discharged such that water is not permitted to infiltrate the backfill and floor slab areas of the building.

Drainage and Groundwater Concerns

The groundwater level at B-10 through B-12 was at depths of about 16 to 36 feet (96 to 117± feet) at the time of the exploration. For the most part, based upon these observations, groundwater-related problems are not anticipated for the proposed construction if the footings are placed at standard frost depths. If minor groundwater seepage is encountered during excavation, it is anticipated that it can be handled by simple means such as pumping from sumps or the use of perimeter trenches to collect and discharge the water away from the work area. However, the use of high capacity sump pumps, with sufficient lifting capacity, may be required for excavations which encroach upon or extend below the groundwater to facilitate construction where Geopiers are installed. The use of temporary casing may be necessary for Geopier construction.

Fluctuations in the groundwater level should be anticipated throughout the year depending on variations in climatological conditions and other factors not apparent at the time the borings were performed. The possibility of groundwater level fluctuation and perched water conditions should be considered when developing the design and construction plans for the project.

Excavations

It is mandated that excavations, whether they be for utility trenches, basement excavations or footing excavations, be constructed in accordance with current Occupational Safety and Health Administration (OSHA) guidelines to protect workers and others during construction. PSI recommends that these regulations be strictly enforced; otherwise, workers could be in danger and the owner(s) and the contractor(s) could be liable for substantial penalties.

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor's "responsible person", as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations.

Sloping, shoring or bracing of the excavation sidewalls will be necessary. Trenching in fill and granular soils may be difficult due to the instability of vertical slopes, and will therefore require a flattening of trench sides, or some other means of protection, to facilitate construction and to protect life and property. Substantial sloughing and caving should be expected within unprotected excavations. Temporary casing, and possibly the use of drilling mud is expected to be required to maintain stability for foundations extending below the groundwater and into wet granular soils. The degree of excavation instability problems is dependent upon the depth and length of time that excavations remain open, excavation bank slopes, water levels and the effectiveness of any dewatering systems. However, severe instability can be expected within fill and granular soils, especially encroaching upon and extending below the groundwater. All excavation work must be performed in accordance with OSHA and local building code requirements.

Where excavations encroach upon or extend below the groundwater or perched zones and into fine sand, silt, soft clay, fill, or organics, they may become substantially unstable when the confining effect of the overburden is removed. Significant sloughing or caving of sidewalls may also occur. Some overexcavation of softened or loosened soils to expose suitable underlying natural soils, in conjunction with the use of a crushed stone working mat, may be necessary to establish a stable bearing subgrade. Additionally, significantly widened excavations may result, or be required to maintain or achieve sidewall stability.

PSI is providing this information solely as a service to our client. PSI does not assume responsibility for construction site safety or the contractor's or other parties' compliance with local, state, and federal safety or other regulations.

Utilities Trenching and Backfilling

Excavation for utility trenches shall be performed in accordance with OSHA regulations as stated in 29 CFR Part 1926. It should be noted that utility trench excavations have the potential to degrade the properties of the adjacent fill materials. Utility trench walls that are allowed to move laterally can lead to reduced bearing capacity and increased settlement of adjacent structural elements and overlying slabs.

Backfill for utility trenches is as important as the original subgrade preparation or structural fill placed to support either a foundation or slab. Therefore, it is imperative that the backfill for utility trenches be placed to meet the project specifications for the structural fill of this project. Unless otherwise specified, the backfill for the utility trenches should be placed in 4 to 6 inch loose lifts and compacted to a minimum of 95%

of the maximum dry density achieved by the modified Proctor test. The backfill soil should be moisture conditioned to be within 3% of the optimum moisture content as determined by the modified Proctor test. Up to 4 inches of bedding material placed directly under the pipes or conduits placed in the utility trench can be compacted to the 90% compaction criteria with respect to the modified Proctor. Compaction testing should be performed for every 200 cubic yards of backfill placed or each lift within 200 linear feet of trench, which ever is less. Backfill of utility trenches should not be performed with water standing in the trench. If granular material is used for the backfill of the utility trench, the granular material should have a gradation that will filter protect the backfill material from the adjacent soils. If this gradation is not available, a geosynthetic non-woven filter fabric should be used to reduce the potential for the migration of fines into the backfill material. Granular backfill material shall be compacted to meet the above compaction criteria. The geotechnical engineer can also specify a relative density specification for clean granular materials. The granular backfill material should be compacted to achieve a relative density greater than 75% or as specified by the geotechnical engineer for the specific material used.

GEOTECHNICAL RISK

The concept of risk is an important aspect of the geotechnical evaluation. The primary reason for this is that the analytical methods used to develop geotechnical recommendations do not comprise an exact science. The analytical tools which geotechnical engineers use are generally empirical and must be used in conjunction with engineering judgment and experience. Therefore, the solutions and recommendations presented in the geotechnical evaluation should not be considered risk-free and, more importantly, are not a guarantee that the interaction between the soils and the proposed structure will perform as planned. The engineering recommendations presented in the preceding section constitutes PSI’s professional estimate of those measures that are necessary for the proposed structure to perform according to the proposed design based on the information generated and referenced during this evaluation, and PSI’s experience in working with these conditions.

REPORT LIMITATIONS

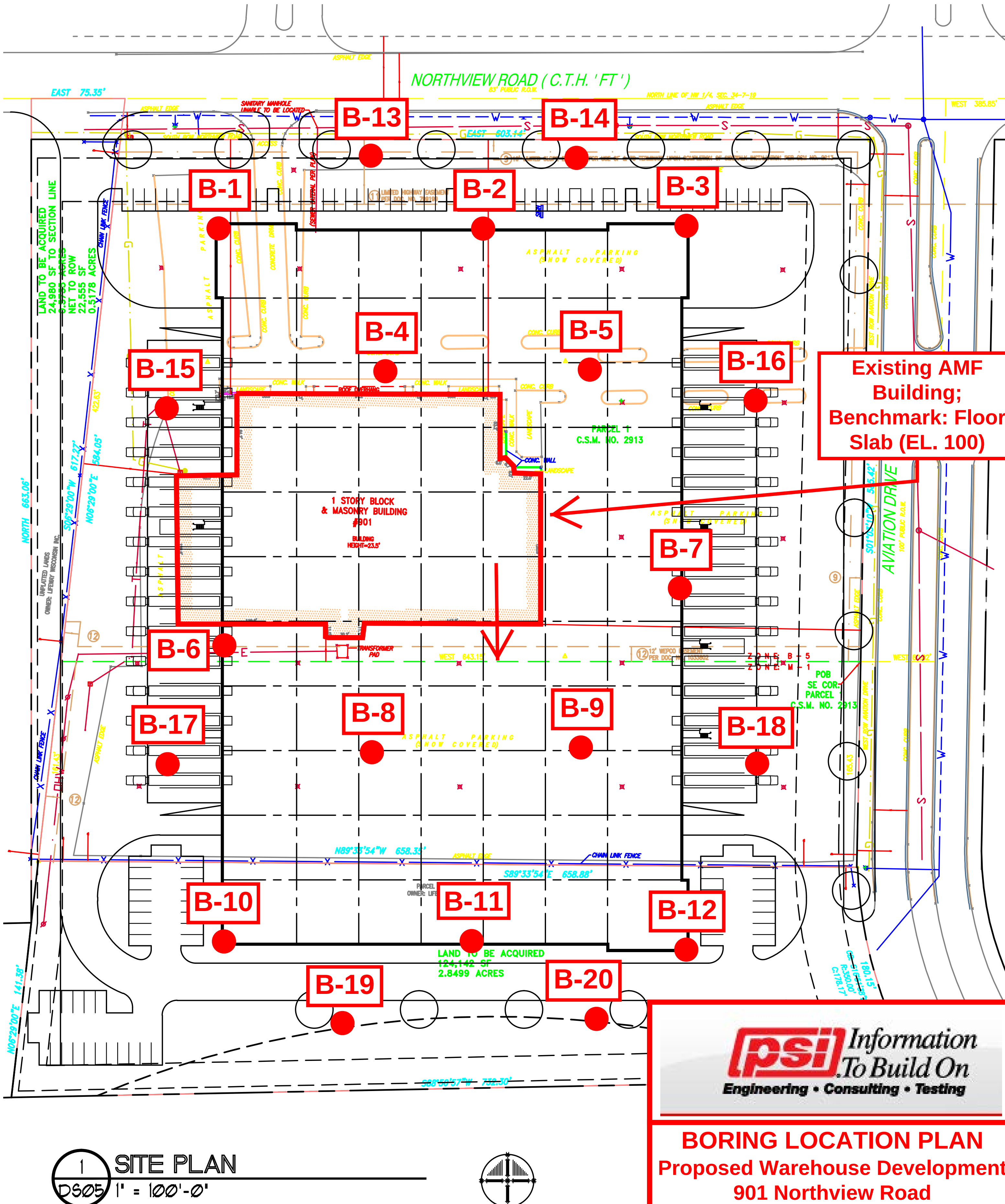
PSI’s recommendations are based on the available subsurface information obtained by PSI and design details furnished by others. If there are any revisions to the plans for this project or if deviations from the subsurface conditions noted in this report are encountered during construction, PSI must be notified immediately to determine if changes in the recommendations are required. If PSI is not retained to perform these functions, PSI will not be responsible for the impact of those conditions on the project.

PSI warrants that the findings, recommendations, specifications, or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed.

After the plans and specifications are complete, PSI must be retained and provided the opportunity to review the final design plans and specifications to check that our engineering recommendations have been properly incorporated into the design documents. At this time, it may be necessary to submit supplementary recommendations. This report has been prepared for the exclusive use by Premier Design Build Group for the proposed Industrial Development to be located at 901 Northview Road, in Waukesha, Wisconsin.

APPENDIX

BORING LOCATION PLAN
LOG OF BORINGS
GENERAL NOTES



Information To Build On
Engineering • Consulting • Testing

BORING LOCATION PLAN

Proposed Warehouse Development

901 Northview Road

Waukesha, WI 53188

PSI Project No. 00521212

*Adapted from site plan provided by client.

StormTech

Tables 8 and **9** provide cumulative storage volumes for the MC-4500 chamber and end cap. These tables can be used to calculate the stage-storage relationship for the retention or detention system. Digital spreadsheets in which the number of chambers and end caps can be input for quick cumulative storage calculations are available at www.stormtech.com. For assistance with site-specific calculations or input into routing software, contact the StormTech Technical Services Department.

TABLE 8 – MC-4500 Incremental Storage Volume Per Chamber

Assumes 40% stone porosity. Calculations are based upon a 9" (229 mm) stone base under the chambers, 12" (305 mm) of stone above chambers, and 9" (229 mm) spacing between chambers.

	Depth of Water in System Inches (mm)	Cumulative Chamber Storage ft³ (m³)	Total System Cumulative Storage ft³ (m³)
TOTAL SYSTEM VOLUME	81 (2057)	0	162.62 (4.605)
	80 (2032)	0	161.40 (4.570)
	79 (2007)	Stone 0	160.18 (4.536)
	78 (1981)	Cover 0	158.96 (4.501)
	77 (1956)	0	157.74 (4.467)
	76 (1930)	0	156.52 (4.432)
	75 (1905)	0	155.30 (4.398)
	74 (1880)	0	154.09 (4.363)
	73 (1854)	0	152.87 (4.329)
	72 (1829)	0	151.65 (4.294)
	71 (1803)	0	150.43 (4.260)
	70 (1778)	0	149.21 (4.225)
	69 (1753)	106.51 (3.016)	147.99 (4.191)
	68 (1727)	106.47 (3.015)	146.75 (4.156)
	67 (1702)	106.35 (3.012)	145.46 (4.119)
	66 (1676)	106.18 (3.007)	144.14 (4.082)
	65 (1651)	105.98 (3.001)	142.80 (4.044)
	64 (1626)	105.71 (2.993)	141.42 (4.005)
	63 (1600)	105.25 (2.981)	139.93 (3.962)
	62 (1575)	104.59 (2.962)	138.31 (3.917)
	61 (1549)	103.79 (2.939)	136.61 (3.869)
	60 (1524)	102.88 (2.913)	134.85 (3.819)
	59 (1499)	101.88 (2.885)	133.03 (3.767)
	58 (1473)	100.79 (2.854)	131.16 (3.714)
	57 (1448)	99.63 (2.821)	129.24 (3.660)
	56 (1422)	98.39 (2.786)	127.28 (3.604)
	55 (1397)	97.10 (2.749)	125.28 (3.548)
	54 (1372)	95.73 (2.711)	123.25 (3.490)
	53 (1346)	94.32 (2.671)	121.18 (3.431)
	52 (1321)	92.84 (2.629)	119.08 (3.372)
3.5' FULL VOLUME	51 (1295)	91.32 (2.586)	116.94 (3.311)
	50 (1270)	89.74 (2.541)	114.78 (3.250)
	49 (1245)	88.12 (2.495)	112.59 (3.188)
	48 (1219)	86.45 (2.448)	110.37 (3.125)
	47 (1194)	84.75 (2.400)	108.13 (3.062)
	46 (1168)	83.00 (2.350)	105.86 (2.998)
3' FULL VOLUME	45 (1143)	81.21 (2.300)	103.56 (2.933)
	44 (1118)	79.38 (2.248)	101.25 (2.867)
	43 (1092)	77.52 (2.195)	98.91 (2.801)
	42 (1067)	75.62 (2.141)	96.55 (2.734)
	41 (1041)	73.69 (2.087)	94.18 (2.667)
	40 (1016)	71.72 (2.031)	91.78 (2.599)
	39 (991)	69.73 (1.974)	89.36 (2.531)
	38 (965)	67.70 (1.917)	86.93 (2.462)
	37 (948)	65.65 (1.859)	84.48 (2.392)
	36 (914)	63.57 (1.800)	82.01 (2.322)
	35 (889)	61.46 (1.740)	79.53 (2.252)
	34 (864)	59.32 (1.680)	77.03 (2.181)
	33 (838)	57.17 (1.619)	74.52 (2.110)
	32 (813)	54.98 (1.557)	71.99 (2.038)
	31 (787)	52.78 (1.495)	69.45 (1.966)
	30 (762)	50.55 (1.431)	66.89 (1.894)
	29 (737)	48.30 (1.368)	64.32 (1.821)
	28 (711)	46.03 (1.303)	61.74 (1.748)
	27 (686)	43.74 (1.239)	59.15 (1.675)
	26 (680)	41.43 (1.173)	56.55 (1.601)
	25 (610)	39.11 (1.107)	53.93 (1.527)
	24 (609)	36.77 (1.041)	51.31 (1.453)
	23 (584)	34.41 (0.974)	48.67 (1.378)
	22 (559)	32.03 (0.907)	46.03 (1.303)
	21 (533)	29.64 (0.839)	43.38 (1.228)
	20 (508)	27.23 (0.771)	40.71 (1.153)
	19 (483)	24.81 (0.703)	38.04 (1.077)
	18 (457)	22.38 (0.634)	35.37 (1.001)
	17 (432)	19.94 (0.565)	32.68 (0.925)
	16 (406)	17.48 (0.495)	29.99 (0.849)
	15 (381)	15.01 (0.425)	27.29 (0.773)
	14 (356)	12.53 (0.355)	24.58 (0.696)
	13 (330)	10.05 (0.284)	21.87 (0.619)
	12 (305)	7.55 (0.214)	19.15 (0.542)
	11 (279)	5.04 (0.143)	16.43 (0.465)
	10 (254)	2.53 (0.072)	13.70 (0.388)
	9 (229)	0	10.97 (0.311)
	8 (203)	0	9.75 (0.276)
	7 (178)	0	8.53 (0.242)
	6 (152)	Stone 0	7.31 (0.207)
	5 (127)	Foundation 0	6.09 (0.173)
	4 (102)	0	4.87 (0.138)
	3 (76)	0	3.66 (0.104)
	2 (51)	0	2.44 (0.069)
	1 (25)	0	1.22 (0.035)

NOTE: Add 1.22 ft³ (0.035 m³) of storage for each additional inch (25 mm) of stone foundation. Contact StormTech for cumulative volume spreadsheets in digital format.

TABLE 9 – MC-4500 Incremental Storage Volume Per End Cap

Assumes 40% stone porosity. Calculations are based upon a 9" (229 mm) stone base under the end caps, 12" (305 mm) of stone above end caps, 9" (229 mm) of spacing between end caps and 12" (305 mm) of stone perimeter.

Depth of Water in System Inches (mm)	Cumulative Chamber Storage ft³ (m³)	Total System Cumulative Storage ft³ (m³)
81 (2057)	0	108.69 (3.078)
80 (2032)	0	107.62 (3.047)
79 (2007)	Stone 0	106.54 (3.017)
78 (1981)	Cover 0	105.46 (2.986)
77 (1956)	0	104.38 (2.956)
76 (1930)	0	103.31 (2.925)
75 (1905)	0	102.23 (2.895)
74 (1880)	0	101.15 (2.864)
73 (1854)	0	100.07 (2.834)
72 (1829)	0	99.00 (2.803)
71 (1803)	0	97.92 (2.773)
70 (1778)	0	96.84 (2.742)
69 (1753)	35.71 (1.011)	95.76 (2.712)
68 (1727)	35.71 (1.011)	94.69 (2.681)
67 (1702)	35.70 (1.011)	93.60 (2.651)
66 (1676)	35.67 (1.010)	92.51 (2.620)
65 (1651)	35.62 (1.009)	91.40 (2.588)
64 (1626)	35.56 (1.007)	90.29 (2.557)
63 (1600)	35.47 (1.004)	89.16 (2.525)
62 (1575)	35.36 (1.001)	88.01 (2.492)
61 (1549)	35.21 (0.997)	86.85 (2.459)
60 (1524)	35.05 (0.992)	85.67 (2.426)
59 (1499)	34.86 (0.987)	84.48 (2.392)
58 (1473)	34.64 (0.981)	83.27 (2.358)
57 (1448)	34.40 (0.974)	82.05 (2.323)
56 (1422)	34.13 (0.966)	80.81 (2.288)
55 (1397)	33.83 (0.958)	79.55 (2.253)
54 (1372)	33.51 (0.949)	78.28 (2.217)
53 (1346)	33.16 (0.939)	77.00 (2.180)
52 (1321)	32.79 (0.928)	75.70 (2.144)
51 (1295)	32.39 (0.917)	74.38 (2.106)
50 (1270)	31.98 (0.906)	73.06 (2.069)
49 (1245)	31.54 (0.893)	71.71 (2.031)
48 (1219)	31.07 (0.880)	70.36 (1.992)
47 (1194)	30.59 (0.866)	68.99 (1.954)
46 (1168)	30.09 (0.852)	67.61 (1.915)
45 (1143)	29.56 (0.837)	66.22 (1.875)
44 (1118)	29.02 (0.822)	64.81 (1.835)
43 (1092)	28.45 (0.806)	63.40 (1.795)
42 (1067)	27.87 (0.789)	61.97 (1.755)
41 (1041)	27.27 (0.772)	60.53 (1.714)
40 (1016)	26.65 (0.755)	59.08 (1.673)
39 (991)	26.01 (0.736)	57.62 (1.632)
38 (965)	25.35 (0.718)	56.15 (1.590)
37 (948)	24.68 (0.699)	54.67 (1.548)
36 (914)	23.99 (0.679)	53.18 (1.506)
35 (889)	23.28 (0.659)	51.68 (1.463)
34 (864)	22.56 (0.639)	50.17 (1.421)
33 (838)	21.82 (0.618)	48.64 (1.377)
32 (813)	21.06 (0.596)	47.11 (1.334)
31 (787)	20.29 (0.575)	45.57 (1.290)
30 (762)	19.50 (0.552)	44.02 (1.247)
29 (737)	18.70 (0.530)	42.46 (1.202)
28 (711)	17.88 (0.506)	40.89 (1.158)
27 (686)	17.04 (0.483)	39.31 (1.113)
26 (680)	16.19 (0.459)	37.73 (1.068)
25 (610)	15.33 (0.434)	36.14 (1.023)
24 (609)	14.46 (0.410)	34.53 (0.978)
23 (584)	13.58 (0.384)	32.93 (0.932)
22 (559)	12.68 (0.359)	31.31 (0.887)
21 (533)	11.77 (0.333)	29.69 (0.841)
20 (508)	10.85 (0.307)	28.06 (0.794)
19 (483)	9.91 (0.281)	26.42 (0.748)
18 (457)	8.97 (0.254)	24.77 (0.702)
17 (432)	8.01 (0.227)	23.12 (0.655)
16 (406)	7.04 (0.199)	21.46 (0.608)
15 (381)	6.07 (0.172)	19.80 (0.561)
14 (356)	5.08 (0.144)	18.13 (0.513)
13 (330)	4.08 (0.116)	16.45 (0.466)
12 (305)	3.07 (0.087)	14.77 (0.418)
11 (279)	2.06 (0.058)	13.09 (0.371)
10 (254)	1.03 (0.029)	11.39 (0.323)
9 (229)	0	9.70 (0.275)
8 (203)	0	8.62 (0.244)
7 (178)	0	7.54 (0.214)
6 (152)	Stone 0	6.46 (0.183)
5 (127)	Foundation 0	5.39 (0.153)
4 (102)	0	4.31 (0.122)
3 (76)	0	3.23 (0.092)
2 (51)	0	2.15 (0.061)
1 (25)	0	1.08 (0.031)

NOTE: Add 1.08 ft³ (0.031 m³) of storage for each additional inch (25 mm) of stone foundation. Contact stormtech for cumulative volume spreadsheets in digital format.