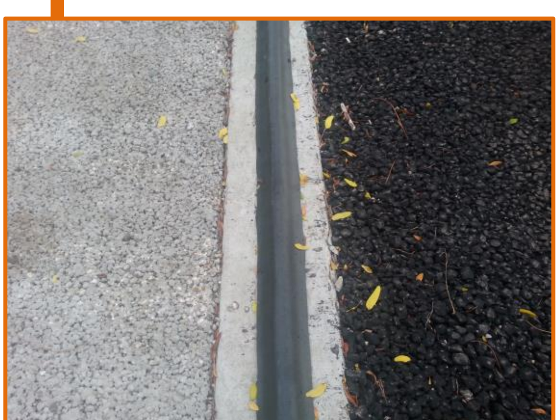
STORMWATER MANAGEMENT PLAN



MANDEL DEVELOPMENT

City of Waukesha, Waukesha County, Wisconsin

PEG Project Number: 1773.00A-WI

July 19, 2024

PINNACLE ENGINEERING GROUP
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Questions and comments can be directed to:

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INTRODUCTION

The proposed project is a multifamily development located west of the intersection of Delafield Street and Buena Vista Avenue. The site is approximately 5.0 acres in size. A location map illustrates the tract of land included in **Appendix 1**. The City and the DNR have regulatory authority over the site in terms of stormwater management and specific associated requirements. Pinnacle Engineering has completed a plan to meet these requirements.

DESIGN CRITERIA

City of Waukesha:.. Chapter 32

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

Water Quantity: City of Waukesha Stormwater Submittal Standards, Retention/Detention Requirements – Peak Discharge. To minimize downstream bank erosion and the failure of downstream conveyance systems, the calculated post-development peak storm water discharge rate shall not exceed the calculated pre-development discharge rates for the 1-year, 2-year, 10-year, and 100-year, 24-hour design storms. In this case, the proposed impervious area is nearly identical to the existing and thus there is no increase in peak flows. This is illustrated in the report.

Water Quality: *WDNR NR 151.122* – This site was previously developed with existing buildings and parking lots. The site is designated as a re-development and thus is required to remove 40% of the total suspended solids (TSS) load on an average annual basis.

Infiltration: WDNR NR 151.124 and City Ordinance 32.10.d.3 – Infiltration is required except in cases where native subsoils are not conducive of infiltration (less than 0.6 in /hr).

Protective Areas: WDNR NR 151.125 – Protective areas are required where impervious surfaces drain directly to wetlands.

PRE-DEVELOPMENT CONDITIONS

The existing site currently is occupied by an existing building and large areas of parking lots. There was previously a second and third building located on the site which were demolished at some point between 2017 and 2020 per historical aerial images from the Waukesha GIS. Drainage is tributary to the Fox River to the east. Soils on the site are classified as Hochheim loam on the USDA soil survey which are Group D soils. The USDA Soil Survey Map can be found in **Appendix 1.** A Post-Development Hydrology Exhibit can be found in **Appendix 2**.

POST-DEVELOPMENT CONDITIONS

The proposed development consists of two buildings, both of which includes underground parking, associated driveways, parking areas, and utilities. Stormwater for the site is managed through a combination of a proposed underground stormwater sewer system, catch basins, a bioretention basin, and diverting off-site stormwater that flows through the site to existing storm structures located on Delafield Street.

A Post-Development Hydrology Exhibit can be found in **Appendix 3**.

ANALYSIS METHODS

TSS reduction characteristics for the proposed water quality facilities were determined using WinSLAMM® (Version 10.4.1) Source Loading and Management Model.

HydroCAD® (Version 10.00) software has been used to analyze stormwater characteristics for this stormwater management plan. HydroCAD uses the accepted TR-55 methodology for determining peak discharge runoff rates. Rainfall depths for the 1, 2, 10 and 100-year storm events are 2.40, 2.70, 3.81 and 6.18 inches in accordance with NOAA Atlas 14, Volume 8, Version 2, Appendix 1.

SUMMARY OF RESULTS

Water Quality SLAMM Model Summary

Area/Pond	Pounds of TSS Generated	Pounds of TSS Remaining	Percent Removal
Rain Garden	143	59	59%
Catch Basins	212	154	27%
Site Total	355	213	40%

The modeling indicates that the stormwater ponds will provide TSS removal to meet the DNR and City ordinance.

Peak Flows

Area	Area (ac)	CN	Tc (min)	1-year (cfs)	2-year (cfs)	10-year (cfs)	100-year (cfs)
EXISTING SITE	5.0	88	6.0*	**11.6**	**13.8**	**22.2**	**40.2**
PROPOSED SITE	5.0	88	6.0*	**11.6**	**13.8**	**22.2**	**40.2**

**A Tc of 6.0 min is used as the actual computed Tc is less than the minimum of 6 min per TR 55.*

Infiltration

This project has underlying soils that consist of predominantly a mix of fill, sandy silt and clay. While some sandier soil is present, there are intermixed layers of silt and clay in between and under the sandy silts, which creates a trapped “bathtub” effect, not allowing the soils to truly infiltrate. Based on this, the site will not support infiltration goals, and is therefore 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, there are no wetlands, therefore, the protective areas do not apply.

Overland Flow Path from Bioretention Basin

In the event of the existing storm sewer in Delafield Street surcharging, an analysis was run using the 100-year storm flows from the site to find out how much water would build up if both the rain garden and the storm sewer were unable to accept water. Two ridge points were used to analyze this (see **Appendix 5**), Overflow 1 was analyzed as a weir and Overflow 2 was analyzed as a channel. Both were analyzed using Hydraflow Express, the data from the Stormwater Calculations model, and the delineated proposed drainage areas which include both onsite and offsite areas. The depth of water collected at Overflow 1 was 0.29 ft (approximately 3.5 inches) and the depth at Overflow 2 was 0.42 ft (approximately 5 inches). Since both of these depths are below 6 inches, in an event of a 100 year flood where the storm sewer is unable to take any flow, the water is likely not to top over the curb.

CONCLUSION

The stormwater management features for the development have been designed to comply with the City of Waukesha ordinance and WDNR technical standards NR216/151. Runoff will be treated in the rain garden to remove total suspended solids as well as in the catch basins further downstream. Infiltration, detention, and protective areas are not required. Maintenance is expected to occur on a regular basis. An agreement with the City of Waukesha will be executed to ensure this occurs.

APPENDIX 1
MAPS

PINNACLE ENGINEERING GROUP

MAP WAUKESHA county

Waukesha County GIS Map

Legend

- Municipal Boundary_2K
- Parcel_Dimension_2K
- Note_Text_2K
- Lots_2K
  - Lot
  - Unit
  - General Common Element
  - Outlot
- SimultaneousConveyance
  - Assessor Plat
  - CSM
  - Condominium
  - Subdivision
- Cartoline_2K
  - EA-Easement_Line
  - PL-DA
  - PL-Extended_Tie_line
  - PL-Meander_Line
  - PL-Note
  - PL-Tie
  - PL-Tie_Line
  - <all other values>
- Railroad_2K

0 221.29 Feet

The information and depictions herein are for informational purposes and Waukesha County specifically disclaims accuracy in this reproduction and specifically admonishes and advises that if specific and precise accuracy is required, the same should be determined by procurement of certified maps, surveys, plats, Flood Insurance Studies, or other official means. Waukesha County will not be responsible for any damages which result from third party use of the information and depictions herein, or for use which ignores this warning.

Notes:

Printed: 3/18/2024

Hydrologic Soil Group—Milwaukee and Waukesha Counties, Wisconsin
(Mandel Delafield Street)

Map Scale: 1:2,080 if printed on A portrait (8.5" x 11") sheet.

Meters: 0, 30, 60, 120, 180

Feet: 0, 100, 200, 400, 600

N

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 Rating Polygons

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Lines

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Points

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

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:
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 19, Sep 8, 2023

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

Date(s) aerial images were photographed: Jul 30, 2022—Sep 13, 2022

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.

Hydrologic Soil Group

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
HmB2	Hochheim loam, 2 to 6 percent slopes, eroded	D	2.4	40.6%
HmC2	Hochheim loam, 6 to 12 percent slopes, eroded	D	2.6	44.5%
HmD2	Hochheim loam, 12 to 20 percent slopes, eroded	D	0.9	14.9%
Totals for Area of Interest			**5.9**	**100.0%**

Description

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The soils in the United States are assigned to four groups (A, B, C, and D) and three dual classes (A/D, B/D, and C/D). The groups are defined as follows:

Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.

Rating Options

Aggregation Method: Dominant Condition

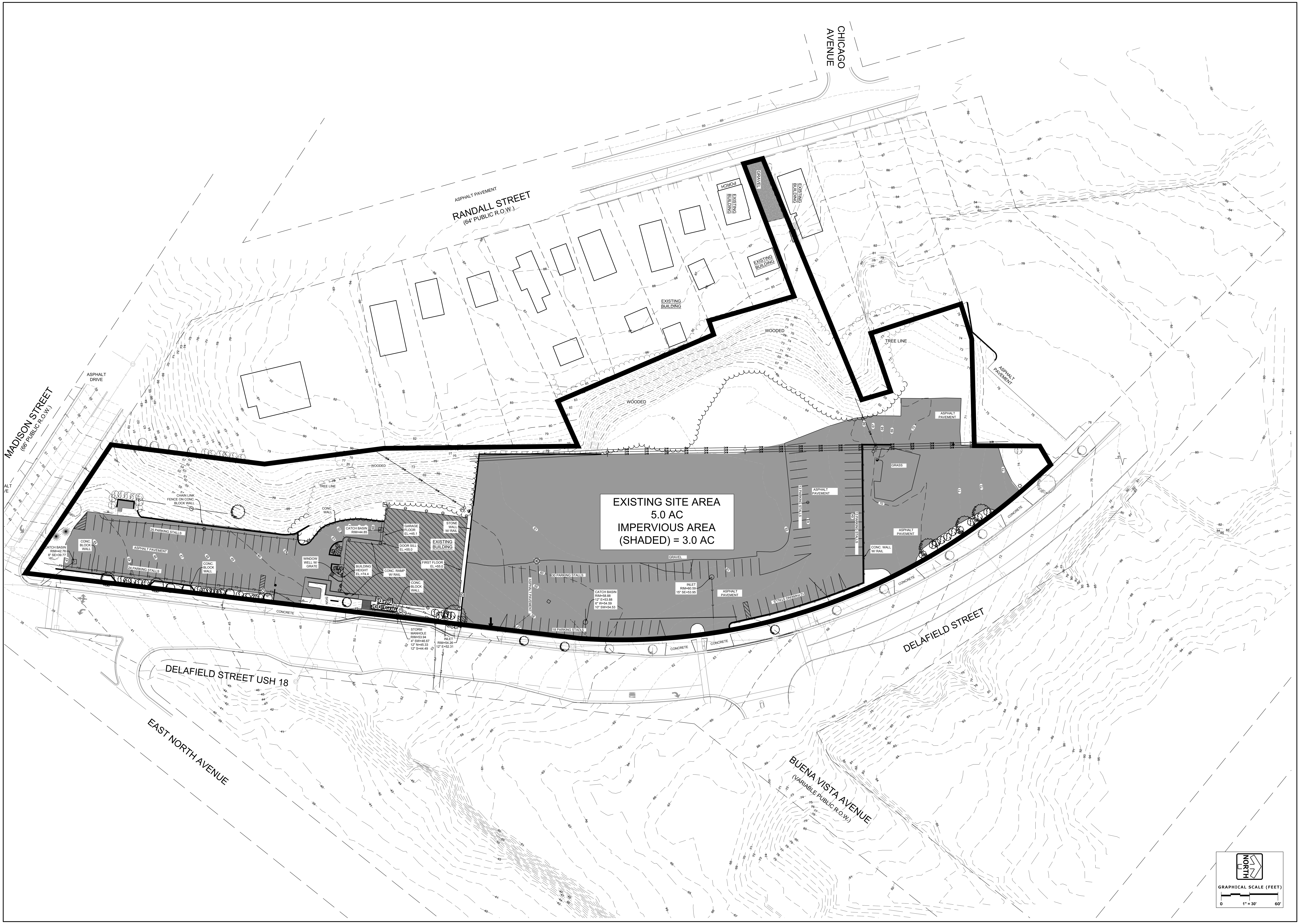
Component Percent Cutoff: None Specified

Tie-break Rule: Higher

IMPERVIOUS AREA AS OF 2017

APPENDIX 2
PRE-DEVELOPMENT CONDITIONS

PINNACLE ENGINEERING GROUP



MANDEL DELAFIELD ST DEVELOPMENT EXISTING HYDROLOGY EXHIBIT

07/09/2024

PINNACLE ENGINEERING GROUP

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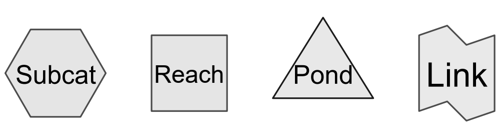
PLAN | DESIGN | DELIVER



Routing Diagram for PRELIM
Prepared by Pinnacle Engineering Group, Printed 7/18/2024

Summary for Subcatchment EX: EXISTING SITE

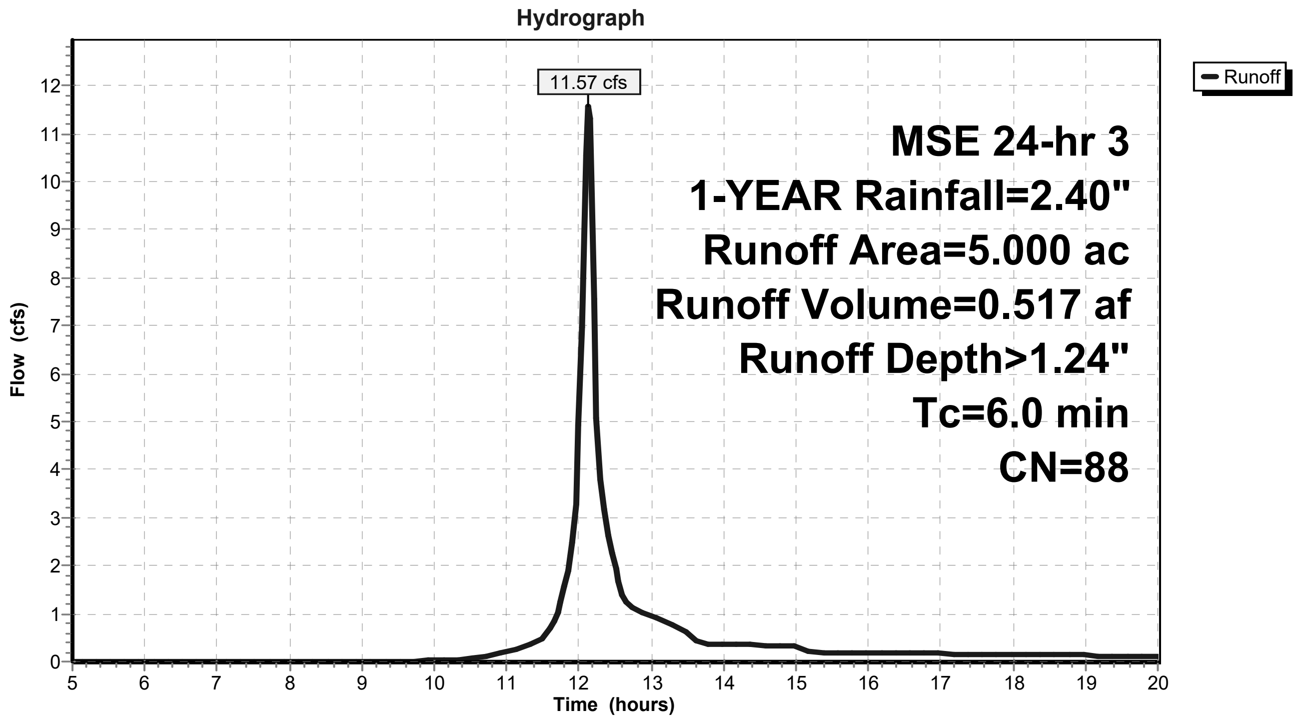
Runoff = 11.57 cfs @ 12.13 hrs, Volume= 0.517 af, Depth> 1.24"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
MSE 24-hr 3 1-YEAR Rainfall=2.40"

	Area (ac)	CN	Description
*	3.000	98	EXISTING IMPERVIOUS
	1.000	74	>75% Grass cover, Good, HSG C
	1.000	70	Woods, Good, HSG C
	5.000	88	Weighted Average
	2.000		40.00% Pervious Area
	3.000		60.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry, MIN PER TR-55**

Subcatchment EX: EXISTING SITE



Summary for Subcatchment EX: EXISTING SITE

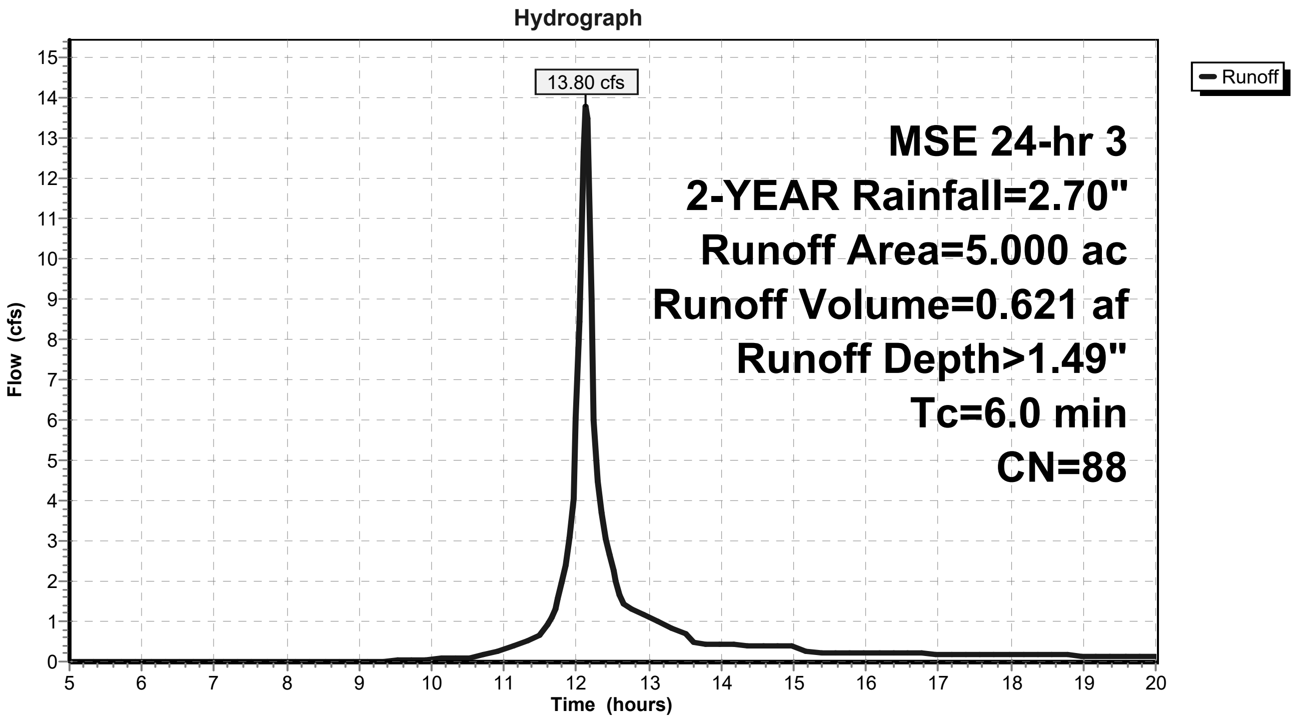
Runoff = 13.80 cfs @ 12.13 hrs, Volume= 0.621 af, Depth> 1.49"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
MSE 24-hr 3 2-YEAR Rainfall=2.70"

	Area (ac)	CN	Description
*	3.000	98	EXISTING IMPERVIOUS
	1.000	74	>75% Grass cover, Good, HSG C
	1.000	70	Woods, Good, HSG C
	5.000	88	Weighted Average
	2.000		40.00% Pervious Area
	3.000		60.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry, MIN PER TR-55**

Subcatchment EX: EXISTING SITE



Summary for Subcatchment EX: EXISTING SITE

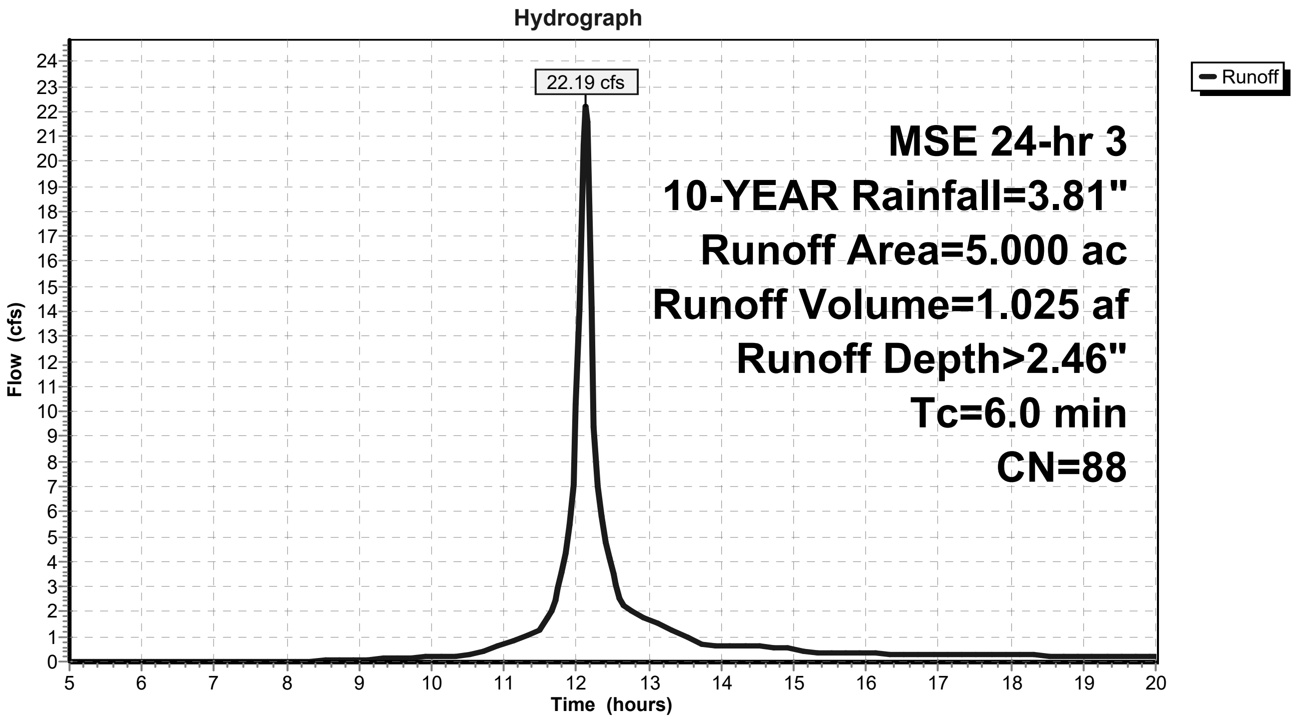
Runoff = 22.19 cfs @ 12.13 hrs, Volume= 1.025 af, Depth> 2.46"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
MSE 24-hr 3 10-YEAR Rainfall=3.81"

	Area (ac)	CN	Description
*	3.000	98	EXISTING IMPERVIOUS
	1.000	74	>75% Grass cover, Good, HSG C
	1.000	70	Woods, Good, HSG C
	5.000	88	Weighted Average
	2.000		40.00% Pervious Area
	3.000		60.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry, MIN PER TR-55**

Subcatchment EX: EXISTING SITE



Summary for Subcatchment EX: EXISTING SITE

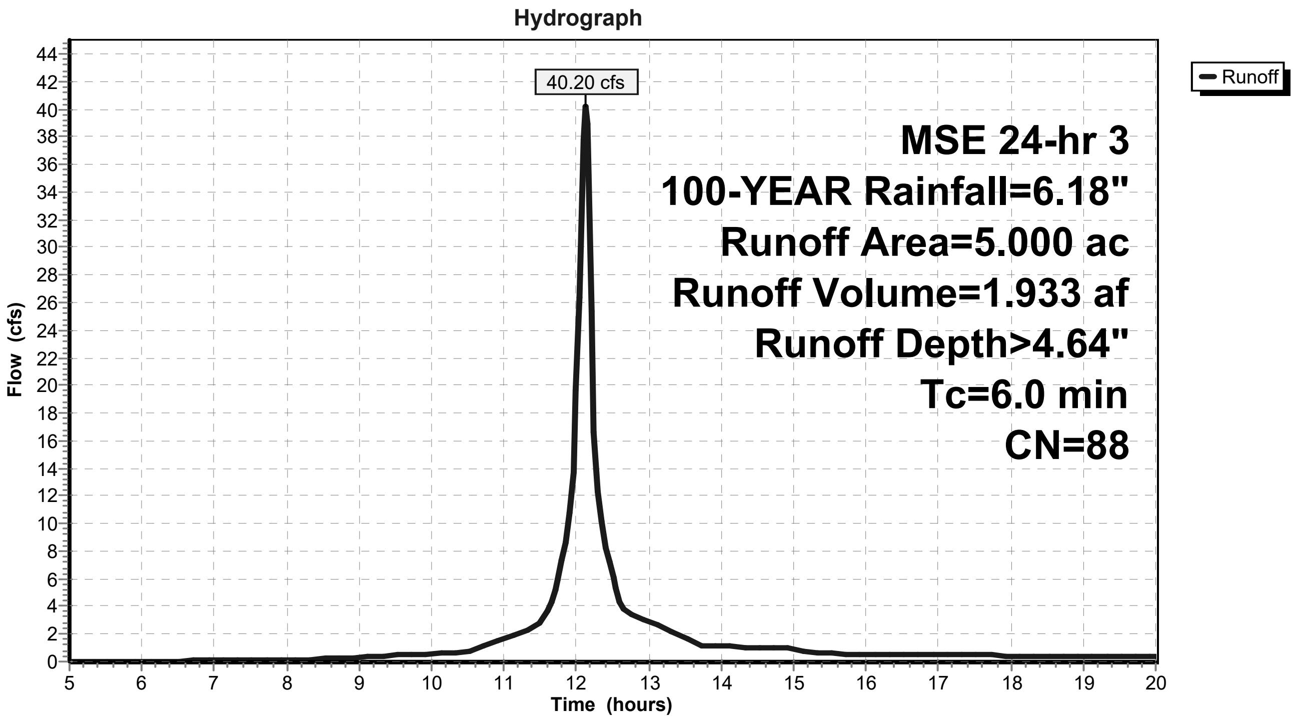
Runoff = 40.20 cfs @ 12.13 hrs, Volume= 1.933 af, Depth> 4.64"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
MSE 24-hr 3 100-YEAR Rainfall=6.18"

	Area (ac)	CN	Description
*	3.000	98	EXISTING IMPERVIOUS
	1.000	74	>75% Grass cover, Good, HSG C
	1.000	70	Woods, Good, HSG C
	5.000	88	Weighted Average
	2.000		40.00% Pervious Area
	3.000		60.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry, MIN PER TR-55**

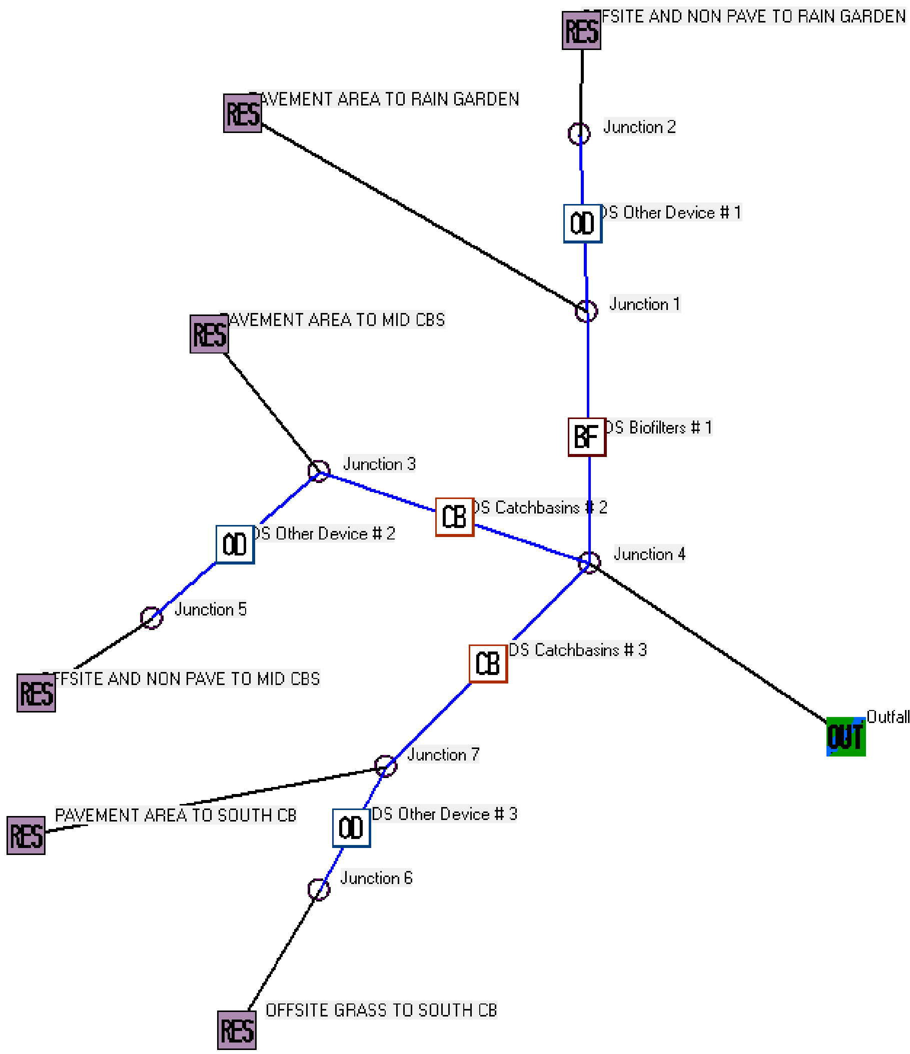
Subcatchment EX: EXISTING SITE



APPENDIX 3
POST-DEVELOPMENT CONDITIONS
WATER QUALITY

PINNACLE ENGINEERING GROUP

SLAMM ROUTING DIAGRAM



Land Use #	Land Use Type	Land Use Label	Land Use Area (acres)
1	Residential	PAVEMENT AREA TO RAIN GARDEN	0.230
2	Residential	OFFSITE AND NON PAVE TO RAIN GAR	2.300
3	Residential	PAVEMENT AREA TO MID CBS	0.280
4	Residential	OFFSITE AND NON PAVE TO MID CBS	0.500
5	Residential	PAVEMENT AREA TO SOUTH CB	0.060
6	Residential	OFFSITE GRASS TO SOUTH CB	0.090

CP #	Control Practice Type	Control Practice Name or Location
1	Biofilter	DS Biofilters # 1
2	Other Device	DS Other Device # 1
3	Other Device	DS Other Device # 2
4	Catchbasin Cleaning	DS Catchbasins # 2
5	Other Device	DS Other Device # 3
6	Catchbasin Cleaning	DS Catchbasins # 3

SLAMM RESULTS SUMMARY

File Name:

Z:\Projects\2019\1773.00A-WI\DESIGN\SWMP\SLAMM\2024-03-14 MANDEL.mdb

Outfall Output Summary

	Runoff Volume (cu. ft.)	Percent Runoff Reduction	Runoff Coefficient (Rv)	Particulate Solids Conc. (mg/L)	Particulate Solids Yield (lbs)	Percent Particulate Solids Reduction
Total of All Land Uses without Controls	146871		0.35	38.76 (1)	355.4 (1)	
Outfall Total with Controls	140124	4.59 %	0.34	24.36	213.1	40.04 %
Current File Output: Annualized Total After Outfall Controls	142070	Years in Model Run:	0.99		216.0	

(1) Values reduced to remove off-site loadings due to setting Other Control Device Concentration Reduction values to 1.

Print Output Summary to .csv File

Print Output Summary to Text File

Print Output Summary to Printer

Total Area Modeled (ac)

3.460

Total Control Practice Costs

Capital Cost	N/A
Land Cost	N/A
Annual Maintenance Cost	N/A
Present Value of All Costs	N/A
Annualized Value of All Costs	N/A

Perform Outfall Flow Duration Curve Calculations

Receiving Water Impacts Due To Stormwater Runoff

(CWP Impervious Cover Model)

	Calculated Rv	Approximate Urban Stream Classification
Without Controls	0.35	Poor
With Controls	0.34	Poor

Data file name: Z:\Projects\2019\1773.00A-WI\DESIGN\SWMP\SLAMM\2024-03-14 MANDEL.mdb
WinSLAMM Version 10.4.1
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
Source Area PSD and Peak to Average Flow Ratio File: C:\WinSLAMM Files\NURP Source Area PSD Files.csv
Cost Data file name:
If Other Device Pollutant Load Reduction Values = 1, Off-site Pollutant Loads are Removed from Pollutant Load % Reduction calculations
Seed for random number generator: -42
Study period starting date: 01/05/69   Study period ending date: 12/31/69
Start of Winter Season: 12/06   End of Winter Season: 03/28
Date: 07-18-2024   Time: 17:41:58
Site information:
MANDEL
LU# 1 - Residential: PAVEMENT AREA TO RAIN GARDEN   Total area (ac): 0.230
     13 - Paved Parking 1: 0.230 ac.   Connected   Source Area PSD File: C:\WinSLAMM Files\NURP.cpz
LU# 2 - Residential: OFFSITE AND NON PAVE TO RAIN GARDEN   Total area (ac): 2.300
     1 - Roofs 1: 0.300 ac.   Pitched   Connected   Source Area PSD File: C:\WinSLAMM Files\NURP.cpz
     13 - Paved Parking 1: 0.200 ac.   Connected   Source Area PSD File: C:\WinSLAMM Files\NURP.cpz
     45 - Large Landscaped Areas 1: 1.800 ac.   Normal Clayey   Source Area PSD File: C:\WinSLAMM Files\NURP.cpz
LU# 3 - Residential: PAVEMENT AREA TO MID CBS   Total area (ac): 0.280
     13 - Paved Parking 1: 0.280 ac.   Connected   Source Area PSD File: C:\WinSLAMM Files\NURP.cpz
LU# 4 - Residential: OFFSITE AND NON PAVE TO MID CBS   Total area (ac): 0.500
     1 - Roofs 1: 0.300 ac.   Pitched   Connected   Source Area PSD File: C:\WinSLAMM Files\NURP.cpz
     13 - Paved Parking 1: 0.200 ac.   Connected   Source Area PSD File: C:\WinSLAMM Files\NURP.cpz
LU# 5 - Residential: PAVEMENT AREA TO SOUTH CB   Total area (ac): 0.060
     13 - Paved Parking 1: 0.060 ac.   Connected   Source Area PSD File: C:\WinSLAMM Files\NURP.cpz
LU# 6 - Residential: OFFSITE GRASS TO SOUTH CB   Total area (ac): 0.090
     51 - Small Landscaped Areas 1: 0.090 ac.   Normal Clayey   Source Area PSD File: C:\WinSLAMM Files\NURP.cpz

      Control Practice 1: Biofilter CP# 1 (DS) - DS Biofilters # 1
         1. Top area (square feet) = 2000
         2. Bottom aea (square feet) = 670
         3. Depth (ft):   5.5

4. Biofilter width (ft) - for Cost Purposes Only: 10
5. Infiltration rate (in/hr) = 0.1
6. Random infiltration rate generation? No
7. Infiltration rate fraction (side): 0.001
8. Infiltration rate fraction (bottom): 1
9. Depth of biofilter that is rock filled (ft) 1
10. Porosity of rock filled volume = 0.35
11. Engineered soil infiltration rate: 3.6
12. Engineered soil depth (ft) = 2
13. Engineered soil porosity = 0.25
14. Percent solids reduction due to flow through engineered soil = 80
15. Biofilter peak to average flow ratio = 3.8
16. Number of biofiltration control devices = 1
17. Particle size distribution file: Not needed - calculated by program
18. Initial water surface elevation (ft): 0

Soil Data	Soil Type Fraction in Eng. Soil
User-Defined Soil Type	1.000

Biofilter Outlet/Discharge Characteristics:

Outlet type: Broad Crested Weir
1. Weir crest length (ft): 3
2. Weir crest width (ft): 10
3. Height of datum to bottom of weir opening: 4.4

Outlet type: Vertical Stand Pipe
1. Stand pipe diameter (ft): 1.25
2. Stand pipe height above datum (ft): 3.5

Outlet type: Drain Tile/Underdrain
1. Underdrain outlet diameter (ft): 0.33
2. Invert elevation above datum (ft): 0.25
3. Number of underdrain outlets: 1

Control Practice 2: Other Device CP# 1 (DS) - DS Other Device # 1
Fraction of drainage area served by device (ac) = 1.00
Particulate Concentration reduction fraction = 1.00
Filterable Concentration reduction fraction = 0.00
Runoff volume reduction fraction = 0

Control Practice 3: Other Device CP# 2 (DS) - DS Other Device # 2
Fraction of drainage area served by device (ac) = 1.00
Particulate Concentration reduction fraction = 1.00
Filterable Concentration reduction fraction = 0.00
Runoff volume reduction fraction = 0

Control Practice 4: Catchbasin Cleaning CP# 1 (DS) - DS Catchbasins # 2
1. Fraction of area served by catchbasins = 1.00
2. Number of catchbasins = 2

3. Average sump depth below catchbasin outlet invert (feet) = 1.5
4. Depth of sediment in catchbasin sump at beginning of study period (ft) = 0
5. Typical outlet pipe diameter (ft) = 1
6. Typical outlet pipe Mannings n = 0.012
7. Typical outlet pipe slope (ft/ft) = 0.005
8. Typical catchbasin sump surface area (square feet) = 19
9. Total catchbasin depth (feet) = 3
10. Inflow hydrograph peak to average flow ratio = 3.8
11. Leakage rate through sump bottom (in/hr) = 0
12. Catchbasin Critical Particle Size File Name: Not needed - calculated by program
13. Catchbasin cleaning frequency: Annually

Control Practice 5: Other Device CP# 3 (DS) - DS Other Device # 3

Fraction of drainage area served by device (ac) = 1.00
Particulate Concentration reduction fraction = 1.00
Filterable Concentration reduction fraction = 0.00
Runoff volume reduction fraction = 0

Control Practice 6: Catchbasin Cleaning CP# 2 (DS) - DS Catchbasins # 3

1. Fraction of area served by catchbasins = 1.00
2. Number of catchbasins = 1
3. Average sump depth below catchbasin outlet invert (feet) = 1.5
4. Depth of sediment in catchbasin sump at beginning of study period (ft) = 0
5. Typical outlet pipe diameter (ft) = 0.7
6. Typical outlet pipe Mannings n = 0.012
7. Typical outlet pipe slope (ft/ft) = 0.005
8. Typical catchbasin sump surface area (square feet) = 19
9. Total catchbasin depth (feet) = 4
10. Inflow hydrograph peak to average flow ratio = 3.8
11. Leakage rate through sump bottom (in/hr) = 0
12. Catchbasin Critical Particle Size File Name: Not needed - calculated by program
13. Catchbasin cleaning frequency: Annually

SLAMM for Windows Version 10.4.1

Data file name: Z:\Projects\2019\1773.00A-WI\DESIGN\SWMP\SLAMM\2024-03-14 MANDEL.mdb
Data file description: MANDEL
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
Pollutant Relative Concentration file name: C:\WinSLAMM Files\WI_GEO03.ppdx
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
Source Area PSD and Peak to Average Flow Ratio File: C:\WinSLAMM Files\NURP Source Area PSD Files.csv
Cost Data file name:
If Other Device Pollutant Load Reduction Values = 1, Off-site Pollutant Loads are Removed from Pollutant Load % Reduction calculations
Seed for random number generator: -42
Start of Winter Season: 12/06   End of Winter Season: 03/28
Model Run Start Date: 01/05/69   Model Run End Date: 12/31/69
Date of run: 07-18-2024   Time of run: 17:42:27
Total Area Modeled (acres): 3.460
Years in Model Run: 0.99

	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:	146871	-	38.76	355.4	-
Outfall Total with Controls:	140124	4.59%	24.36	213.1	40.04%
Annualized Total After Outfall Controls:	142070			216.0	

APPENDIX 4
POST DEVELOPMENT PEAK FLOWS

PINNACLE ENGINEERING GROUP

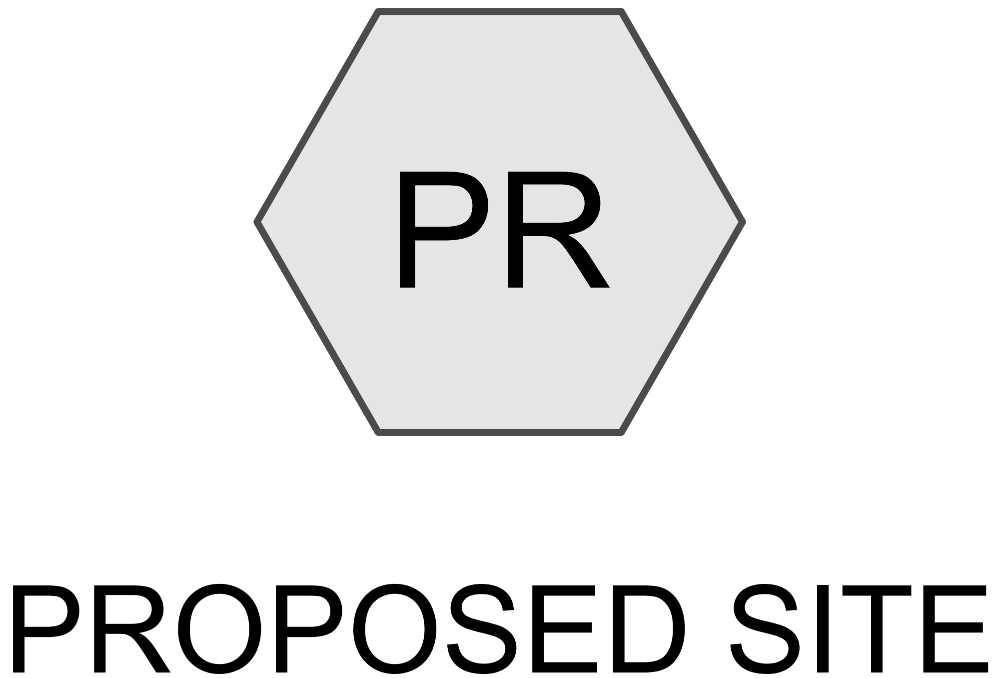
MANDEL DELAFIELD ST DEVELOPMENT PROPOSED HYDROLOGY EXHIBIT

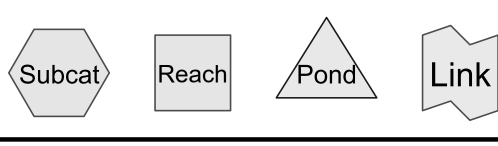
7/19/2024

PINNACLE ENGINEERING GROUP

20725 WATERTOWN ROAD | SUITE 100 | BROOKFIELD, WI 53186 | WWW.PINNACLE-ENGR.COM | ADMIN@PINNACLE-ENGR.COM

PLAN | DESIGN | DELIVER





Routing Diagram for PRELIM
Prepared by Pinnacle Engineering Group, Printed 7/18/2024

Summary for Subcatchment PR: PROPOSED SITE

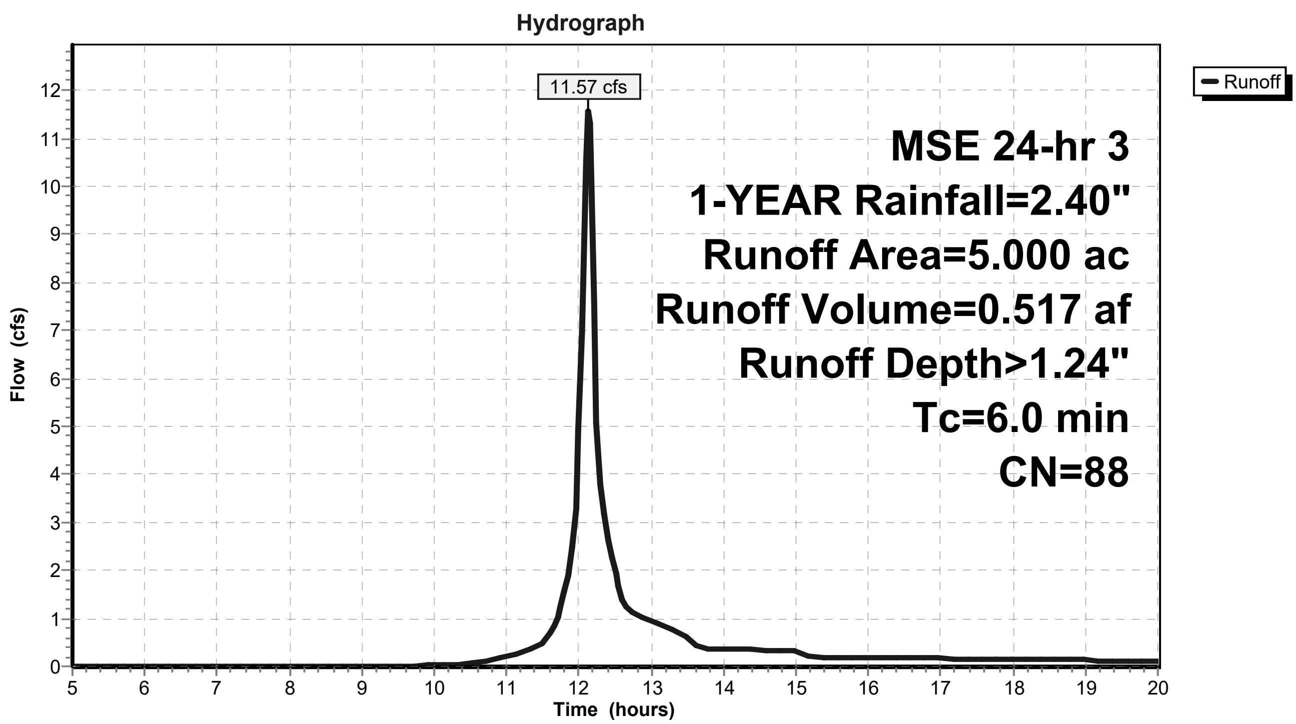
Runoff = 11.57 cfs @ 12.13 hrs, Volume= 0.517 af, Depth> 1.24"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
MSE 24-hr 3 1-YEAR Rainfall=2.40"

	Area (ac)	CN	Description
*	2.300	98	ROOF
*	0.700	98	PAVEMENT
*	1.000	74	GRASS
	1.000	70	Woods, Good, HSG C
	5.000	88	Weighted Average
	2.000		40.00% Pervious Area
	3.000		60.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry, MIN PER TR-55**

Subcatchment PR: PROPOSED SITE



Summary for Subcatchment PR: PROPOSED SITE

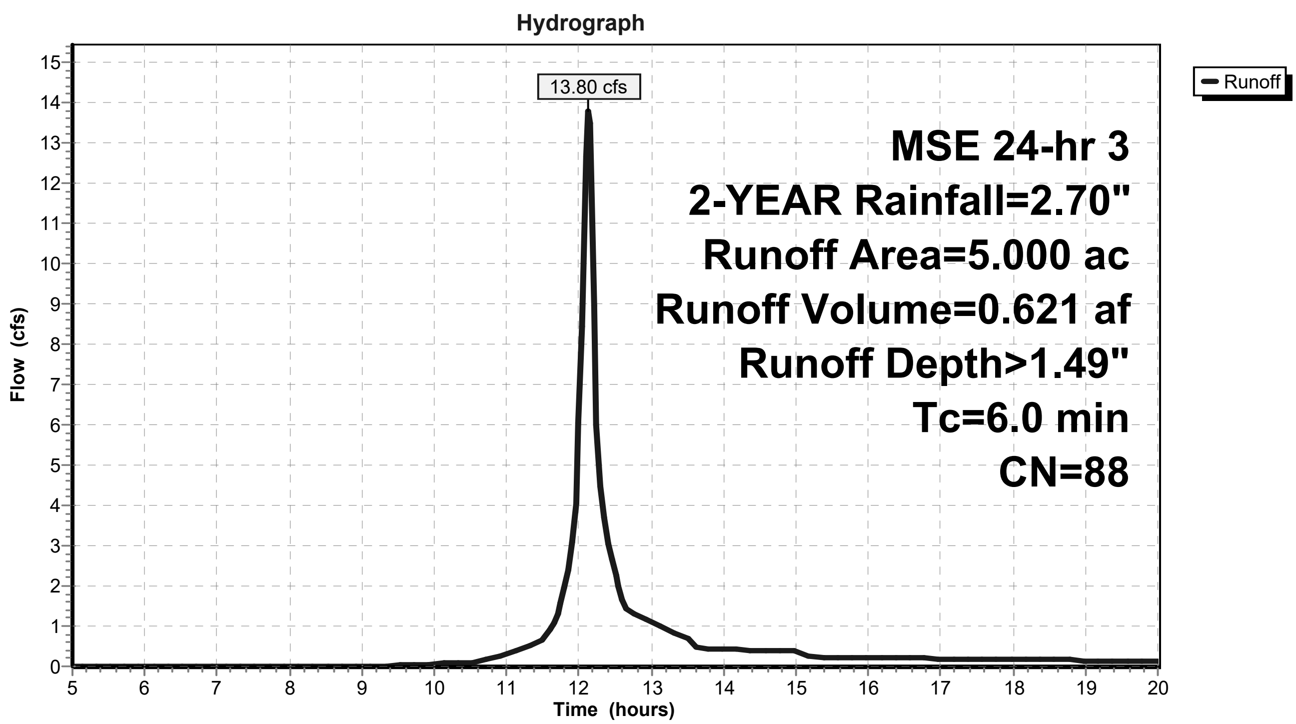
Runoff = 13.80 cfs @ 12.13 hrs, Volume= 0.621 af, Depth> 1.49"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
MSE 24-hr 3 2-YEAR Rainfall=2.70"

	Area (ac)	CN	Description
*	2.300	98	ROOF
*	0.700	98	PAVEMENT
*	1.000	74	GRASS
	1.000	70	Woods, Good, HSG C
	5.000	88	Weighted Average
	2.000		40.00% Pervious Area
	3.000		60.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry, MIN PER TR-55**

Subcatchment PR: PROPOSED SITE



Summary for Subcatchment PR: PROPOSED SITE

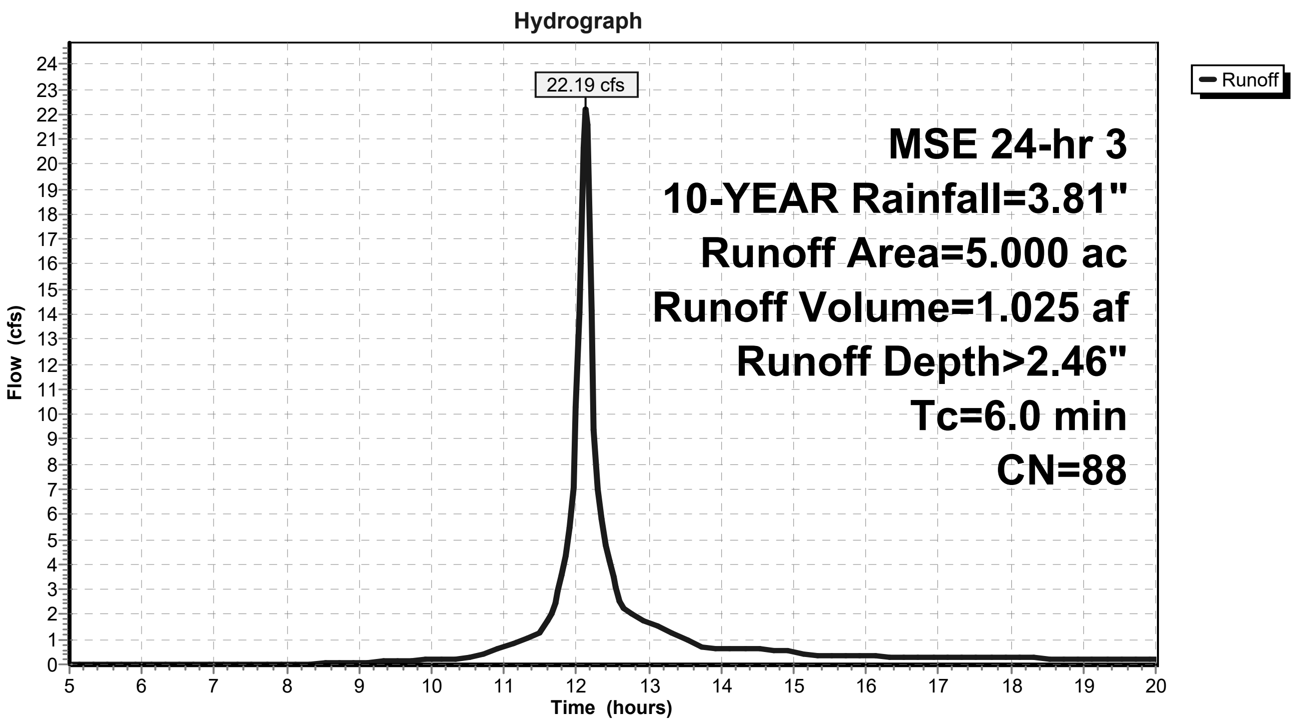
Runoff = 22.19 cfs @ 12.13 hrs, Volume= 1.025 af, Depth> 2.46"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
MSE 24-hr 3 10-YEAR Rainfall=3.81"

	Area (ac)	CN	Description
*	2.300	98	ROOF
*	0.700	98	PAVEMENT
*	1.000	74	GRASS
	1.000	70	Woods, Good, HSG C
	5.000	88	Weighted Average
	2.000		40.00% Pervious Area
	3.000		60.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry, MIN PER TR-55**

Subcatchment PR: PROPOSED SITE



Summary for Subcatchment PR: PROPOSED SITE

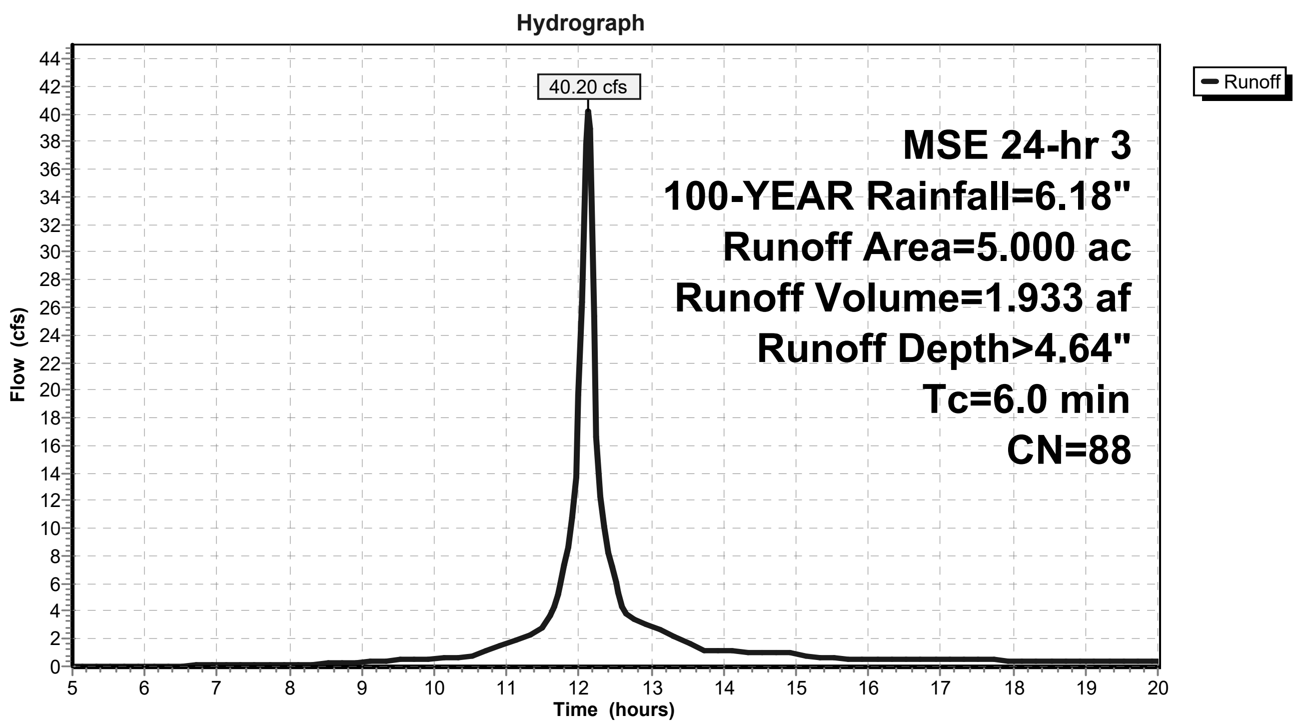
Runoff = 40.20 cfs @ 12.13 hrs, Volume= 1.933 af, Depth> 4.64"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs
MSE 24-hr 3 100-YEAR Rainfall=6.18"

	Area (ac)	CN	Description
*	2.300	98	ROOF
*	0.700	98	PAVEMENT
*	1.000	74	GRASS
	1.000	70	Woods, Good, HSG C
	5.000	88	Weighted Average
	2.000		40.00% Pervious Area
	3.000		60.00% Impervious Area

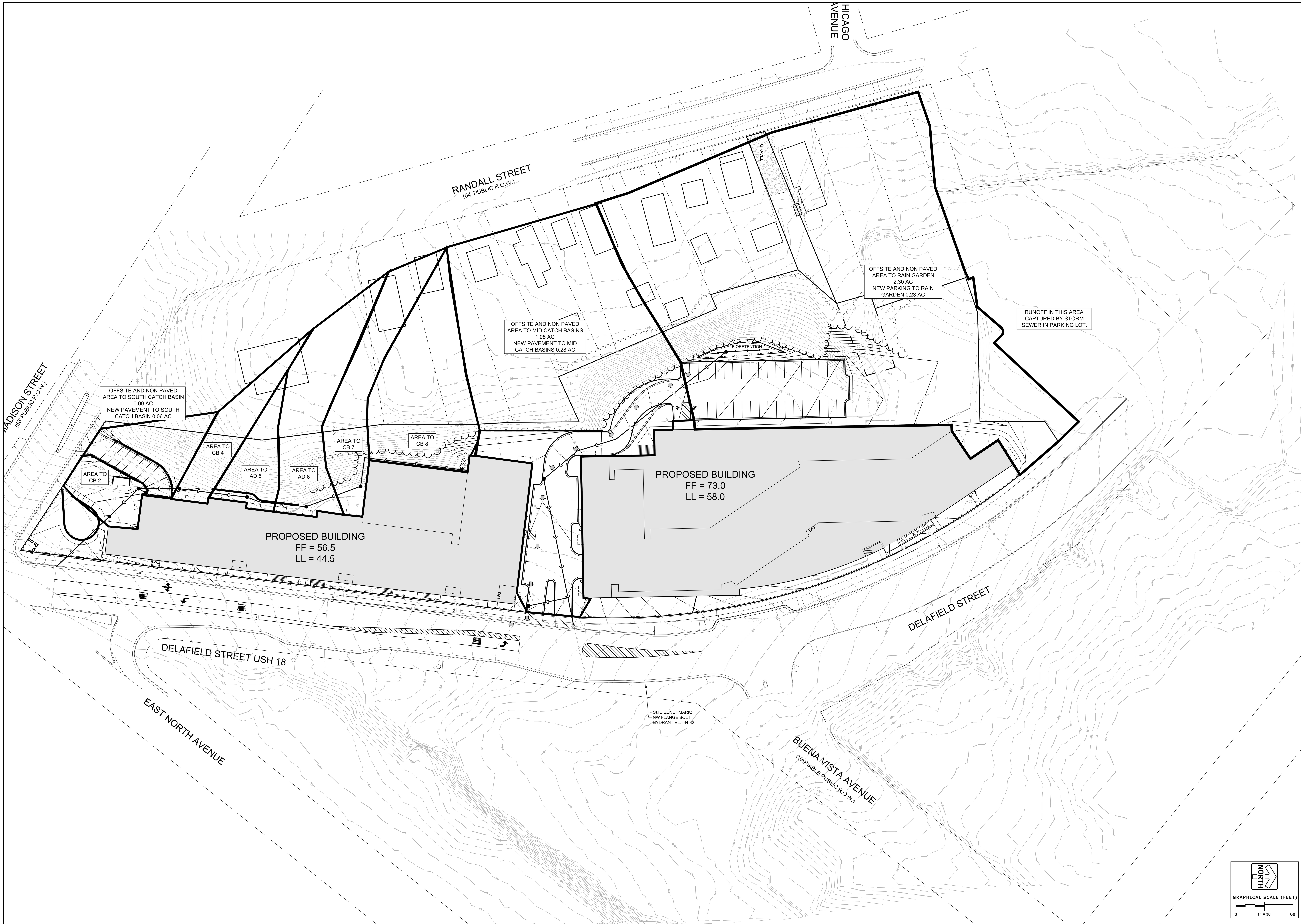
Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry, MIN PER TR-55**

Subcatchment PR: PROPOSED SITE



APPENDIX 5
SUPPLEMENTAL MATERIAL

PINNACLE ENGINEERING GROUP



MANDEL DELAFIELD ST DEVELOPMENT STORM SEWER AREA EXHIBIT

7/19/2024

PINNACLE ENGINEERING GROUP

20725 WATERTOWN ROAD | SUITE 100 | BROOKFIELD, WI 53186 | WWW.PINNACLE-ENGR.COM | ADMIN@PINNACLE-ENGR.COM

PLAN | DESIGN | DELIVER

STORM SEWER COMPUTATIONS
FOR
MANDEL DELAFIELD STREET DEVELOPMENT

PINNACLE ENGINEERING GROUP
Plan | Design | Deliver
www.pinnacle-engr.com

WAUKESHA, WISCONSIN

SHEET 1 OF 1
DESIGN BY: AEK
PROJECT NUMBER: 1773.00A
DATE: 7/18/2024

DESIGN DATA

County: Waukesha | **Design Storm:** 100 yr | **Storm Duration:** 6 min | **DESIGN INTENSITY (I):** 9.93 in/hr | Intensity calculated using SEWRPC IDF equations.

STRUCTURE DATA | DRAINAGE AREA AND FLOW DATA (Flow is determined by Rational Method, Q = CIA) | PIPE DATA | PIPE CAPACITY INFORMATION (Pipe capacity is determined by Manning's Equation, $Q = 1.486/n\ AR^{2/3}\ S^{1/2}$) | ELEVATIONS

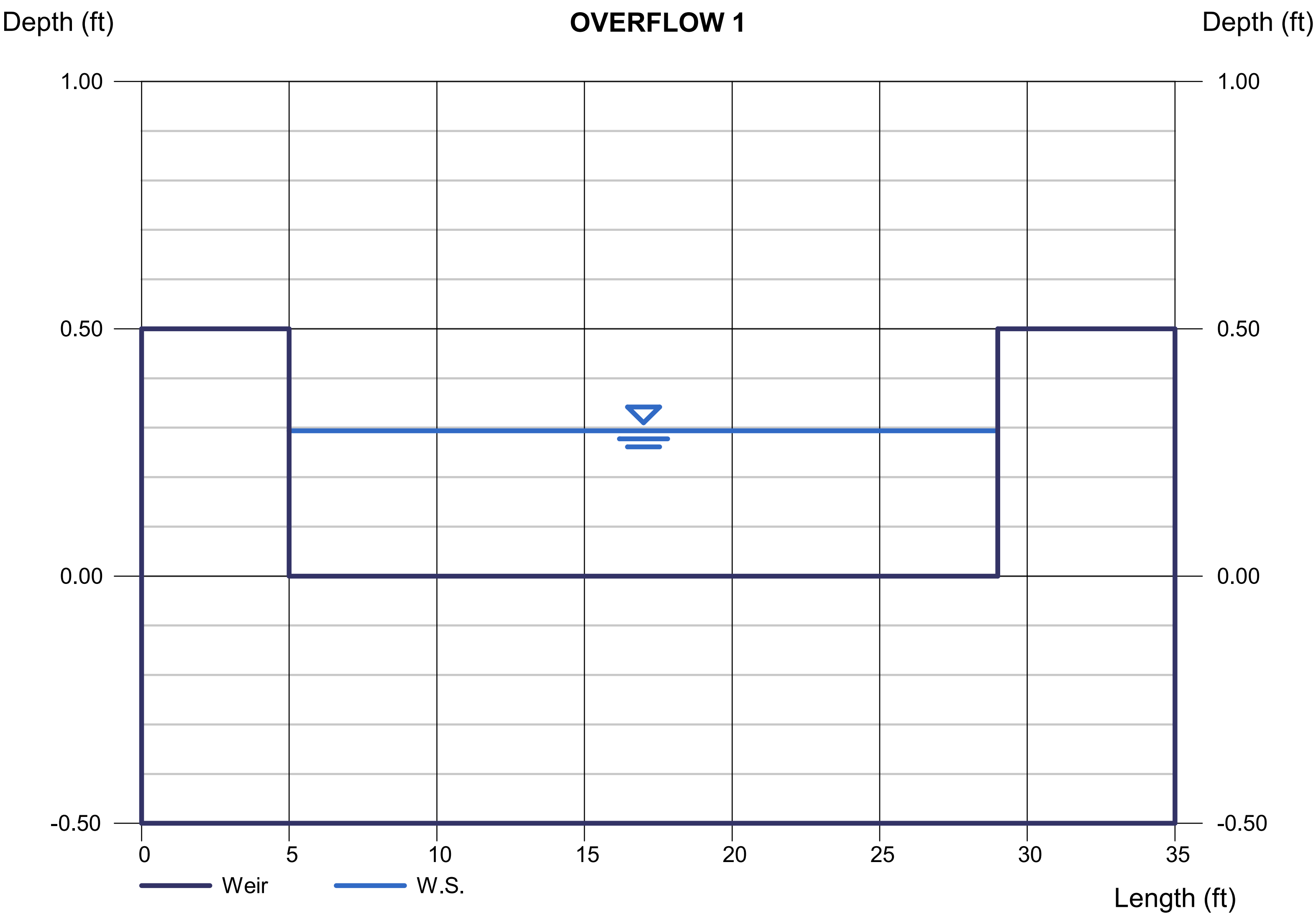
Notes	Upstream Structure	Downstream Structure	Individual Acres A	Individual Coefficient C	Individual Flow Q (cfs)	Cumulative Flow (cfs)	Length (ft)	Diameter (in)	Slope (%)	Manning Coefficient	Required Drop (ft)	Actual Drop (ft)	Percent Full (%)	Actual Velocity (fps)	Max. Capacity (cfs)	Rim/Toc Up	Invert Up	Invert Down
	POND OUTLET	CB 1	2.53	0.41	10.30	10.30	235.20	15	2.00	0.012	5.10	4.70	92%	9.16	10.65	63.00	59.75	55.05
	CB 1	EX MH	1.15	0.47	5.37	17.19	155.80	24	0.50	0.012	0.77	0.78	87%	6.29	18.64	58.30	54.31	53.53
	CB 1A	WYE	0.21	0.73	1.52	1.52	42.50	10	0.50	0.012	0.17	0.21	80%	3.49	1.81	56.90	54.47	54.26
	CB 8	CB 7	0.38	0.40	1.51	1.51	139.00	8	2.00	0.012	1.85	2.78	73%	5.91	1.99	63.80	60.72	51.94
	CB 7	AD 6	0.24	0.40	0.95	2.46	60.80	12	0.50	0.012	0.25	0.30	80%	3.93	2.94	55.00	51.94	51.64
	AD 6	AD 5	0.28	0.40	1.11	3.57	62.90	12	1.00	0.012	0.54	0.63	82%	5.58	4.15	55.00	51.64	51.01
	AD 5	CB 4	0.11	0.40	0.44	4.01	71.10	12	1.00	0.012	0.77	0.71	91%	5.58	4.15	55.00	51.01	50.30
	CB 4	CB 3	0.14	0.40	0.56	4.57	42.90	12	1.50	0.012	0.60	0.64	85%	6.86	5.08	54.50	41.07	40.43
	CB 3	CB 2	0.00	0.40	0.00	5.46	42.30	12	4.00	0.012	0.85	1.69	65%	10.66	8.30	46.00	40.43	38.74
	CB 2	EXIST CB	0.09	0.40	0.36	5.82	75.90	12	4.00	0.012	1.73	3.04	69%	10.80	8.30	44.00	38.74	35.70
	TRENCH DRN	CB 3A	0.15	0.60	0.89	0.89	4.00	6	4.00	0.012	0.09	0.16	67%	6.77	1.31	44.40	42.06	41.90
	CB 3A	CB 3	0.00	0.00	0.00	0.89	10.00	6	4.00	0.012	0.22	0.40	67%	6.77	1.31	45.10	41.90	41.50

Weir Report

Hydraflow Express Extension for Autodesk® Civil 3D® by Autodesk, Inc. Monday, Mar 18 2024

OVERFLOW 1

Rectangular Weir		**Highlighted**	
Crest	= Broad	Depth (ft)	= 0.29
Bottom Length (ft)	= 24.00	Q (cfs)	= 9.930
Total Depth (ft)	= 0.50	Area (sqft)	= 7.04
		Velocity (ft/s)	= 1.41
Calculations		Top Width (ft)	= 24.00
Weir Coeff. Cw	= 2.60		
Compute by:	Known Q		
Known Q (cfs)	= 9.93		



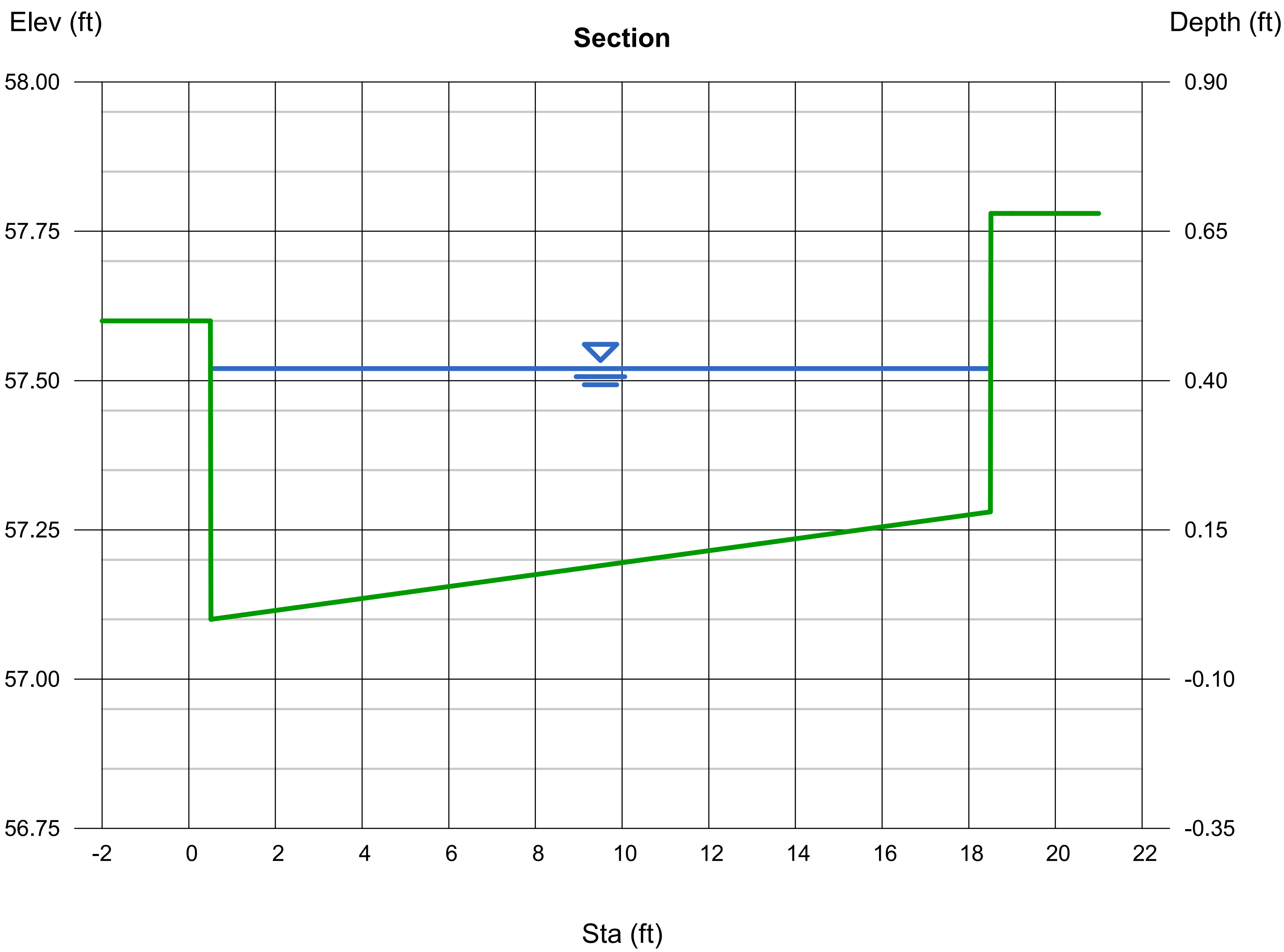
Channel Report

Overflow 2

User-defined		**Highlighted**	
Invert Elev (ft)	= 57.10	Depth (ft)	= 0.42
Slope (%)	= 1.20	Q (cfs)	= 17.24
N-Value	= 0.025	Area (sqft)	= 5.94
		Velocity (ft/s)	= 2.90
Calculations		Wetted Perim (ft)	= 18.65
Compute by:	Known Q	Crit Depth, Yc (ft)	= 0.40
Known Q (cfs)	= 17.24	Top Width (ft)	= 18.00
		EGL (ft)	= 0.55

(Sta, El, n)-(Sta, El, n)...

(0.00, 57.60)-(0.50, 57.60, 0.025)-(0.51, 57.10, 0.025)-(18.50, 57.28, 0.025)-(18.51, 57.78, 0.025)-(19.00, 57.78, 0.025)



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GEOTECHNICAL ENGINEERING REPORT
Delafield Street Apartments Development
Northwest Corner of Madison Street and Delafield Street
Waukesha, Wisconsin

April 30, 2024
File No. 20.0158728.00

PREPARED FOR:
Mandel Group, Inc.
Milwaukee, Wisconsin

GZA GeoEnvironmental, Inc.
17975 West Sarah Lane, Suite 100 | Brookfield, WI 53045
262-754-2560

32 Offices Nationwide
www.gza.com

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CONSTRUCTION MANAGEMENT

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April 30, 2024
File No. 20.0158728.00

Mr. Daniel Romnek
Mandel Group, Inc.
330 East Kilbourn Avenue, Suite 600 South
Milwaukee, Wisconsin 53202-3144

Re: Geotechnical Engineering Report
Delafield Street Apartments Development
Northwest Corner of Madison Street and Delafield Street
Waukesha, Wisconsin

Dear Mr. Romnek:

GZA GeoEnvironmental, Inc. (GZA) is pleased to present the attached Geotechnical Engineering Report ("Report") for the Delafield Street Apartments Development located at the northwest corner of Madison Street and Delafield Street in Waukesha, Wisconsin ("Site"). The Report provides findings, conclusions, and recommendations that GZA derived from our geotechnical evaluation and are based on our current understanding of the project.

We appreciate the opportunity to provide services for this project. Please feel free to contact us with questions.

Very truly yours,

GZA GeoEnvironmental, Inc.

Colin J. Byron, E.I.T.
Geotechnical Engineer

Jesse D. Graham, P.E.
Senior Project Manager, Geotechnical Engineer

Ian J. Mosbrucker, P.E.
Project Manager

Ernest R. Hanna, P.E.
Senior Principal

J:\158700to158799\158728\Report\FINAL 20.0158728.00 Geotechnical Eng Rpt_Waukesha WI 4-30-24.docx

Attachments

FIGURES

APPENDICES

1.0 INTRODUCTION

At the request of Mandel Group, Inc. ("Client"), GZA GeoEnvironmental, Inc. (GZA) conducted a geotechnical engineering evaluation for the potential development of three previously developed parcels located at 130 Delafield Street, 201 Delafield Street, and 318 Delafield Street, in Waukesha, Wisconsin ("Site"). A Site Location Plan is provided as **Figure 1**. GZA's services were conducted in general accordance with our January 19, 2024 Proposal for Geotechnical Exploration Services, GZA File No. 20.P000566.24. The findings, conclusions, and recommendations that GZA derived from the geotechnical evaluation are provided in this Geotechnical Engineering Report ("Report") and are subject to the Limitations provided in **Appendix A**. Boring elevations, when provided, are in reference to the City of Waukesha Vertical Datum (COW) and are based on elevation contours provided in an electronic drawing file titled "1773.00 ALTA," provided to GZA by the Client on January 8, 2024. Elevations are assumed to be accurate to the nearest +/- 1 foot unless specified otherwise.

1.1 SUMMARY

Geotechnical findings and recommendations for the potential development are summarized below. The summary is provided for the convenience of the non-technical reader and must be read in complete context with the remaining Report.

1. GZA field staff oversaw and logged 19 soil test borings at the Site. The 19 borings were drilled to depths of approximately 3.5 to 28.5 feet below ground surface (bgs) to evaluate the subsurface conditions for the potential development. The borings were advanced on the existing vacant parcel at the approximate locations shown on **Figure 2**.
2. Based on the subsurface conditions and estimated structural loads/foundations sizes for the future development, it is our opinion that shallow foundation systems will be feasible for building support at the Site. However, due to the existing fill and somewhat variable soils present at the Site, overexcavation and replacement will be necessary. Other methods of soil improvement, such as rammed aggregate piers, could also be considered.
3. Isolated areas of organic and soft soils should be removed from areas below structural footings and replaced with approved, compacted, engineered fill materials or lean-mix concrete slurry. Additional overexcavation may be required in areas of previously placed thick fill deposits. A maximum, net, allowable bearing capacities of **2,500 and 3,000 pounds per square foot (psf), for the north and south buildings, respectively,** may be used for foundation design, depending on final grades and location on-Site. Strip footings should be at least 24 inches wide and a maximum of 6 feet wide. Isolated column footings should be at least 30 inches wide and a maximum of 12 feet wide.
4. We recommend that perimeter foundations in unheated areas extend at least 48 inches below the lowest adjacent grade, or deeper if required by local Building Code.
5. A vertical modulus of subgrade reaction (k_v) of 90 pounds per cubic inch (pci) is recommended for concrete slab-on-grade design assuming the recommendations within this Report are followed. If encountered at footing grade, organic soils and deleterious fill materials should be removed prior to proof rolling and slab-on-grade placement. Backfilling of overexcavated soils with approved backfill materials will likely increase the k_v value in these locations.
6. Subgrade soils exposed during excavation and grading may be sensitive to moisture changes and disturbance due to construction traffic. Therefore, the exposed subgrade soils should be protected from moisture changes and monitored for disturbance from construction equipment. We recommend that a layer of compacted, well-graded, aggregate fill be placed over exposed construction traffic routes and laydown areas (minimum of 9 inches thick) to reduce disturbance potential. Due to the fine-grained nature of subgrade soils, a separation geotextile may also be

required along construction traffic routes and laydown areas. Excavation into the silt and/or clay soils should be done with smooth-edged equipment to reduce disturbance. Concrete mud slabs should be considered for construction taking place during periods of seasonal moisture increases.

7. Groundwater was observed in 14 soil borings at depths ranging from about 3.7 to 23 feet bgs, after allowing the groundwater to stabilize for times ranging from 5 minutes to 2.5 hours. Temporary dewatering should be expected when performing excavations for the foundations unless Site grades are raised. It is expected that conventional sumps/pumps placed in the excavation should be sufficient for dewatering the expected groundwater infiltration.

1.2 PURPOSE AND SCOPE OF SERVICES

GZA's objective for this project was to develop geotechnical engineering recommendations for design and construction of foundations, slabs-on-grade, fill placement, conditions regarding the existing structures, and construction considerations for the potential development. To achieve our objective, GZA:

- Reviewed previous work in the area completed by GZA and other publicly available information;
- Coordinated a geotechnical subsurface exploration program that consisted of 19 soil borings;
- Classified soil samples recovered from the borings based on field observations and prepared boring logs using our visual classifications;
- Conducted field and laboratory tests on a select number of soil samples to aid in the evaluation of the engineering properties of the subsurface soils;
- Performed geotechnical engineering analyses and developed geotechnical engineering recommendations; and
- Prepared this Report, which summarizes GZA's findings from the geotechnical evaluation and provides geotechnical recommendations for the potential project.

2.0 BACKGROUND

2.1 EXISTING CONDITIONS AND HISTORIC DEVELOPMENT

2.1.1 Existing Conditions

The Site is located at the northwest corner of Madison Street and Delafield Street in Waukesha, Wisconsin in the northeast quarter of Section 3, Township 6 North, Range 19 East of the 4th Principal Meridian. The Site is approximately 3.95 acres and is bounded by Delafield Street on the north and east, Madison Street on the south, and private property on the west. Site surface elevations range from about Elevation (El.) 41 to El. 74, COW Vertical Datum. The Site is currently occupied by the previous City of Waukesha municipal building and three asphalt parking lots separated by two concrete retaining walls.

Existing underground utilities, including overhead electric lines, storm sewer, sanitary sewer, electrical, gas, and communications, are present on and adjacent to the Site. Public utilities were marked in the field by utility locating companies as part of our services.

2.1.2 Historic Development

Based on the review of select historic aerial photographs, topographic maps, and other publicly available information, an abbreviated history of the Site can be interpreted. Individual interpreted parcel development histories are provided below. The summary presented below is not intended to be exhaustive.

- As shown on the original plat map of southeastern Wisconsin (1837) and a United States Geological Survey (USGS) topographic map (1906), there is evidence of a river or stream that previously ran through the center of the Site from northwest to southeast until approximately the 1950s. The stream was likely filled in during construction activities that occurred on the parcels included within the Site boundaries during and prior to the 1960s.
- 130 Delafield Street - A vacated City of Waukesha municipal building with underground parking and an at-grade asphalt parking lot were constructed between 1963 and 1970, and remain today.
- 201 Delafield Street - A shopping center and asphalt parking lot appear to have been constructed between 1950 and 1963. A dry cleaner was located on the north end of the shopping center and has documented evidence of contamination (Wisconsin Department of Natural Resources [WDNR] Bureau for Remediation and Redevelopment Tracking System [BRRTS] #02-68-494990). The Site is listed as "closed" on the Wisconsin Remediation and Redevelopment Database (WRRD). The shopping center was demolished in 2018, but the asphalt parking lot remains.
- 318 Delafield Street - A gas station and asphalt parking lot appear to have been constructed between 1950 and 1963. Eight underground storage tanks (USTs) were utilized by the gas station during operation. At least seven of the eight USTs were removed. There is documented evidence of a gasoline leak on the parcel. Overexcavation and removal of the contaminated material were performed in 1989. The Site is listed as "closed" on the WRRD. The gas station was demolished between 2017 and 2020, but the asphalt parking lot remains.

2.2 POTENTIAL DEVELOPMENT

The potential development will likely consist of two mixed-use residential and retail structures with underground parking. Foundation details were not provided to GZA at the time of writing this Report. However, based on work we have performed for other residential and retail developments, we anticipate that the typical (dead plus live) structural loads will range between about 400 and 600 kips for column loads and between 2 and 6 kips per foot for bearing walls. The finished first floor elevation for the north and south buildings are currently planned at approximately 61.5 feet and 46 feet (COW vertical datum), respectively. Updated floor plans and structural loading information should be provided to GZA for review prior to finalization of this Report.

3.0 SUBSURFACE EXPLORATION PROGRAM

GZA's subsurface exploration consisted of 19 soil test borings, as described below. The approximate locations of the soil borings are presented on **Figure 2** and detailed soil boring logs are provided in **Appendix B.** Three subsurface profile diagrams (fence diagrams) are provided in **Appendix C**, one for the north building, one for the south building, and one Site-wide fence diagram.

3.1 SOIL TEST BORINGS

GZA's drilling subcontractor, GESTRA of Milwaukee, Wisconsin, drilled 19 soil test borings (B-01 through B-14, B-02A, B-04A, B-04B, B-05A, and B-06A) at selected locations across the Site. The approximate locations of the borings are presented on **Figure 2**. The borings were drilled in two mobilizations with the first from January 30 to February 1, and the second on February 22, 2024. The borings were drilled to depths between 3.5 and 28.5 feet bgs (from approximate ground surface El. 44 to El. 70 [+/- 2 feet], COW Vertical Datum). The actual termination depths are noted on the boring logs in **Appendix B**.

GESTRA used a Diedrich D-50 track-mounted drill rig; a CME-75 truck-mounted drill rig; and hollow-stem auger drilling techniques to advance the borings to the termination depths. Split-spoon soil samples were obtained in accordance with ASTM D1586, *Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils*. The SPT consists of driving a 1⅜-inch inside-diameter (ID) sampler for at least 18 inches with a 140-pound hammer falling 30 inches. The number of blows required for each 6 inches of penetration was recorded, and the number of blows required to drive the sampler from 6 to 18 inches of penetration is the SPT N-value, a commonly used indicator of soil density and consistency. Pocket penetrometer testing was also performed on fine-grained soil to aid in the soil's in-situ consistency estimation. Undisturbed Shelby tube samples were also attempted within four of the test borings, and we were able to retrieve one relatively undisturbed sample. Refer to the boring logs in **Appendix B** for more information. Soil samples were collected in the split-spoon sampler and were stored in sealed, labeled, glass jars. Samples were classified in general accordance with ASTM D2488, *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*.

4.0 FIELD AND LABORATORY TESTING PROGRAM

Soil samples were classified in the field to determine approximate plasticity (for cohesive soils) and grain-size distribution (for granular soils). SPT N-values and/or pocket penetrometer testing results were utilized to evaluate relative density/consistency of the soils encountered and are recorded on the boring logs.

Laboratory tests were performed on select soil samples to aid in soil classification. The testing included natural moisture content, fines content (the percent of material passing the U.S. No. 200 Sieve, by weight), plasticity characterization (Atterberg Limits), grain size distribution, unit weight (density testing), moisture-density (Proctor) tests, and consolidation testing. Results of the laboratory tests are summarized on the boring logs in **Appendix B** and laboratory summary report in **Appendix D**.

5.0 GENERALIZED SUBSURFACE CONDITIONS

A generalized description of the subsurface conditions encountered at the soil boring locations are summarized in this section. Refer to the boring logs in **Appendix B** and the fence diagrams in **Appendix C** for more specific information. A summary of subsurface conditions is provided in the table below.

Boring No.	Surface Elev. (+/- 1 foot)	Boring Depth (feet bgs)	Approximate Thickness of Fill/Topsoil (feet)	Approximate Depth to Groundwater (feet bgs)	Approximate Bedrock Depth [Elevation] (+/- 1 foot)
B-01	44	25	7	21	25 [19]
B-02	46	19	4.5	15.3	19 [27]
B-02A	47	17	2	8.2	17 [30]
B-03	50	21	4.5	15.2	21 [29]
B-04	52	3.5	3.5	DRY	NE
B-04A	52	16.5	7	DRY	16.5 [35.5]
B-04B	52	15	NM	DRY	15 [37]
B-05	58	22	12	DRY	22 [36]

Boring No.	Surface Elev. (+/- 1 foot)	Boring Depth (feet bgs)	Approximate Thickness of Fill/Topsoil (feet)	Approximate Depth to Groundwater (feet bgs)	Approximate Bedrock Depth [Elevation] (+/- 1 foot)
B-05A	58	16	9.5	DRY	NE
B-06	59	6	6	NM	NE
B-06A	60	21.5	12	NM	21.5 [38.5]
B-07	61	25	13	15.9	25 [36]
B-08	61	24	8.5	DRY	24 [37]
B-09	61	20	12	11.5	NE
B-10	62	22	6	7.5	22 [40]
B-11	64	10	0.3	6.5	NE
B-12	64	21	7	5.9	NE
B-13	70	28.5	14.5	23	28.5 [41.5]
B-14	61	10	3.5	3.7	NE

Notes:

1. NM = not measured.
2. NE = not encountered.

5.1 SURFACE AND SUBSURFACE SOIL CONDITIONS

5.1.1 Fill and Topsoil

Surface conditions at the boring locations consisted of asphalt, topsoil, or fill underlain with native soils. Where encountered, the surficial asphalt layer was typically 4 inches thick and underlain with 6 to 12 inches of gravel base. Where measured, the surficial topsoil layer was approximately 3 to 5 inches thick. Topsoil generally consisted of sandy, lean clay with trace to little organics, and was generally distinguished by dark brown to black coloring. Fill soils were encountered in 17 borings at depths ranging from 2 to 14.5 feet bgs, and generally consisted of silty to clayey sand with trace to little gravel and sandy, lean clay. Moisture contents on tested samples within the fill ranged from about 7% to 38%, by weight. The granular fill soils had an average SPT N-value of 11 blows per foot (bpf), indicating medium-dense soil conditions. The fine-grained fill soils had consistencies ranging from very soft to very stiff and pocket penetrometer-measured unconfined compressive strengths ranging from 0.25 to 3.5 tons per square foot (tsf), with an average of 1.7 tsf, indicating an average stiff consistency. Gravel and cobbles were encountered in the fill layer at many of the locations explored.

5.1.2 Native Soils

Native soils were present under the surficial fill/topsoil layer and continued to bedrock. The native soils varied from silty and clayey sand (SM/SC), silt (ML), lean clay (CL) with trace to little sand to poorly-graded gravel (GP). The native soil generally was brown to tan, with isolated samples exhibiting orange mottling. Most of the clayey soils encountered in the native soil layer had stiff to hard consistencies and pocket penetrometer-measured unconfined compressive strengths ranged from 0.5 to 4.5+ tsf, with most tests being greater than 2.5 tsf. The granular soils had SPT N-values ranging from 1 to 50+, indicating loose to very dense soil conditions, and averaged approximately 29 bpf, indicating medium-dense soil conditions. Gravel and cobbles were generally encountered in the native soil layer, near the bedrock.

Moisture contents on tested samples ranged from about 6% to 31%, by weight. Atterberg limit tests were performed on selected samples to determine plasticity characteristics of the plastic, fine-grained soils. Plastic indices on tested samples ranged from about 0 to 33, indicating non-plastic to high plasticity soils. The average plasticity index was 12, and the median was 8 to 11.

A one-dimensional consolidation test was also performed on a Shelby tube sample of silty, lean clay from boring B-09 collected at a depth of 16 to 18 feet. Results of the consolidation test indicated normally consolidated to slightly over-consolidated soil conditions with a compression index (C_C) of 0.120, a Recompression Index (C_R) of 0.023, preconsolidation stresses (P_C) of 2,000 psf, and an estimated overconsolidation ratio of approximately 1.2.

5.2 BEDROCK

Based on auger refusal, bedrock is believed to have been encountered in 12 borings (B-01, B-02, B-02A, B-03, B-04A, B-04B, B-05, B-06A, B-07, B-08, B-10, B-13). Depth to bedrock varied across the Site and ranged from 15 to 28.5 feet bgs. The bedrock surface generally dipped down from north to south and bedrock elevations ranged from 41.5 feet to 19 feet (COW Vertical Datum), respectively.

5.3 GROUNDWATER

Groundwater was encountered and measured in 11 borings at depths ranging from 3.7 to 23.0 feet bgs (El. 23 to El. 58). Groundwater was measured in the borings after drilling. Groundwater was variable across the Site, but, on average, was encountered around 12 to 15 feet bgs. Fluctuations of the groundwater table will likely occur due to seasonal variations in the amount of rainfall, runoff, and other factors not evident at the time the soil borings were performed. Therefore, groundwater levels during construction or at other times over the life of the structures may be different than the levels indicated on the boring logs.

6.0 GEOTECHNICAL CONCLUSIONS AND RECOMMENDATIONS

Geotechnical design and related construction recommendations for the Site are provided in the following sections. The recommendations are based on subsurface conditions encountered in the borings, results of laboratory testing, and the results of GZA's geotechnical engineering analyses. As the project design continues to develop, these recommendations should be reviewed so that more structure-specific geotechnical design criteria can be developed.

6.1 GENERAL GEOTECHNICAL CONSIDERATIONS

The fill encountered on-Site was likely imported for historic construction activities that took place at the Site. The fill has variable composition, consistency, moisture content, and strength characteristics. GZA understands that structures are planned to be constructed where fill is present. The existing fill soils are not suitable for foundation support without some overexcavation and replacement or ground improvement methods. Additionally, the presence of the former stream at the Site may lead to ongoing groundwater control costs during construction and in areas of the proposed structures constructed below the groundwater table.

Based on the subsurface conditions in the soil borings and estimated structural loads, it is our opinion that shallow foundation systems consisting of strip footings with isolated column pads are feasible for building support at the Site. Areas of soil removal and replacement will be required where unsuitable bearing strength soils and fill soils are present below designed structural footing elevations. Alternatively, ground improvement methods, such as rammed aggregate piers (RAPs), could be considered in areas where overexcavation may no longer be considered economically viable.

It is likely that remnants of former structure(s) are present below grade and will require removal during excavation and prior to construction of new foundations. **Possible buried concrete slabs and/or footings were encountered at boring B-04.** Additionally, the existing structure on-Site will have subgrade foundations and basement walls that will also need to be removed prior to construction of new foundations. Remnants of previous foundations, existing foundations, buried utilities, and other structural elements should be removed in their entirety prior to placement of new fill or foundation elements.

6.2 NORTH BUILDING FOUNDATIONS

The planned north building is underlain by relatively horizontal bedrock varying from about El. 36 to 42. However, the existing fill present in the northern borings varied in relative thickness below the basement slab, as well as general consistency with SPT N-values varying from 0 to 50+ bpf. The highly variable existing fill soils will require improvement prior to foundation and/or lower level slab placement.

Fill located below foundations for the north building should be overexcavated to native soils or at least 6 feet if native soils are not encountered. Subgrade bearing soils should be evaluated by a representative of the geotechnical engineer to document bearing capacity of the soil and compaction of the structural backfill. If overexcavations do not encounter native soils, the bottom of the overexcavation should be checked to determine that either a correlated unconfined compressive strength of not less than 2 tsf or correlated blow count N-value of not less than 11 is determined using appropriate testing methods. Overexcavations should extend laterally beyond the edge of footing at least 2 feet or equal to depth of the overexcavation, whichever is greater. Sloping overexcavations and/or shoring will likely be required to allow for testing of soils at the base of the overexcavation.

After testing and approval, the overexcavated subgrade should be compacted prior to placing backfill. Overexcavations should be backfilled with structural backfill placed in 12-inch loose lifts and compacted to a minimum of 95% of the modified proctor compaction test (ASTM D1557, *Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort*). Alternatively, use of a lean-mix cement slurry (1,500 pounds per square inch [psi] minimum compressive strength) can be used to backfill overexcavations. If lean-mix is utilized, the width of the excavation only needs to be as wide as the planned footing width, provided the excavation sidewalls are stable.

It is our opinion that the north building foundation can be supported on shallow foundation systems consisting of strip footings and isolated column pads, provided the recommendations above are adhered to. Based on subsurface conditions and laboratory testing, **a maximum, net, allowable bearing capacity of 2,500 psf is recommended for the north building foundation design**. Strip footings should be at least 24 inches wide and a maximum of 6 feet wide. Isolated column footings should be at least 30 inches wide and a maximum of 12 feet wide.

We recommend that shallow spread footings in unheated areas that will be subject to freezing conditions bear at least 48 inches below the lowest adjacent exterior grade for frost protection.

Foundation settlement will depend on variations within the subsurface soil profile, the actual structural loading conditions, the embedment depth of the footings, and the quality of the earthwork operations. Based on the subsurface conditions and planned construction described herein, we estimate that post-construction total settlements of footings designed and constructed in accordance with the recommendations in this Report will be about 1 to 1.5 inches. Differential settlement across a distance of 25 feet within the building area is estimated to be ½-inch. These settlement estimates assume that footings will be supported on suitable existing soils or a prepared structural compacted fill soil subgrade.

6.3 SOUTH BUILDING FOUNDATIONS

The planned south building is underlain by relatively horizontal bedrock on its north half, and then the bedrock surface dips down to the south. The apparent bedrock surface within the south building varies from about El. 27.5 to El. 36. Also, the existing fill present in the southern borings is anticipated to be removed in full below the basement slab and foundation elements.

The variable depth to bedrock could result in differential settlement of foundation elements, especially between borings B-04A and B-03 where the bedrock appears to drop in elevation by about 5 feet in a relatively short distance. If possible, structural breaks should be considered in this area to allow for additional differential settlements between the northern and southern sections of the south building.

It is our opinion that the south building foundation can be supported on shallow foundation systems consisting of strip footings and isolated column pads founded on native soils with a minimum correlated unconfined compressive strength of not less than 2.5 tsf or correlated N-value of not less than 15, as determined by applicable testing methods. Based on subsurface conditions and laboratory testing, **a maximum, net, allowable bearing capacity of 3,000 psf is recommended for the south building foundation design.** Strip footings should be at least 24 inches wide and a maximum of 6 feet wide. Isolated column footings should be at least 30 inches wide and a maximum of 12 feet wide.

We recommend that shallow spread footings in unheated areas that will be subject to freezing conditions bear at least 48 inches below the lowest adjacent exterior grade for frost protection.

Foundation settlement will depend on variations within the subsurface soil profile, the actual structural loading conditions, the embedment depth of the footings, and the quality of the earthwork operations. Based on the subsurface conditions and planned construction described herein, we estimate that post-construction total settlements of footings designed and constructed in accordance with the recommendations in this Report will be about 1.5 inches. Differential settlement across a distance of 25 feet within the building area is estimated to be ½-inch, with the exception of the areas between B-03 and B-04 where increased differential settlements of up to 1 inch may be encountered. These settlement estimates assume that footings will be supported on suitable existing soils.

6.4 GROUND IMPROVEMENT OPTION

To increase allowable bearing capacities for the building foundations, ground improvement, such as RAPs are recommended for foundation design. RAPs are densified columns of crushed stone over which shallow footings and slabs-on-grade can be constructed. RAPs are installed using a bottom-feed pipe fitted with a specially shaped tamper at the tip of the mandrel. The piers are typically installed by hammering a 10- to 12-inch hollow mandrel through the soft/loose soils. As the mandrel is removed, well-graded aggregate is fed through the mandrel and densified in lifts by hammering down at the bottom of the hole, creating columns of dense, crushed stone. At the bottom of the pier is a bulb of “clean stone,” which is typically located within a competent bearing stratum, such as the naturally deposited, very stiff to hard till soils or bedrock underlying the fill soils. The hammering both densifies the aggregate and forces the aggregate laterally into the sidewalls of the hole. This action increases the lateral stresses in the surrounding soil, thereby further stiffening the stabilized composite soil mass. Since the RAP elements act as soil reinforcement versus a structural section, there are no connections to be made from the foundation to the slab. Therefore, construction of the foundation can be completed “on-grade” using the RAP option. The above process of reinforcing soils with aggregate piers provides for a more uniform and consistent subgrade with less risk for differential settlement.

Based on initial conversations with a RAP design engineer, if RAPs are utilized for ground improvement, it is anticipated that the foundation systems can be designed to allow for a maximum footing bearing pressure of 5,500 psf. The actual bearing pressure will be determined by the RAP design engineer.

The RAPs should be designed by the specialty foundation contractor and installed so that the tips of the stone columns are embedded in the top 2 feet of the competent, very stiff to hard, till deposits or bedrock (approximate Elev. 42 to 50 for the north building and Elev. 20 to 41 for the south building). Footings should be centered on top of the piers or pier clusters. Following installation of the aggregate piers, the building footprint should be regraded to a level surface and heavily proof compacted. Structural fill, as defined in **Appendix E**, should then be placed in lifts not exceeding 12 inches and compacted in accordance with the recommendations provided below to achieve the required final subgrade elevations. Based on the results of the test borings, aggregate piers below the slab-on-grade do not appear to be necessary. Preparation of the footing and slab subgrade should be done in accordance with the recommendations below.

6.5 FLOOR SLAB

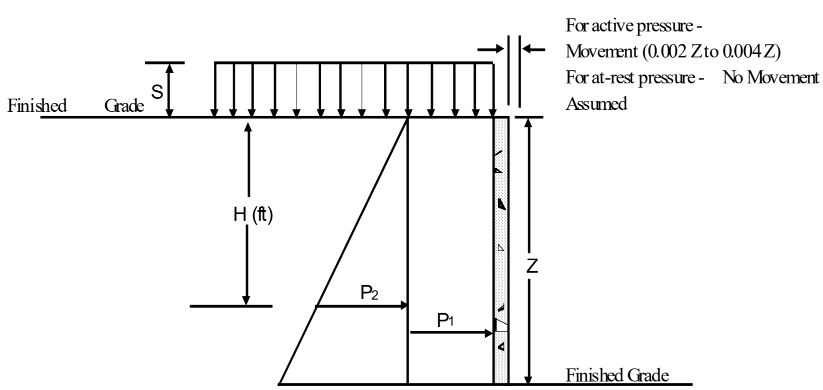
Concrete slab-on-grade or possibly basement floors will likely be proposed for the bottom floor of the structures. Based on the subsurface information, at-grade floor may be designed using a vertical modulus of subgrade reaction (k_v) of 90 pci, provided the slab, subgrade, and base are prepared in accordance with this Report. The floor slab should be isolated from walls and columns to allow for independent movement. Joints should be constructed at regular intervals, as recommended by the American Concrete Institute (ACI). The structural engineer should specify actual details of the floor slab, including thickness, reinforcing, and joint details.

Floor slabs should generally be underlain by a minimum 6-inch-thick, coarse aggregate base course. The coarse aggregate base course should consist of material meeting the requirements for Wisconsin Department of Transportation (WisDOT) Standard Specifications for Highway and Structure Construction. Base course materials should be placed and compacted in accordance with Section 6.7.2. An experienced geotechnical engineer should test and approve base course material prior to placement.

Depending on location and elevation, a water-resistant layer (waterproofing membrane) may be required.

6.6 LATERAL EARTH PRESSURES

Below grade walls and/or retaining walls are planned for the Site. Unbalanced backfill levels on opposite sides should be designed for earth pressures at least equal to those indicated in the following table. Earth pressures will be influenced by structural design of the walls, conditions of wall restraint, methods of construction and/or compaction, and the strength of the materials being restrained. Three wall restraint conditions are shown. Active earth pressure is commonly used for design of freestanding cantilever retaining walls that are unrestrained at the tops and free to move. The "at-rest" condition should be used for walls restrained from movement and rotation and would be appropriate for the basement walls proposed. Passive earth pressures represent movement of the retaining wall into the retained earth. Significant movement of the wall may be required to fully develop passive earth pressures. The recommended design lateral earth pressures are for cast-in-place concrete walls only and do not include a factor of safety or any provision for possible hydrostatic pressure on the walls.



Earth Pressure Coefficients

	Coefficient for Backfill Types	Equivalent Fluid Pressure (pcf)	Surcharge Pressure, P_1 (psf)	Earth Pressure, P_2 (psf)
Active (Ka)	Granular - 0.33 Lean Clay - 0.42	36 54	(0.33)S (0.42)S	(36)H (54)H
At-Rest (Ko)	Granular - 0.46 Lean Clay - 0.58	51 75	(0.46)S (0.58)S	(51)H (75)H
Passive (Kp)	Granular - 3.0 Lean Clay - 2.4	330 312	---	---

Conditions applicable to the above conditions include:

- For active earth pressure, the wall must rotate about base, with top lateral movements 0.002 Z to 0.004 Z, where Z is wall height;
- For passive earth pressure, the wall must move horizontally to mobilize resistance;
- Uniform surcharge, where S is the surcharge pressure;
- A maximum in-situ soil backfill weight of 110 pounds per cubic foot (pcf) should be used for granular backfill and 130 pcf for cohesive materials;
- Horizontal backfill, compacted to at least 90% of the ASTM D1557 maximum dry density;
- Loading from heavy compaction equipment is not included;
- No groundwater is acting on wall;
- No safety factor is included; and
- Ignore passive pressure in frost zone.

Under no circumstances should high-plasticity, clayey soils (those soils with liquid limits greater than 40 and/or plastic indices greater than 18) be used as backfill in retaining wall systems. Backfill placed against walls should consist of granular soils. For the granular values to be valid, the granular backfill should extend out from the base of the wall at an angle of at least 45° and 60° from vertical for the active and passive cases, respectively. Additional design considerations are required where these conditions are not met. To calculate the resistance to sliding, a value of 0.35 could be used as the allowable coefficient of friction between the footing and the underlying soil. To control the water level behind the wall, we recommend that perimeter drain tiles be utilized with outlet to storm sewers.

6.7 UTILITIES

Existing utilities, including water, electrical, gas, and communications, are present on the Site near the existing structure and along Delafield Street. If below grade utilities will remain for use with the new development, those utilities should be evaluated and rerouted prior to beginning construction, as necessary. Additionally, existing utilities adjacent to the proposed structure should be protected by crane mats or other bridging devices during construction. Abandoned utility pipes/conduit should be properly abandoned, removed, or plugged so it does not serve as conduits for water inflow and subsequent soil wetting and/or subsurface erosion, which could adversely affect the new foundation.

New underground pipes and utilities should be placed on bedding in accordance with the manufacturer's recommendations. Trench backfill should be placed in loose lifts not to exceed 6 inches thick and should be compacted in accordance with the specifications provided in this Report. Due to potential shallow groundwater conditions, buoyancy forces should be considered in the design of subsurface utility structures.

6.8 SEISMIC PARAMETERS

Based on the subsurface conditions encountered in the borings, the Site Class is considered to be "C" based on the 2015 International Building Code (IBC). The following seismic design parameters presented below are calculated using the California Office of Statewide Health Planning and Development (OSHPD) and Structural Engineers Association of California (SEAOC) *U.S. Seismic Design Maps* calculator using Site Class "C" and Risk Category II parameters per the IBC.

Parameter	Value (unit)
S_s = Short Period Mapped Spectral Acceleration	0.089 (g)
S_1=1-second Period Mapped Spectral Acceleration	0.047 (g)
Fa = Short Period Site Coefficient	1.2 (unitless)
Fv = 1-sec. Period Site Coefficient	1.7 (unitless)
$SM_s = S_s \times Fa$	0.106(g)
$SM_1 = S_1 \times Fv$	0.090(g)
$SD_s = 0.667 \times SM_s$	0.071 (g)
$SD_1 = 0.667 \times SM_1$	0.053 (g)
Seismic Design Category	A

6.9 SITE PREPARATION AND CONSTRUCTION

6.9.1 Subgrade Preparation

Topsoil, surface vegetation, and unsuitable subgrade soils should be removed from areas planned for development. Based on the soil borings, the topsoil was approximately 3 to 5 inches thick and asphalt was approximately 4 to 5 inches thick. Greater topsoil and asphalt thicknesses may be encountered in other areas. After stripping to the design subgrade elevations, the exposed subgrade should either be proof rolled with a fully loaded, quad-axle dump truck with a minimum 10-cubic yard capacity to detect unstable soil; or, if space constraints do not allow proof roll, other appropriate testing methods (such as Static Cone Penetrometer testing) can be used. Overexcavation may be required due to the possible shallow groundwater, presence of fill soils, and variable strength of the near surface soils encountered.

A GZA geotechnical engineer should observe the exposed subgrade conditions and proof roll or subgrade acceptance operations. High-plasticity soils, if encountered, should not be used as fill or backfill under pavements, structures, or slabs. Additionally, unsuitable bearing soil detected during proof rolling should be scarified and compacted in-place, replaced with fill material, or stabilized, as recommended by the geotechnical engineer. After the subgrade is determined to be stable, Site grades may be raised, where needed, by placing and compacting General Fill material. Recommendations regarding fill placement and compaction of fill materials are provided in Section 6.8.2.

The upper soils encountered at the soil boring locations may be susceptible to disturbance by construction activity, especially if exposed to precipitation and/or surface water. The contractor should expect that soft and potentially unstable subgrades will likely be encountered or developed during construction. It is our experience that even if the near surface soils exhibit a relatively firm or stable surface when initially exposed, repetitive construction traffic and/or wetting will reduce the strength and stability of these soils. We recommend that repetitive traffic on subgrade soils be minimized. Traffic/work mats and/or clear stone gravel layers are recommended for this Site.

6.9.2 Placement and Compaction of Fill

Fill material should be placed on a properly prepared subgrade, as noted in Section 6.8.1. Also, the subgrade or fill soil should not be frozen. Fill soil should be placed and compacted in uniform, loose lifts that are between 6 and 12 inches thick. A GZA geotechnical engineer should determine the actual fill thickness based on characteristics of the fill materials and the type of compaction equipment that is used.

Fill, base course, and backfill materials should be compacted to at least the minimum degree of compaction relative to the maximum dry density determined by the modified Proctor test (ASTM D1557), as noted in the following table. Vibratory compaction methods should be used with caution for fill placed on wet, native soils or near the water table.

During placement, fill soil should have uniform water content within about 2% of the optimum water content determined by the modified Proctor compaction test.

Fill Area	Percent of Maximum Dry Density Determined by ASTM D1557
Below Slabs-on-Grade and Slab-on-Grade Base Course	95
Between Exterior Retaining Walls and Earth Retention Systems	90
Pavement Base Course and Within 2 Feet of Surface Pavement Base Course	95
More Than 2 Feet Below Pavement Base Course	92
Below Landscape Areas	90

We recommend that a GZA geotechnical engineer test each layer of fill to measure in-place dry density and water content. A subsequent layer of fill should not be placed until the density and water content of the prior lift are in accordance with our recommendations. In-place fill should be protected from moisture increases and construction traffic disturbance. Disturbed fill material may be scarified, moisture conditioned, and re-compacted, or it may be replaced with suitable fill material. Properly placed and compacted fill should be protected from freezing. General Fill, as defined in **Appendix E**, can be used to raise Site grades.

6.9.3 On-Site Fill Borrow Material

Based on the boring logs and results of laboratory testing, a majority of the lean clay and fill material present on-Site are suitable for use as General Fill to raise grades below pavements, slabs, or foundations. The on-Site fill is anticipated to be heterogeneous and sections of the fill may not be suitable for use as General Fill. A geotechnical engineer should be on-Site to evaluate the on-Site fill and collect samples for plasticity characterization (Atterberg testing). Soils should be checked and tested to ensure that only granular and/or fine-grained soils with plastic indices less than 18 are used for fill under structural slabs or foundations.

Although not generally encountered at the Site, soils exhibiting high plastic indices are generally harder to moisture condition, harder to adequately compact, and may have some shrink/swell potential after compaction. Soils with liquid limits greater than about 40 and plastic indices greater than 18 should only be used in landscaping areas and not used under slabs, pavements, foundations, or other structural elements. Additionally, relatively high-plasticity clays could be treated and/or blended to reduce the plasticity and allow for more general fill applicability.

6.9.4 Foundations

We recommend that a GZA geotechnical engineer monitor foundation excavation on an on-call basis to confirm that the foundation soil strength is consistent with the design parameters. The geotechnical engineer should perform hand-auger probes and in-situ strength tests at the foundation bearing grade to confirm that the soil within the foundation influence zone can support the design bearing capacity.

Foundation footings should be constructed immediately after excavation and soil testing to protect the soil bearing surface. In addition, footing excavations should be backfilled as soon as possible after foundation construction. Excavations along foundation walls should be filled such that the fill at the interior and exterior sides of the walls are at about the same height as much as practical for lateral pressure considerations. Backfill along foundation walls may consist of General Fill.

6.9.5 Excavation Slopes and Shoring

Based on the Site location and current state of the Site, we understand that space limitations and excavation depths will not likely require an earth retention system for foundation construction. Open-sloped excavations should be feasible for the construction of subsurface grade structures.

Excavations should be in accordance with current United States Department of Labor, Occupational Safety and Health Administration (OSHA) guidelines to protect workers and others during construction. Excavations must be shored, sloped, or benched, as required by OSHA. Per OSHA Standard 29 CFR 1926 Subpart P, the soils present at Site should be classified as Type "C" soils. Excavations should also be in accordance with local, State, and Federal safety regulations. Due to shallow groundwater excavations are expected to slough where groundwater seeps into the excavations. Exposed excavation slope faces should be protected. The geotechnical engineer should evaluate the stability of proposed slopes.

6.10 SHORT- AND LONG-TERM DEWATERING RECOMMENDATIONS

Groundwater was encountered at depths from 3.7 to 21 feet bgs at the Site during drilling. Due to the varied depths of groundwater encountered, additional soil borings and/or groundwater monitoring wells could be considered when building plans are developed to better determine localized groundwater conditions. The presence of a historical stream in the area may lead to increased dewatering demand both during construction and for basements. It is expected that conventional sumps/pumps placed in the excavation may be sufficient for dewatering the expected groundwater infiltration; however, pumping tests should be considered to better estimate groundwater control parameters.

6.11 PAVEMENT RECOMMENDATIONS

We understand that areas of pavement outside of the anticipated structures are planned. Based on the soil and groundwater conditions in the borings, we assume that the subgrade in these areas will generally consist of lean clay and sandy, lean clay soil. Based on our experience within these materials, a CBR value of about 4 was used by GZA to determine the recommended pavement thickness. Similarly, a modulus of subgrade reaction value equal to 90 pci could be used for design of rigid concrete pavement sections placed on compacted subgrade. Prior to placement of base course, subgrade soils should be prepared in accordance with this Report.

Typical pavement thickness recommendations are provided in the following table for asphalt and concrete pavements. The recommended pavement sections are based on an assumed moderate volume of passenger vehicle traffic and low volume of traffic from delivery or garbage trucks, and also should be considered minimum design thicknesses. Thickness recommendations for Passenger Vehicle Parking sections are based on light passenger vehicle traffic (gross weight less than 4 tons) and only occasional truck traffic such as fire trucks and snow removal trucks (2001 Wisconsin Asphalt Paving Association [WAPA] Traffic Class II). The Driveways sections are based on occasional garbage truck, buses, and delivery truck traffic (WAPA Traffic Class III).

MINIMUM PAVEMENT SECTIONS (CBR 4)

Pavement	Passenger Vehicle Parking	Driveways
Portland Cement Concrete (PCC) Granular Base Course	5 inches 6 inches	6 inches 6 inches
Flexible Pavement Section Asphalt Cement Concrete (ACC) Granular Base Course	 4 inches 8 inches	 6 inches 10 inches

1. All materials should meet the current WisDOT Standard Specifications for Highway and Structure Construction.
2. In areas of anticipated heavy traffic, delivery trucks, or concentrated loads (e.g. dumpster pads), a minimum concrete thickness of 7 inches is recommended, but should be evaluated further when loading conditions are known.
3. A minimum 6-inch granular base should be used below PCC pavements.
4. A minimum 1.5-inch surface course should be used on ACC pavements.

The estimated pavement sections provided in this Report are minimums for the assumed design criteria and, as such, periodic maintenance should be expected. Areas for parking of heavy vehicles, concentrated turn areas, and start/stop maneuvers could require thicker pavement sections and may benefit from a geotextile separation layer between the subgrade and granular base course. Final design sections should consider details such as traffic loadings, traffic volumes, the desired design life and any applicable local or City requirements. If you wish, we would be pleased to perform a detailed pavement section design using traffic volumes and American Association of State Highway and Transportation Officials (AASHTO) or ACI procedures when this information is available.

A maintenance program that includes surface sealing, joint cleaning and sealing, and timely repair of cracks and deteriorated areas will increase the pavement's service life. As an option, thicker sections and/or the use of a geotextile separation layer could be constructed to decrease future maintenance.

The pavement base course is recommended to meet the gradation requirement of Structural Fill in **Appendix E**. We recommend that the pavement subgrade be sloped to discharge water to the perimeter edges of the pavement or to a catch basin/drain pipe system.

7.0 FINAL DESIGN AND CONSTRUCTION PHASE SERVICES

After building plans are available, additional geotechnical exploration may need to be conducted for the Site prior to commencement of construction. During construction, we recommend that GZA be retained to observe earthwork activities and geotechnical-related construction for compliance with our recommendations. These activities would include confirming that subsurface conditions encountered during construction are consistent with those anticipated, observation of general excavation work, subgrade preparation for foundations, for slab and pavement base course, foundation construction, slab and pavement base course placement and general fill placement.