KSingh | Engineers Scientists Consultants

STORMWATER MANAGEMENT MEMORANDUM

DATE	:	August 31, 2023
TO	:	City of Wauwatosa
FROM	:	Tom Spieles, PE / KSingh
SUBJECT	:	Secure Residential Care Center for Children and Youth (SRCCCY) - Stormwater
COPY TO	:	Corey Lapworth / Continuum

Introduction

K. Singh & Associates, Inc. (KSingh) was retained to provide Civil Engineering services for the Secure Residential Care Center for Children and Youth (SRCCCY) located at 10201 Watertown Plank Rd, City of Wauwatosa, Milwaukee County, Wisconsin. The property is owned by Milwaukee County, and the parcel area is approximately 16 acres. The site is part of the stormwater management plan for the Milwaukee County Research Park. The total area of study for stormwater modeling is 2.26 acres. This represents the catchment area for the proposed building site that will collect and discharge to the existing site pond. Figure 1 is a location map that illustrates the project site and the proposed disturbance area.

The proposed development includes a building addition, parking, and driveway pavement to the existing Vel R. Phillips Juvenile Justice Center. A proposed fire access drive connects the south lot of the building to the existing parking lot drive to the west of the building. Adequate stormwater management features have been provided to meet applicable requirements from the existing Regional Stormwater Plan. The stormwater management approach is for the proposed building addition and installation of associated utilities.

Code Compliance

Please see below for a summary of the proposed site activities:

- Total area of property = 16 acres
- Total area of stormwater analysis = 2.26 acres
- Total Impervious area before construction = 1.40 acres
- Total Impervious after construction = 1.36 acres

The following stormwater-related permits / requirements are triggered:

- Milwaukee County Research Park Stormwater Planning and Design Document
- Wisconsin Department of Natural Resources (WDNR) Water Resources Application for Project Permits (WRAPP)
- WDNR NR151/216

- Department of Safety and Professional Services (DSPS) Review of Private Exterior Site Plumbing

The City of Wauwatosa requires that the site conform with the requirements of the Regional Stormwater Plan, which are as follows:

- Total green space:
    - Minimum 35% pervious surface area.

Please refer to Attachment E to view the Wauwatosa City Attorney Stormwater Management Review which cites the item above. Please refer to Attachments A and B which show the existing and proposed drainage area calculations and the proposed site grading plan.

Method of Analysis

The analysis of the pre- and post-developed site was performed utilizing HydroCAD® Stormwater Analysis software, Version 10.00-26. HydroCAD® uses TR-55 methodology for hydrologic and hydraulic analysis. Rainfall depths for the 1, 2, 10, and 100-year storm events are shown in Table 1 below and on Figure 2.

Table 1 – Storm Event Rainfall Depths

Storm Event (year)	Rainfall (inches) *
1	2.36
2	2.66
10	3.76
100	6.12

*Rainfall data is based on NRCS runoff modeling methodology volume 8 of Atlas 14, published by the U.S. Department of Commerce, National Oceanic and Atmospheric Administration, National Weather Service, 2015.

Limits of Assessment

The existing conditions were developed using a topographic survey dated June 27, 2023 by Chaput Land Surveys.

The analysis and report were prepared using data from the NRCS Web Soil Survey and WDNR Surface Water Data Viewer. A geotechnical report was prepared by Giles Engineering Associates, Inc. on February 13, 2023. The report can be found in Attachment H.

Pre-Development Conditions

The existing 16-acre site is primarily urban commercial land with wetland present along the south side of the parcel. Please refer to Attachment C for location of wetlands on site.

Within the 2.26 acre area of study, the existing surface elevation ranges from 160' at the north end of the site to 144' at the south end of the site at the top of wetland area. The overall topography of the site is relatively flat. Stormwater within the area of study collects on the site and drains to the wetland area to the south.

The subsurface soil conditions of the site are described using geologic data gathered from the NRCS Web Soil Survey website and geotechnical report. According to the NRCS Web Soil Survey, native soils on the site are predominantly Mequon silt loam and Ozaukee silt loam (See Attachment D). Based on the geotechnical report (Attachment H), native soils are generally medium dense to dense clayey sand. Type C soils were used when selecting curve numbers for both existing and proposed conditions.

Post-development Conditions

The total analysis area for the site is 2.26 acres. The proposed development consists of a building addition, parking stalls, driveway, landscaping, and utilities. Existing BMPs will be utilized to ensure the site meets stormwater requirements. In total, there are 1.36 acres of impervious area, leaving 0.90 acres of pervious area. Please refer to Attachment A for a breakdown of the proposed site impervious and open space areas along with disturbance limits and area of study.

The proposed overall drainage patterns flow from north to south to the existing pond, which is the same as the existing conditions. The proposed land slopes for the site will be 0.5% to 4.5% on pavement and 0.5% to 14% on green space. The existing pond is located at the southern limit of the site to meet applicable stormwater requirements from the Regional Stormwater Plan. This pond captures flow from approximately two thirds of the parcel.

Stormwater will be conveyed using storm piping, sized for the 10-year, 24-hour storm event and overland flow to swales on the south-west side of the addition. The storm pipes convey the stormwater to the south pond, which ultimately outlet to the City storm sewer on HWY100 and Watertown Plank Rd. Please refer to Attachment F for the existing Milwaukee County storm sewer map and outlet locations. The pipe sizing and layout for the storm sewer plan can be found in Attachment G.

Analysis

Hydrologic and hydraulic analyses were required to determine peak stormwater runoff rates from the 2.26 acre catchment area of study for existing and proposed post-developed conditions. The site is located within a regional stormwater management study area with stormwater management facilities designed to reduce the overall peak flows for the Research Park. Peak inflows to the south pond were analyzed and the overall results are shown in Table 2 below.

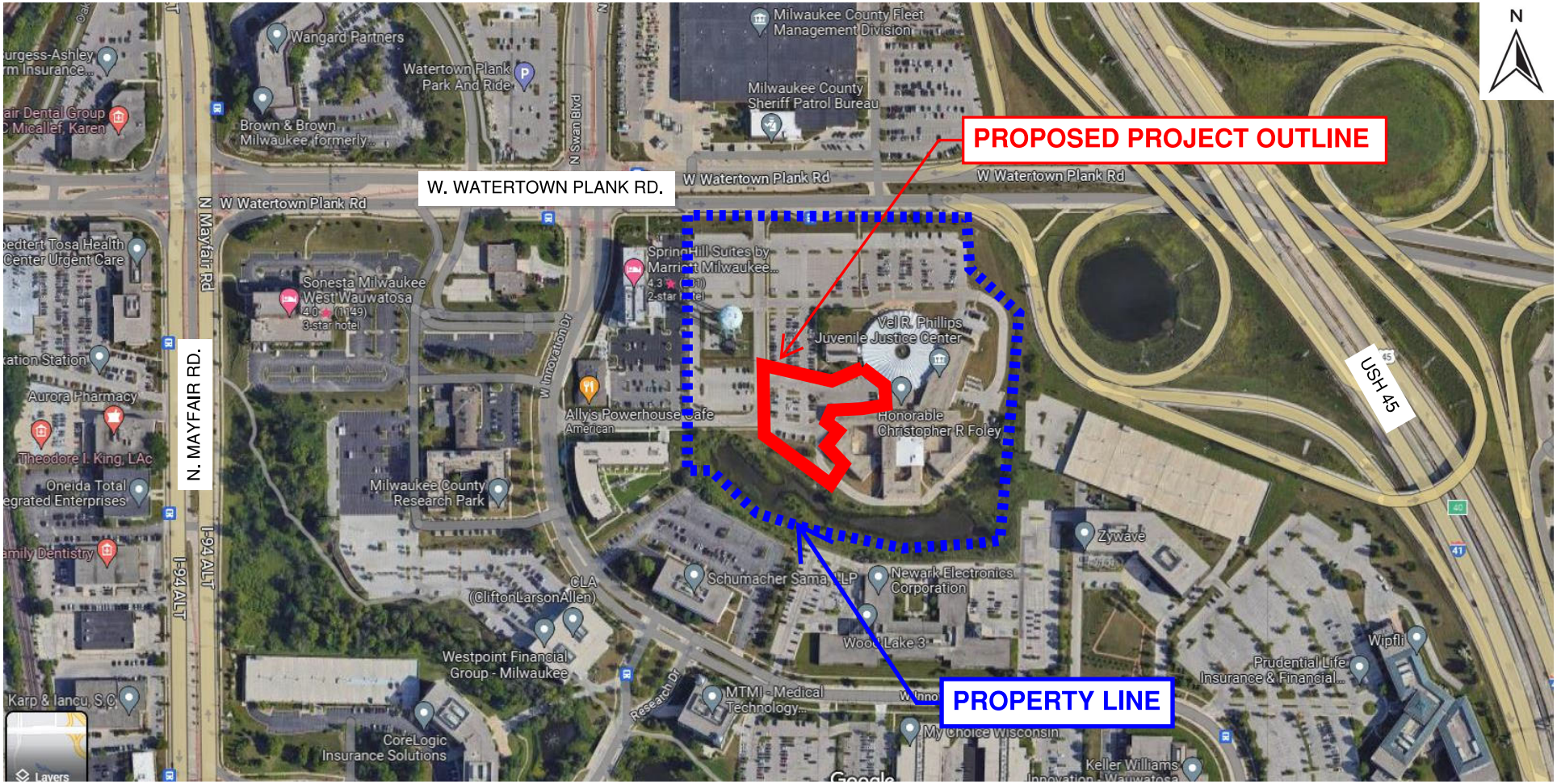
Table 2 – Overall Site Existing vs. Proposed Peak Flows

Storm Event (Year)	Existing (cfs)	Proposed (cfs)
1	5.27	5.03
2	6.28	6.03
10	10.00	9.75
100	17.96	17.75

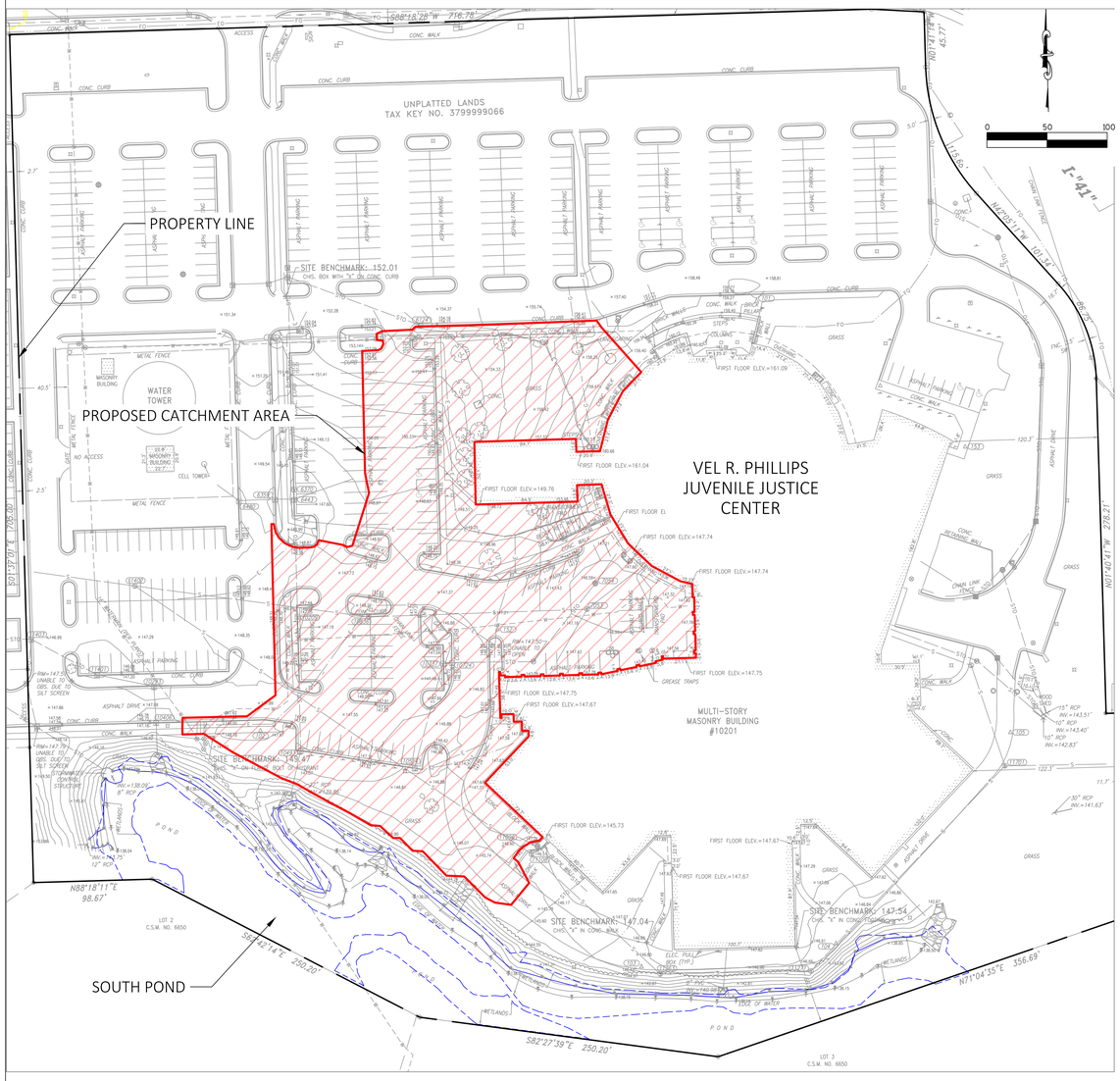
The existing south pond was modeled using direct routing in HydroCAD.

The modeling indicates that the proposed improvements will not increase peak flows to the existing south pond. Attachment A illustrates that the proposed site will meet the 35% green space requirement outlined in the existing Regional Stormwater Management Plan.

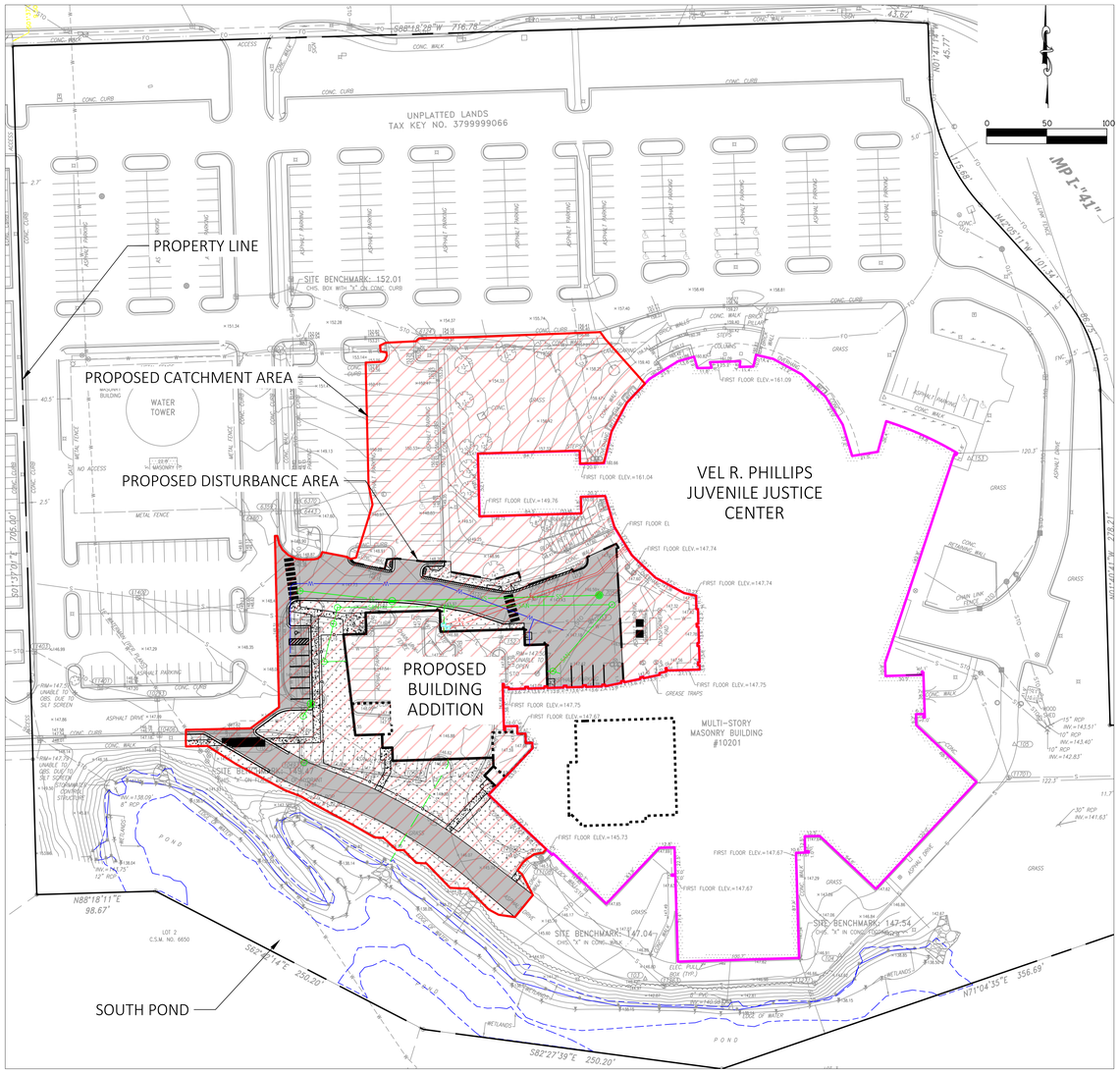
FIGURE 1: VICINITY MAP



ATTACHMENT A: STORMWATER IMPACT EXHIBIT



EXISTING CONDITION



PROPOSED CONDITION

LEGEND

- PROPOSED CONCRETE PAVEMENT/SIDEWALK
- PROPOSED LANDSCAPING
- PROPOSED HMA PAVMENT
- STORMWATER CATCHMENT AREA
- EXISTING VEL R. PHILLIPS JUVENILE JUSTICE CENTER
- WETLAND INDICATOR
- CONSTRUCTION DISTURBANCE LIMITS
- PARCEL BOUNDARY
- PROPOSED BOLLARD
- PROPOSED SANITARY SEWER
- PROPOSED WATER PIPE
- PROPOSED STORM SEWER
- PROPOSED ELECTRICAL LINE

NOT FOR CONSTRUCTION

THESE DRAWINGS ARE RELEASED FOR DESIGN DEVELOPMENT PURPOSES ONLY. THEY ARE NOT TO BE USED FOR REGULATORY APPROVAL, PERMIT, OR CONSTRUCTION PURPOSES .

SECURE RESIDENTIAL CARE CENTER FOR CHILDREN AND YOUTH AT VEL R. PHILLIPS JUVENILE JUSTICE CENTER

MKE COUNTY ASSET OR FACILITY

10201 W Watertown Plank Rd Milwaukee, WI 53226

T 414.220.9640
751 N Jefferson St.
Suite 200
Milwaukee, WI 53202

MILWAUKEE CO. DEPT. OF ADMINISTRATIVE SERVICES
FACILITIES MANAGEMENT DIVISION
ARCHITECTURE - ENGINEERING - ENVIRONMENTAL SERVICES
633 WEST WISCONSIN AVENUE, SUITE 1000 MILWAUKEE, WI 53203

REVISIONS:

DATE:
08/31/2023

COUNTY PROJECT #:
WS12401

SITE NO:
360

BUILDING NO:
500

CAP PROJECT #:
180802

PROJECT STATUS:
DD SET

ATTACHMENT B: PROPOSED GRADING PLAN

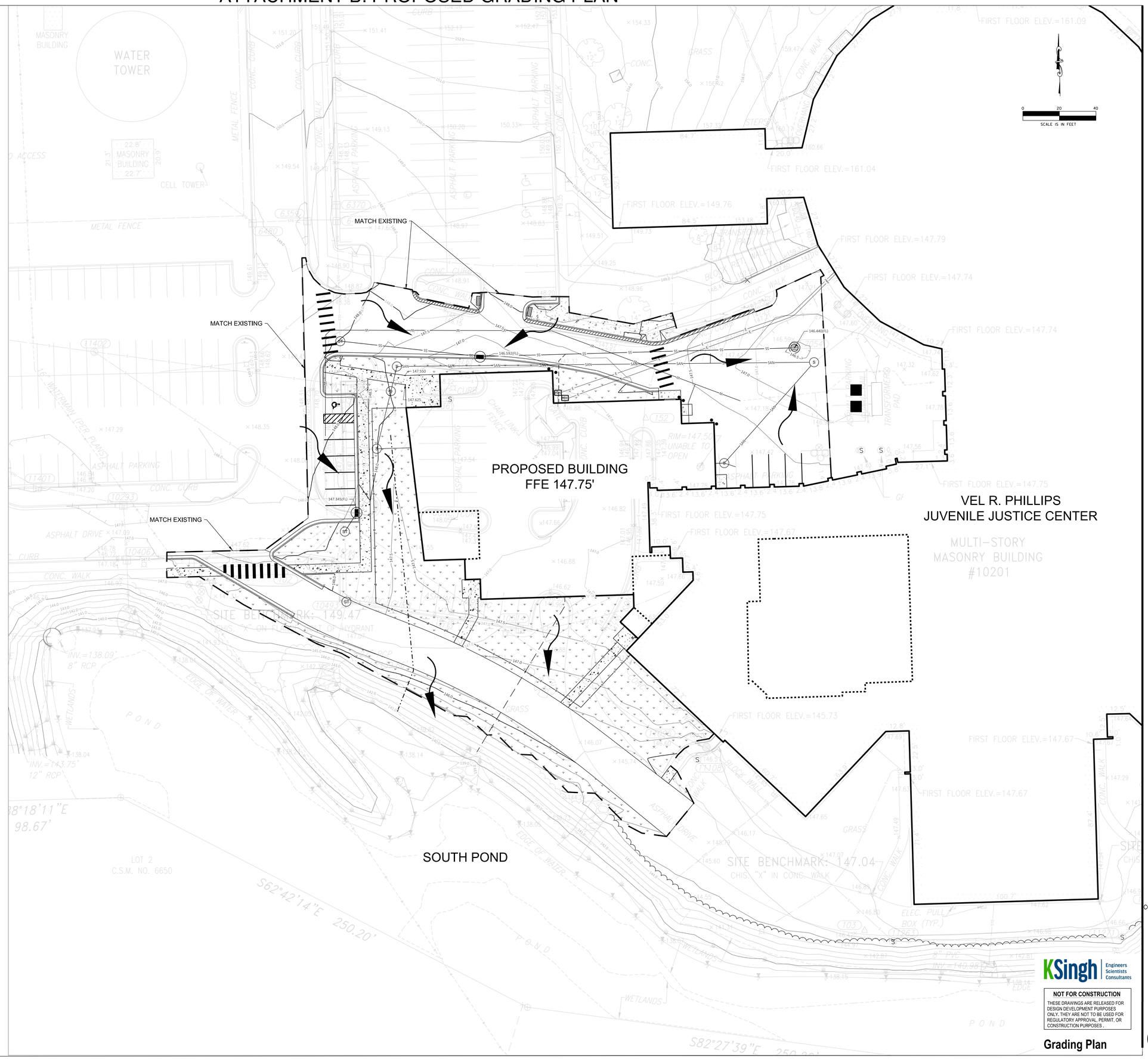
GENERAL NOTES:

1. REFER TO DEMOLITION PLAN C100 FOR REMOVAL OF EXISTING UTILITIES, STRUCTURES, AND PAVEMENT REMOVAL.
2. GRADE ALL SURFACES FOR POSITIVE DRAINAGE AND AWAY FROM BUILDINGS.
3. ALL GRADES SHOWN ARE SURFACE OR FLOW LINE GRADES, UNLESS OTHERWISE NOTED.
4. CONTRACTOR TO STRIP AND STOCKPILE TOPSOIL FROM SITE BEFORE GRADING.
5. PAVEMENT RESTORATIONS SHALL MEET ADJACENT EXISTING PAVEMENTS AS A FLUSH MATCH.
6. FINAL ADJUSTMENTS SHALL BE MADE AT THE TIME OF PAVING TO MATCH THE PAVEMENT SURFACE.
7. ALL UTILITIES WITHIN THE PROJECT LIMITS SHALL BE ADJUSTED TO PROPOSED GRADING BY THE CONTRACTOR, UNLESS A GRADE MATCH IS INDICATED AT A SPECIFIC UTILITY
8. THE CONTRACTOR SHALL PAY ALL FEES, PERMITS, AND DEPOSITS FOR THE ADJUSTMENT WORK.
9. ALL ADJUSTMENT WORK SHALL BE TO THE SPECIFICATIONS OF THE UTILITY OWNER AND IN ACCORDANCE WITH ANY REQUIRED PERMITS.
10. ALL SIDEWALK GRADES SHALL MEET ADA REQUIREMENTS:
    - 10.1. CROSS SLOPES OF SIDEWALKS SHALL NOT EXCEED 2.0%
    - 10.2. LONGITUDINAL SLOPES OF SIDEWALKS SHALL NOT EXCEED 5.0%
11. REFER TO SITE PLAN AND LANDSCAPING PLANS FOR RESTORATION DETAILS.
12. CONTRACTOR TO VERIFY MATCH EXISTING GRADES PRIOR TO BEGINNING WORK AND DISCUSS AND DISCREPANCIES WITH ENGINEER.

DIGGERS HOTLINE
Dial 811 or (800) 242-8511
www.DiggersHotline.com

LEGEND

190.90	PROPOSED SPOT GRADE
ME:190.9 ±	MATCH EXISTING SPOT GRADE
200.90(FL)	PROPOSED SPOT GRADE FLOW LINE
200.90(TOC)	PROPOSED SPOT GRADE TOP OF CURB
FFE:200.90	PROPOSED FINISHED FLOOR ELEVATION
– – – – –	PROPOSED DISTURBANCE LIMITS
— · — · —	PROPOSED OVERLAND FLOW ROUTE
191	PROPOSED GRADING CONTOUR
191	EXISTING GRADING CONTOUR
E	PROPOSED UNDERGROUND ELECTRIC (FOR REFERENCE ONLY, SEE ELECTRICAL PLANS FOR DETAILS)
SS	PROPOSED STORM SEWER
W	PROPOSED WATER
SAN	PROPOSED SANITARY
	DRAINAGE FLOW PATH
	PROPOSED CATCH BASIN / CURB INLET
ST	PROPOSED STORM MANHOLE
S	PROPOSED SANITARY MANHOLE



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CAP PROJECT #: **180802**

PROJECT STATUS: **DD SET**

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Grading Plan

C300

FIGURE 2: NOAA RAINFALL DATA

NOAA Atlas 14, Volume 8, Version 2
Location name: Milwaukee, Wisconsin, USA*
Latitude: 43.044°, Longitude: -88.0407°
Elevation: 727 ft**
* source: ESRI Maps
** source: USGS

POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Deborah Martin, Sandra Pavlovic, Ishani Roy, Michael St. Laurent, Carl Trypaluk, Dale Unruh, Michael Yekta, Geoffery Bonnin

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.328 (0.256-0.411)	0.394 (0.306-0.493)	0.498 (0.387-0.625)	0.584 (0.452-0.734)	0.700 (0.526-0.891)	0.787 (0.583-1.01)	0.873 (0.631-1.14)	0.959 (0.673-1.26)	1.07 (0.730-1.43)	1.15 (0.773-1.56)
10-min	0.481 (0.375-0.602)	0.576 (0.449-0.721)	0.730 (0.567-0.915)	0.855 (0.661-1.07)	1.02 (0.771-1.30)	1.15 (0.854-1.48)	1.28 (0.924-1.66)	1.40 (0.985-1.85)	1.57 (1.07-2.10)	1.69 (1.13-2.28)
15-min	0.587 (0.457-0.734)	0.703 (0.547-0.880)	0.890 (0.691-1.12)	1.04 (0.807-1.31)	1.25 (0.940-1.59)	1.40 (1.04-1.80)	1.56 (1.13-2.03)	1.71 (1.20-2.26)	1.91 (1.30-2.56)	2.06 (1.38-2.79)
30-min	0.819 (0.638-1.02)	0.984 (0.766-1.23)	1.25 (0.970-1.56)	1.46 (1.13-1.84)	1.76 (1.32-2.23)	1.97 (1.46-2.53)	2.19 (1.58-2.85)	2.40 (1.69-3.17)	2.68 (1.83-3.59)	2.88 (1.93-3.90)
60-min	1.06 (0.824-1.32)	1.27 (0.987-1.59)	1.62 (1.26-2.03)	1.91 (1.48-2.40)	2.32 (1.76-2.98)	2.64 (1.96-3.41)	2.97 (2.16-3.88)	3.31 (2.33-4.38)	3.76 (2.57-5.06)	4.11 (2.76-5.57)
2-hr	1.30 (1.02-1.60)	1.55 (1.22-1.92)	1.98 (1.56-2.46)	2.36 (1.85-2.93)	2.89 (2.22-3.68)	3.32 (2.50-4.24)	3.76 (2.76-4.87)	4.22 (3.00-5.54)	4.84 (3.35-6.47)	5.34 (3.62-7.17)
3-hr	1.45 (1.16-1.78)	1.72 (1.37-2.12)	2.20 (1.74-2.71)	2.62 (2.07-3.23)	3.24 (2.51-4.12)	3.75 (2.85-4.78)	4.29 (3.18-5.54)	4.86 (3.49-6.38)	5.66 (3.95-7.54)	6.30 (4.30-8.42)
6-hr	1.74 (1.40-2.12)	2.02 (1.63-2.46)	2.54 (2.04-3.09)	3.01 (2.41-3.67)	3.74 (2.95-4.73)	4.36 (3.37-5.53)	5.03 (3.79-6.47)	5.77 (4.20-7.52)	6.82 (4.82-9.03)	7.68 (5.28-10.2)
12-hr	2.06 (1.68-2.48)	2.33 (1.90-2.80)	2.84 (2.31-3.42)	3.33 (2.70-4.02)	4.10 (3.29-5.15)	4.77 (3.74-6.01)	5.52 (4.21-7.04)	6.34 (4.68-8.20)	7.53 (5.38-9.89)	8.52 (5.92-11.2)
24-hr	2.36 (1.95-2.80)	2.66 (2.20-3.16)	3.22 (2.66-3.83)	3.76 (3.08-4.48)	4.60 (3.73-5.70)	5.32 (4.22-6.62)	6.12 (4.72-7.72)	7.00 (5.22-8.96)	8.27 (5.97-10.8)	9.32 (6.54-12.1)
2-day	2.63 (2.20-3.08)	3.02 (2.54-3.55)	3.73 (3.12-4.39)	4.38 (3.64-5.16)	5.35 (4.38-6.53)	6.17 (4.94-7.56)	7.05 (5.48-8.76)	8.00 (6.01-10.1)	9.34 (6.80-12.0)	10.4 (7.39-13.4)
3-day	2.88 (2.44-3.36)	3.30 (2.78-3.84)	4.04 (3.40-4.71)	4.71 (3.95-5.51)	5.71 (4.71-6.92)	6.56 (5.28-7.98)	7.46 (5.84-9.22)	8.44 (6.39-10.6)	9.82 (7.19-12.5)	10.9 (7.80-14.0)
4-day	3.10 (2.64-3.60)	3.53 (3.00-4.10)	4.29 (3.63-4.98)	4.97 (4.19-5.79)	6.00 (4.97-7.22)	6.86 (5.55-8.30)	7.78 (6.12-9.56)	8.77 (6.66-11.0)	10.2 (7.48-12.9)	11.3 (8.10-14.4)
7-day	3.65 (3.14-4.19)	4.14 (3.55-4.75)	4.98 (4.27-5.74)	5.73 (4.89-6.62)	6.84 (5.71-8.12)	7.74 (6.32-9.26)	8.70 (6.90-10.6)	9.72 (7.44-12.0)	11.1 (8.25-14.0)	12.3 (8.86-15.5)
10-day	4.14 (3.59-4.73)	4.68 (4.05-5.34)	5.60 (4.84-6.41)	6.40 (5.50-7.35)	7.56 (6.34-8.91)	8.50 (6.98-10.1)	9.48 (7.56-11.4)	10.5 (8.09-12.9)	11.9 (8.88-14.9)	13.1 (9.47-16.4)
20-day	5.64 (4.96-6.36)	6.29 (5.52-7.10)	7.37 (6.45-8.33)	8.28 (7.21-9.38)	9.54 (8.08-11.0)	10.5 (8.74-12.3)	11.5 (9.29-13.7)	12.6 (9.76-15.2)	14.0 (10.5-17.2)	15.0 (11.0-18.7)
30-day	6.95 (6.16-7.78)	7.72 (6.84-8.65)	8.96 (7.91-10.1)	9.98 (8.76-11.2)	11.4 (9.67-13.0)	12.4 (10.3-14.3)	13.4 (10.9-15.8)	14.4 (11.3-17.3)	15.8 (11.9-19.3)	16.8 (12.4-20.8)
45-day	8.66 (7.74-9.62)	9.62 (8.60-10.7)	11.1 (9.92-12.4)	12.3 (10.9-13.8)	13.9 (11.9-15.7)	15.0 (12.6-17.2)	16.1 (13.1-18.8)	17.1 (13.4-20.3)	18.4 (13.9-22.2)	19.2 (14.3-23.7)
60-day	10.1 (9.11-11.2)	11.3 (10.2-12.5)	13.1 (11.8-14.6)	14.5 (12.9-16.1)	16.3 (14.0-18.3)	17.5 (14.7-19.9)	18.6 (15.2-21.5)	19.6 (15.4-23.1)	20.8 (15.8-25.0)	21.5 (16.0-26.4)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

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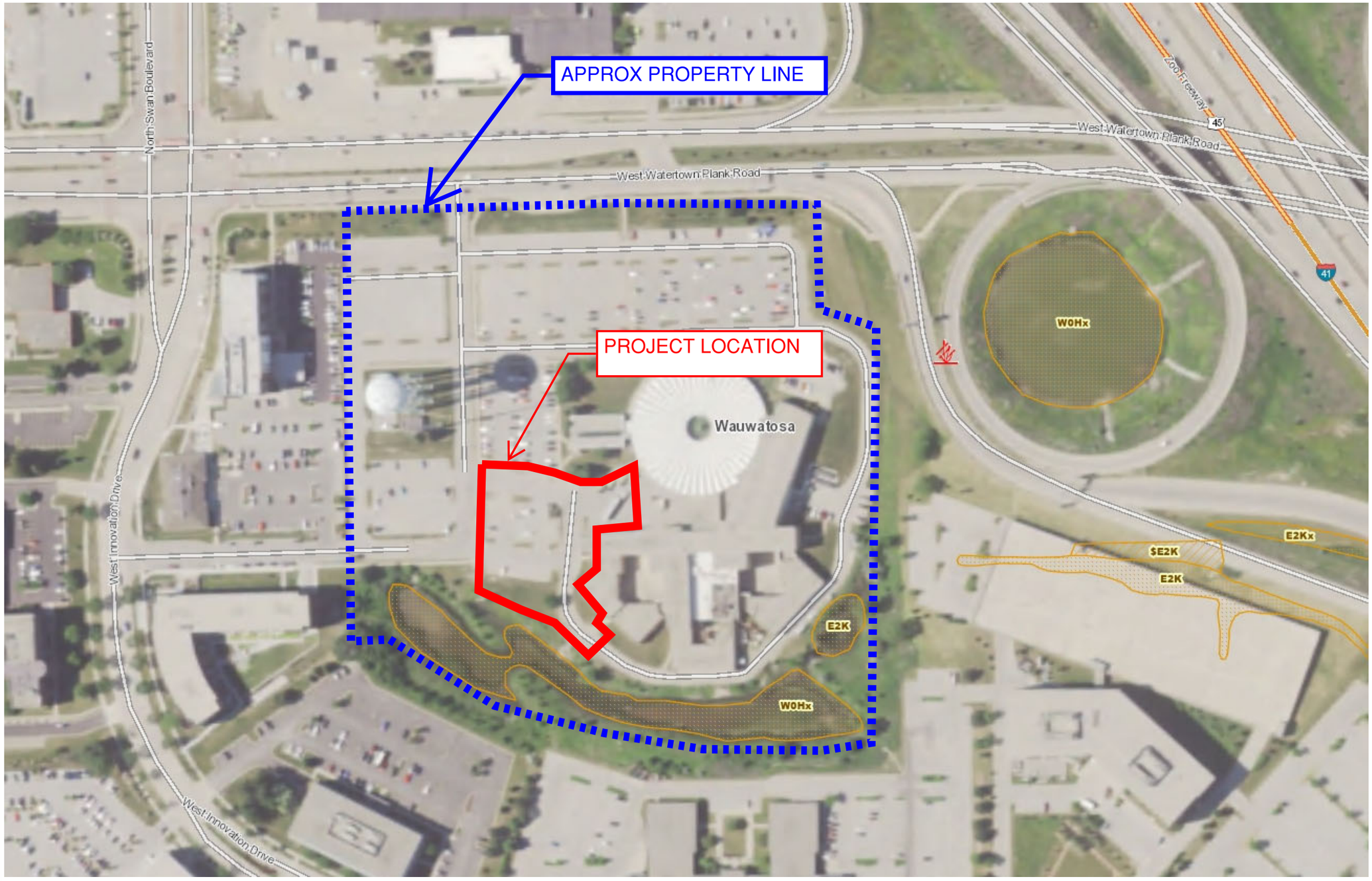
PF graphical

ATTACHMENT C: Surface Water Data Viewer Map



Legend

- Wetland Class Areas
- Wetland Class Points
  - Dammed pond
  - Excavated pond
  - Filled/drained wetland
  - Wetland too small to delineate
  - Filled excavated pond
- Filled Points
- Wetland Class Areas
- Filled Areas
- Wetland Identifications and Confirmations
- Railroads

0.1 0 0.03 0.1 Miles

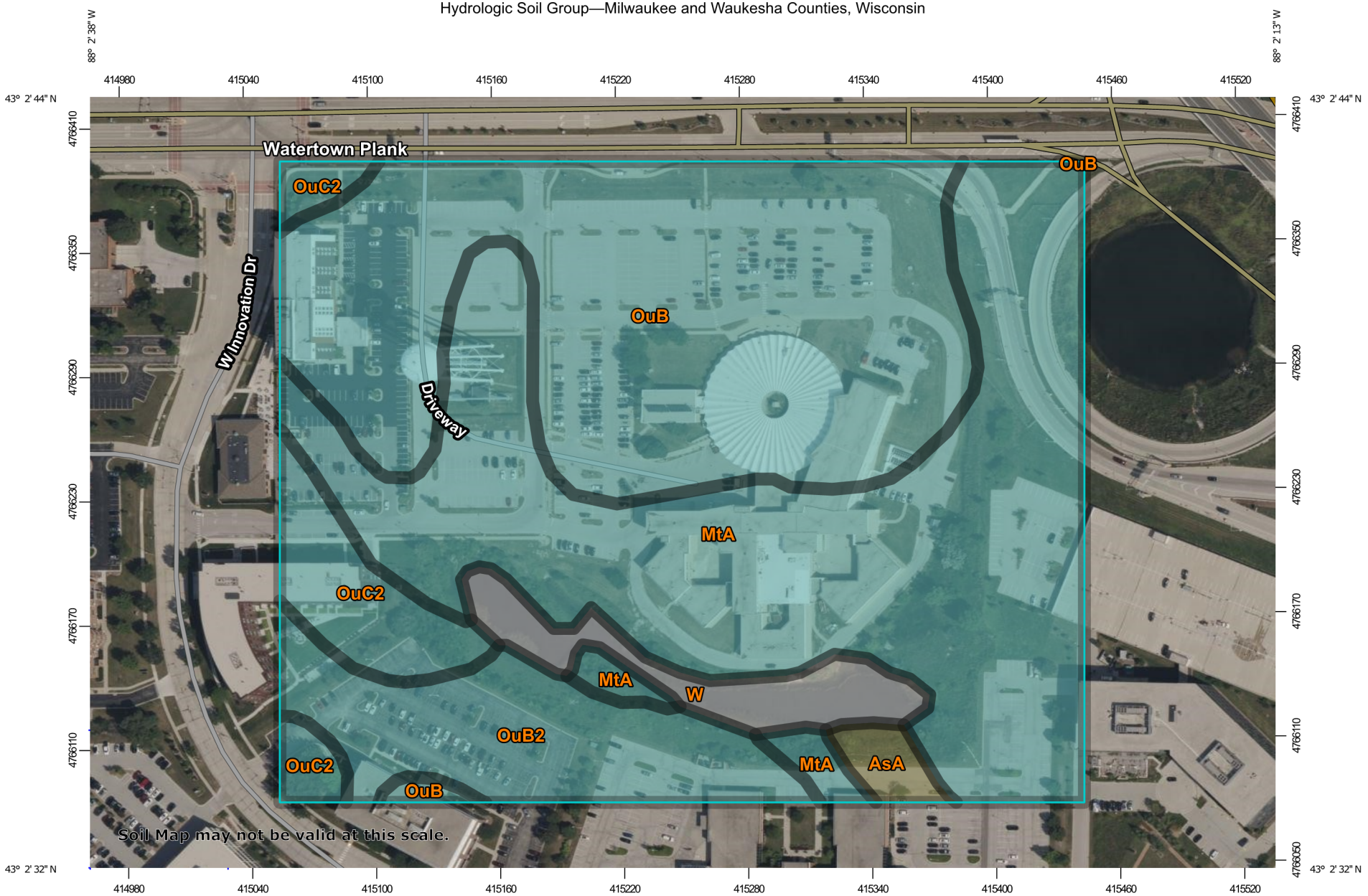
1:1,980

NAD_1983_HARN_Wisconsin_TM

Notes

ATTACHMENT D: HSG MAP

Hydrologic Soil Group—Milwaukee and Waukesha Counties, Wisconsin



Map Scale: 1:2,620 if printed on A landscape (11" x 8.5") sheet.

N

0 35 70 140 210 Meters

0 100 200 400 600 Feet

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 18, Sep 7, 2022

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

Date(s) aerial images were photographed: Aug 4, 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
AsA	Ashkum silty clay loam, 0 to 2 percent slopes	C/D	0.4	1.2%
MtA	Mequon silt loam, 1 to 3 percent slopes	C	12.4	41.4%
OuB	Ozaukee silt loam, high carbonate substratum, 2 to 6 percent slopes	C	10.9	36.5%
OuB2	Ozaukee silt loam, high carbonate substratum, 2 to 6 percent slopes, eroded	C	3.1	10.5%
OuC2	Ozaukee silt loam, high carbonate substratum, 6 to 12 percent slopes, eroded	C	1.8	6.1%
W	Water		1.3	4.3%
Totals for Area of Interest			**29.9**	**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

ATTACHMENT E: CITY ATTORNEY LETTER

Alan R. Kesner

Assistant City Attorneys

Beth Thorson Aldana
Eileen Miller Carter

Special Assistants

George R. Schimmel
George M. Schimmel

Wauwatosa City Attorney

July 14, 2004

Mr. Guy T. Mascari
Director of Development
Milwaukee County Research Park
10437 Innovation Drive, Suite 123
Wauwatosa, WI 53226-4815

RE: Storm Water Management Review for Milwaukee County Research Park

Dear Guy:

I am writing in response to your letter to Tom Wontorek dated July 8, 2004. With that letter you provided certain documentation regarding storm water management at the Research Park, and you have requested City confirmation in writing of compliance with all appropriate storm water regulations and exemption from the MMSD Chapter 13 rules. For purposes of this letter, I will adopt and utilize the definitions and terms in your July 8, 2004 letter.

The City has reviewed the documents you provided in conjunction with your letter, and your representations of compliance with the Planning Documents. Based upon the representation that the pervious/impervious surface ratio in the Design Standards have been complied with, the Research Park, as a whole, would be in compliance with the Planning Documents with respect to existing development at the Research Park, including the new GE Healthcare Development. So long as future development within the Research Park continues to keep the Research Park, overall, in compliance with the Design Standards' minimum 35% pervious surface requirement as contemplated in the Planning Documents, the Research Park would continue to be in compliance with the Planning Documents overall.

By this letter, the City reaffirms its prior approval of the Pond Construction Documents and Roadway and Utility Documents.

As long as the Research Park, including existing or future individual projects within the Research Park, remains in compliance with the Planning Documents, it shall not be subject to any City Storm Water Quantity Management Ordinances which have been adopted between the date of the prior City approval and the current date. This exemption includes an exemption to MMSD Regulations Chapter 13 – Surface Water and Storm

7725 West North Avenue Wauwatosa, Wisc[illegible] Phone: (414) 479-8905 Fax: (414) 471-8414
ht

MEMO TO FILE

RE: RESEARCH PARK STORMWATER MANAGEMENT PLAN

7/14/04

A telephone call was received from Bill Wehrley, Wauwatosa City Engineer, concerning the Milwaukee County Research Park (west of 45, south of Watertown Plank, north of Wisconsin Avenue, east of Hwy 100). He indicated that his legal staff was putting together an opinion concerning the applicability of MMSD's Chapter 13 on this site. He believed that this site had an approved stormwater management plan in place prior to the effective date of the rule and, therefore, it did not need to comply with Chapter 13. He was soliciting MMSD's opinion.

A records search revealed the storm sewer plan submittal and sanitary sewer submittal from 1999. These records indicated that: 40% of the site was to be buildings and there would be a minimum of 35% green space, which would leave 25% for other impervious features (parking, roads, sidewalks, etc.).

Because the ponds were built in 1999-2000 and the sizing was based a minimum of 35% green space, it was agreed that:

- this is a stormwater management plan that was approved prior to 1/1/02;
- the site (Research Park) could be constructed without undertaking additional stormwater management if it maintained a minimum of 35% green space.

If the site has construction that minimizing the green space and the green space becomes less than 35%, then the construction that is over that indicated above (40% buildings, 25% other imperviousness) would need to follow Chapter 13.

ATTACHMENT F: MILWAUKEE COUNTY STORM SEWER MAP

STORM SEWER LEGEND

ACTIVE COUNTY STORM SEWER

- DWG. NO. ON WHICH MANHOLE APPEARS.
- STORM SEWER MANHOLE.
- MANHOLE NUMBER.
- STORM SEWER DIRECTION OF FLOW.
- ABANDONED COUNTY STORM SEWER
- MANHOLE WITH CATCH BASIN COVER.
- CATCH BASIN OR CURB INLET.
- WAUWATOSA OWNED STORM SEWER
- OWNED BY OTHERS
- MANHOLE NUMBER.
- ALL LINES WITHOUT SIZE INDICATED ARE ROOF DRAINS.
- STORM SEWER SIZE. INVERT ELEVATIONS.
- MANHOLE RIM ELEVATIONS

0 200 400 800

SHEET NO. 1 OF 1	PROJECT NO.	TITLE: MILWAUKEE COUNTY GROUNDS STORM SEWER SYSTEM	SCALE: 1"=200'	DRAWN BY: BRB / MJS	REVISIONS 3/08/06 GENERAL REVISIONS 08/13/12 GENERAL REVISIONS	PREPARED BY: DTPW SUPPORT SERVICES–GIS SECTION FOR MILWAUKEE COUNTY DEPARTMENT OF TRANPORTATION AND PUBLIC WORKS 10310 WATERTOWN PLANK ROAD MILWAUKEE, WISCONSIN 53226
SHEET SIZE E	FILE NO. STALL		DATE: 06/13/12	CHECKED BY: TOM T.		

UTILITY NOTES:

1. DURING CONSTRUCTION ONE OR MORE OF THE FOLLOWING SHALL APPLY:
   1.1. CONTRACTOR SHALL VERIFY ALL EXISTING UTILITY LOCATION (VERTICALLY AND HORIZONTALLY) PRIOR TO CONSTRUCTION.
   1.2. ALL UTILITIES NOT INDICATED TO BE REMOVED/ABANDONED ARE CONSIDERED CRITICAL AND MUST BE MAINTAINED AND PROTECTED DURING CONSTRUCTION.
   1.3. DAMAGES TO UTILITIES RESULTING FROM CONSTRUCTION WORK AND EXCAVATION SHALL BE IMMEDIATELY REPORTED TO THE UTILITY OWNERS AND UTILITY USERS. THE CONTRACTOR IS RESPONSIBLE FOR ALL WORK AND COSTS ASSOCIATED WITH CORRECTING THE DAMAGES.
   1.4. SITE FEATURES THAT ARE DETERMINED BY THE CONTRACTOR TO BE TEMPORARILY MOVED TO PROVIDE SUFFICIENT CLEARANCE FOR CONSTRUCTION ACTIVITIES SHALL BE REMOVED, SALVAGED AND REPLACED IN-KIND OR IN BETTER CONDITION UPON COMPLETION OF THE CONSTRUCTION WORK. COORDINATE SITE FEATURE TEMPORARY MOVE WITH OWNER.
2. REFER TO C100 FOR UTILITY DEMOLITION NOTES AND COORDINATION.
3. CONNECT AND INSTALL ALL SANITARY, STORM AND WATER SERVICES IN ACCORDANCE WITH THE CITY OF WAUWATOSA STANDARDS AND SPECIFICATIONS.
4. MAINTAIN CODE REQUIRED SEPARATION BETWEEN SEWER AND WATER FACILITIES AS LISTED IN THE SPECIFICATIONS.
5. PIPE LENGTHS AND SLOPES ARE TO CENTER OF STRUCTURE, UNLESS OTHERWISE NOTED.
6. CONTRACTOR SHALL ADJUST ALL EXISTING AND NEW UTILITY STRUCTURES AS NEEDED FOR FINAL GRADING.
7. MAINTAIN PROPER DRAINAGE AT ALL TIMES DURING CONSTRUCTION.
8. WATER SERVICES SHALL BE DUCTILE IRON (D.I.) CLASS 55, AWWA C151/A21.51-96 IN ACCORDANCE WITH CITY OF WAUWATOSA SPECIFICATIONS.
9. CONTRACTOR SHALL VERIFY EXISTING PIPE INVERT, PIPE MATERIAL, PIPE SIZE AND LOCATION PRIOR TO CONSTRUCTION OF THE UTILITIES.
10. A MINIMUM OF 6.0 FEET OF COVER SHALL BE MAINTAINED OVER ALL WATER MAIN. VERIFY ALL EXISTING INVERT ELEVATIONS, SIZES AND MATERIALS PRIOR TO CONSTRUCTION.
11. CONTRACTOR SHALL ADJUST ALL EXISTING AND PROPOSED UTILITY CASTINGS, MANHOLES, CLEANOUTS, AND ACCESS BOXES TO PROPOSED GRADING.
12. PROVIDE TRACER WIRE FOR NON-METALLIC UTILITY SERVICES. SEE SPECIFICATIONS.
13. STORM SEWER PIPING SHALL BE PVC, SDR 35 & CLASS IV REINFORCED CONCRETE IN ACCORDANCE WITH THE SPECIFICATIONS. SANITARY PIPING SHALL BE PVC, SDR 26.
14. WATER SERVICE OUTAGES SHALL BE LIMITED TO ONE 4-HOUR SHUTDOWN WITHIN A 24-HOUR PERIOD. PRIOR TO BEGINNING, CONTRACTOR SHALL PROVIDE A CONSTRUCTION SCHEDULE. DO NOT INTERRUPT EXISTING WATER SUPPLY WITHOUT APPROVAL FROM THE MUNICIPAL WATER UTILITY AND LOCAL FIRE DEPARTMENT. SEE SPECIFICATIONS FOR DETAILS AND ADDITIONAL REQUIREMENTS.
15. ALL UTILITY WORK WITHIN THE CITY RIGHT OF WAY SHALL COMPLY WITH THE CITY OF WAUWATOSA STANDARD SPECIFICATIONS.
16. REFER TO PLUMBING PLANS FOR BUILDING CONTINUATION OF WATER, SANITARY AND STORM UTILITIES AND CLEANOUTS. ALL SITE WATER, SANITARY AND STORM BUILDING CONNECTIONS SHOWN 5-FEET OUTSIDE OF BUILDING.
17. ELECTRICAL UTILITIES SHOWN IN APPROXIMATE LOCATIONS. REFER TO ELECTRICAL PLANS FOR ELECTRICAL DETAILS.

LEGEND:

- PROPOSED SANITARY SEWER
- PROPOSED WATER UTILITY
- PROPOSED STORM SEWER
- PROPOSED UNDERGROUND ELECTRIC UTILITY *
- ROOF DRAIN PIPE
- STORM MANHOLE
- SANITARY MANHOLE
- CLEANOUT
- CATCH BASIN / CURB INLET
- PROPOSED TAPPING SLEEVE, GATE VALVE, AND VALVE BOX
- PROPOSED CONCRETE PAVEMENT/SIDEWALK
- LANDSCAPING, SEE LANDSCAPE PLANS FOR DETAILS
- PROPOSED HMA PAVMENT

* FOR REFERENCE ONLY, SEE ELECTRICAL PLANS FOR DETAILS

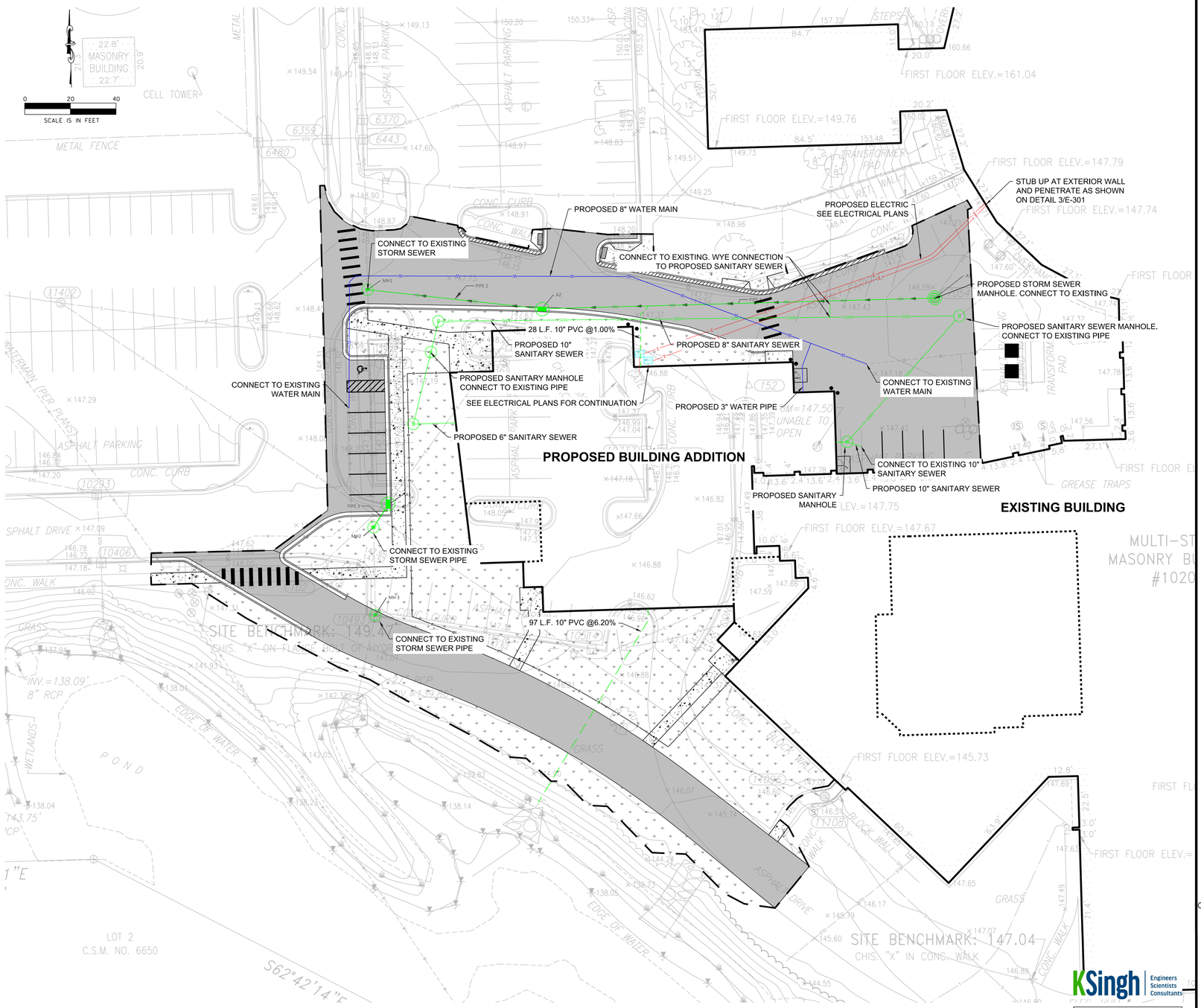
UTILITY DETAILS:

1. STORM SEWER MANHOLE / INLET (3/C502) / (9/C502)
2. PIPE TRENCHING (1|2/C502)
3. MANHOLE PIPE CONNECTION (4/C502)
4. SANITARY MANHOLE (8/C502)
5. TAPPING SLEEVE AND GATE VALVE ASSEMBLY (6/C502)
6. GATE VALVE IN A VALVE BOX (5/C502)
7. WYE CONNECTION DETAIL (7/C502)

STORM SEWER STRUCTURES AND PIPE TABLES

STRUCTURE TABLE				
STRUCTURE NAME	TYPE	RIM	PIPE IN	PIPE OUT
A1	MANHOLES 4-C	146.440		142.380
A2	MANHOLES 4-C	146.597	141.414	141.414
A3	MANHOLES 4-C	147.376		140.361
MH 3	MANHOLES 4-C	147.682		
MH1	MANHOLES 4-C	147.975	141.030	
MH2	MANHOLES 4-C	147.913	140.220	

PIPE TABLE				
PIPE NUMBER	SIZE (IN)	LENGTH (FT)	SLOPE	PIPE DESCRIPTION
PIPE 1	15.000	172.234	0.56%	SSPRC CLASS II 15-INCH
PIPE 2	15.000	76.726	0.50%	SSPRC CLASS II 15-INCH
PIPE 3	12.000	11.994	1.18%	SSPRC CLASS II 12-INCH



ATTACHMENT G: PROPOSED UTILITIES

SECURE RESIDENTIAL CARE CENTER FOR CHILDREN AND YOUTH AT VEL R. PHILLIPS JUVENILE JUSTICE CENTER

MKE COUNTY ASSET OR FACILITY

10201 W Watertown Plank Rd Milwaukee, WI 53226

T 414.220.9640
751 N Jefferson St.
Suite 200
Milwaukee, WI 53202

MILWAUKEE CO. DEPT. OF ADMINISTRATIVE SERVICES
FACILITIES MANAGEMENT DIVISION
ARCHITECTURE - ENGINEERING - ENVIRONMENTAL SERVICES
633 WEST WISCONSIN AVENUE, SUITE 1000 MILWAUKEE, WI 53203

REVISIONS:

DATE:

COUNTY PROJECT #: **WS12401**

SITE NO: **360**

BUILDING NO: **500**

CAP PROJECT #: **180802**

PROJECT STATUS: **DD SET**

KSingh Engineers Scientists Consultants

NOT FOR CONSTRUCTION

THESE DRAWINGS ARE RELEASED FOR DESIGN DEVELOPMENT PURPOSES ONLY. THEY ARE NOT TO BE USED FOR REGULATORY APPROVAL, PERMIT, OR CONSTRUCTION PURPOSES .

Utility Plan

C400

ATTACHMENT H:

Geotechnical Engineering Exploration and Analysis

Proposed Improvements
Vel R. Phillips Juvenile Justice Center
10201 W. Watertown Plank Road
Wauwatosa, Wisconsin

Prepared for:

Continuum Architects + Planners
Milwaukee, Wisconsin

February 13, 2023
Project No. 1G-2301013

GILES
ENGINEERING ASSOCIATES, INC.

GILES
ENGINEERING ASSOCIATES, INC.
GEOTECHNICAL, ENVIRONMENTAL & CONSTRUCTION MATERIALS CONSULTANTS

• Dallas, TX
• Los Angeles, CA
• Manassas, VA
• Milwaukee, WI

February 13, 2023

Continuum Architects + Planners
751 N. Jefferson Street, Suite 200
Milwaukee, WI 53202

Attention: Mr. Corey Lapworth, NCARB, AIA, CDT
Principal

Subject: Geotechnical Engineering Exploration and Analysis
Proposed Improvements
Vel R. Phillips Juvenile Justice Center
10201 W. Watertown Plank Road
Wauwatosa, Wisconsin
Giles Project No. 1G-2301013

Dear Mr. Lapworth:

As requested, Giles Engineering Associates, Inc. conducted a *Geotechnical Engineering Exploration and Analysis* for the proposed project. The accompanying report describes the services that were performed, and it provides geotechnical-related findings, conclusions, and recommendations that were derived from those services.

We sincerely appreciate the opportunity to provide geotechnical services for the proposed project. Please contact the undersigned if there are questions about the report, or if we may be of further service.

Very truly yours,

GILES ENGINEERING ASSOCIATES, INC.

Andrew J. Globig
Project Professional

Benjamin M. Stark, P.E.
Project Engineer

Distribution: Continuum Architects + Planners
Attn: Mr. Corey Lapworth (pdf: corey.lapworth@continuumarchitect.com)

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GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS

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VEL R. PHILLIPS JUVENILE JUSTICE CENTER
10201 W. WATERTOWN PLANK ROAD
WAUWATOSA, WISCONSIN
PROJECT NO. 1G-2301013

GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS

PROPOSED IMPROVEMENTS
VEL R. PHILLIPS JUVENILE JUSTICE CENTER
10201 W. WATERTOWN PLANK ROAD
WAUWATOSA, WISCONSIN
PROJECT NO. 1G-2301013

1.0 SCOPE OF SERVICES

This report provides the results of the *Geotechnical Engineering Exploration and Analysis* that Giles Engineering Associates, Inc. ("Giles") conducted for the proposed project. The *Geotechnical Engineering Exploration and Analysis* included a geotechnical subsurface exploration program, geotechnical laboratory services, and geotechnical engineering. The scope of each service area was narrow and limited, as directed by our client, and based on our understanding and assumptions about the project. Service areas are briefly described later. Environmental-related consulting services were beyond Giles' scope for this project.

Geotechnical-related recommendations are provided in this report for design and construction of the foundations and at-grade floors for the proposed additions. Pavement recommendations are also provided. Furthermore, preliminary information is included regarding the infiltration of stormwater at the proposed stormwater management areas. Site preparation recommendations are included, but these recommendations are only preliminary because the means and methods of site preparation will depend on factors that were unknown when this report was prepared. Those factors include, but are not limited to, the weather before and during construction, the subsurface conditions that are exposed during construction, and the final details of the proposed project.

2.0 SITE DESCRIPTION

The subject site is south of W. Watertown Plank Road and east of W. Innovation Drive, in Wauwatosa, Wisconsin. The site area is shown on the *Test Boring Location Plan*, enclosed as Figure 1 in Appendix A. When the test borings (described later) were conducted, the Vel R. Phillips Juvenile Justice Center building occupied the site along with a parking lot, grassy areas, and sidewalks. Topographically, the site was flat and level. Ground elevations at the test borings varied between ±El. 98.6 and ±El. 100; these elevations are referenced to Giles' adopted benchmark, shown on the *Test Boring Location Plan*.

3.0 PROJECT DESCRIPTION

Proposed Additions

The proposed project will include an addition to the Vel R. Phillips Juvenile Justice Center building. The proposed location of the addition is shown on the *Test Boring Location Plan*. It is understood that the addition is planned to be a one- to two-story masonry structure with a small penthouse. Bearing walls will assumedly support the additions, possibly along with interior columns. Maximum foundation loads were not provided but are assumed to be 5,000 pounds per lineal foot (plf) from bearing walls and 150 kips per column. The addition is floor is planned to be a slab-on-

grade. The maximum floor load for the ground-bearing floor slab is assumed to be 100 pounds per square foot (psf). It is understood that the at-grade floor of the addition will match the at-grade floor of the existing building (El. 100 referenced to Giles' adopted benchmark). Therefore, only minor grade changes are expected in the addition area.

Proposed Parking Lot

The proposed improvements will include parking areas and drives around the proposed addition. Additionally, a road will pass south of the proposed addition, in an east-west orientation. It is assumed that the parking lots and drives will be paved with asphalt-concrete, but Portland cement concrete pavement will be in higher stress areas. Because Giles was not provided with traffic information, the pavement recommendations provided later are based on arbitrarily assumed traffic conditions. Also, because proposed pavement grades were not provided, this report assumes that pavement grades will be within 1 to 2 feet of existing surface grades.

Proposed Stormwater Management Areas

Stormwater management areas are planned to be constructed south of the proposed addition. Because the proposed bottom elevation of the stormwater area was not provided, this report assumes the bottom would be several feet below the current ground grades.

4.0 GEOTECHNICAL SUBSURFACE EXPLORATION PROGRAM

To explore subsurface conditions, nine test borings were conducted at the site using a mechanical drill-rig. Test Borings 1 through 6 were in the proposed addition area and were advanced to ±26 feet below-ground. Test Borings 7, 8, and 9 were in the proposed road and possible stormwater management areas and were advanced to ±21 feet below-ground. Test boring locations were positioned on-site based on the existing site features and by estimating right angles. Approximate locations of the test borings are shown on the *Test Boring Location Plan*.

Samples were collected from each test boring, at certain depths, using the Standard Penetration Test (SPT), conducted with the drill rig. A brief description of the SPT is given in Appendix B along with descriptions of other field procedures. Immediately after sampling, select portions of the SPT samples were placed in containers that were labeled at the site for identification. A Standard Penetration Resistance value (N-value) was determined from each SPT. N-values are reported on the *Test Boring Logs* (in Appendix A), which are records of the test borings.

The boreholes were backfilled upon completion; however, backfill materials will likely settle or heave, creating a hazard that can injure people and animals. Borehole areas should, therefore, be carefully and routinely monitored by the property owner or by others; settlement and heave of backfill materials should be repaired immediately. Giles will not monitor or repair boreholes.

Ground elevations at the test borings were determined by differential leveling referenced to Giles' adopted benchmark, shown on the *Test Boring Location Plan*. The test boring elevations are noted on the *Test Boring Logs* and are considered accurate within about one foot.

5.0 GEOTECHNICAL LABORATORY SERVICES

Samples that were retained from the test borings were transported to Giles' geotechnical laboratory, where the samples were classified using the descriptive terms and particle-size criteria shown on the *General Notes* in Appendix D and by using the Unified Soil Classification System (ASTM D 2488) as a general guide. Classifications are shown on the *Test Boring Logs* along with horizontal lines that show estimated depths of material change. Field-related information pertaining to the test borings is also shown on the *Test Boring Logs*. For simplicity and abbreviation, terms and symbols are used on the *Test Boring Logs;* the terms and symbols are defined on the *General Notes*.

Soil samples that were retained from Test Borings 7, 8 and 9 (conducted near the proposed stormwater area) were also visually classified using the USDA textural classification system in general accordance with the guidelines provided in the *Field Book for Describing and Sampling Soils* (USDA, Sept. 2012). USDA classifications of the retained samples are shown on the Wisconsin DSPS *Soil Evaluation – Storm* log, enclosed in Appendix A. Supplemental information regarding soil classifications, including the USDA and USCS soil classification systems, is included in the *Soil Classification Notes* enclosure within Appendix D.

Unconfined compression (without controlled strain), calibrated penetrometer resistance, and moisture content tests were performed on select cohesive soil samples to evaluate their general engineering properties. Results of the laboratory tests are on the *Test Boring Logs*. Because SPT samples were used, which are categorized as disturbed samples, results of the unconfined compression and calibrated penetrometer tests are considered to be approximate. Laboratory procedures are briefly described in Appendix C.

6.0 MATERIAL CONDITIONS

Because material sampling at the test borings was discontinuous, it was necessary to estimate conditions between sample intervals. Estimated conditions at the test borings are briefly discussed in this section and are described in more detail on the *Test Boring Logs*. The conclusions and recommendations in this report are based only on the estimated conditions.

6.1. Surface Materials

About 6 inches of asphalt-concrete pavement was at the surface of Test Borings 1, 2, 3, and 4. The asphalt pavement was underlain by about 4 to 6 inches of base course material. Topsoil was at the surface of the other test borings. The topsoil was about 6 to 24 inches thick and generally consisted of lean clay.

6.2. Fill Material

At Test Borings 1, 2, 4, 5, and 7, material classified as fill was beneath the surface materials and extended to depths between ±2 and ±6½ feet below-ground. The fill material was variable but generally consisted of lean clay, gravelly clay, or gravelly silty sand. The cohesive lean clay and gravelly clay fill exhibited a stiff to hard comparative consistency, based on laboratory testing.

6.3. Native Soil

Native soil was below the materials described above and extended to the termination depth at each test boring. The native soil generally consisted of lean clay, but sandy clay was encountered in the upper ±4 and ±6½ feet at Test Boring 6 and 2, respectively. Silty sand was also encountered to ±21 and ±11 feet at Test Boring 1 and 2, respectively. Sandy silt and silty sand lenses were present within the native lean clay. Native lean clay exhibited stiff and very stiff comparative consistencies based on laboratory testing. SPT N-values within native silty sand typically correlate to a firm relative density.

7.0 GROUNDWATER CONDITIONS

Based on the moisture conditions of the retained soil samples and encountered water at some of the test borings, it is estimated that the water table was about ±6½ to ±11 feet below-ground at the test boring locations when the test borings were conducted. However, the water table will fluctuate and might be shallower at certain times. It is important to note that the groundwater conditions discussed above are only an estimate; if a precise determination of the groundwater conditions is needed, groundwater observation wells are recommended to be installed and monitored at the site. Giles can install and monitor groundwater observation wells.

8.0 CONCLUSIONS AND RECOMMENDATIONS

8.1. Seismic Design Considerations

A soil Site Class C is recommended for seismic design. By definition, Site Class is based on the average properties of subsurface materials to 100 feet below-ground. Because 100-foot test borings were not requested or authorized for the project, it was necessary to estimate the Site Class based on the test borings, presumed area geology, and the International Building Code.

8.2. Building Addition Foundation Recommendations

A spread-footing foundation is recommended for the addition. However, existing fill is unsuitable for direct or indirect support of foundations. Each footing must bear on suitable native soil or on new engineered fill or lean-concrete backfill (both discussed below) placed on suitable native soil. Based on Test Borings 1 through 6 (conducted in the proposed addition areas), foundations for

the proposed additions are recommended to be designed using a 4,000 psf maximum, net, allowable soil bearing capacity. For geotechnical considerations and regardless of the calculated foundation-bearing stress, strip footings are recommended to be at least 18 inches wide, and isolated footings are recommended to be at least 24 inches wide and long. It is recommended and assumed that a structural engineer will provide specific foundation details, including footing dimensions, reinforcing, etc.

A minimum 48-inch foundation-embedment depth is required by the building code. It is, therefore, recommended that footings for perimeter walls and other exterior elements of the additions bear at least 48 inches below the adjacent finished ground-grade. Interior footings within the additions can bear directly below the floor slab, assuming that the addition will be heated and support soil will not freeze. However, it is recommended that foundations for the additions bear at the same elevation as the existing foundations, assuming that the required embedment depth will be met for frost protection. Therefore, extension of the addition foundations might be necessary to match the bearing elevation of existing foundations.

A frictional coefficient of 0.30 is recommended to determine the lateral resistance of the foundations. The recommended frictional coefficient is only for concrete cast directly on suitable native soil or on new engineered fill or lean-concrete backfill used to replace unsuitable materials. Lateral resistance due to friction should be determined based on dead load only. Also, the ultimate lateral resistance determined from the frictional coefficient is recommended to be factored to determine an allowable value. Passive resistance is recommended to be neglected to at least the recommended 48-inch foundation-embedment depth due to seasonal changes and due to the amount of lateral movement necessary to develop full passive pressure.

The following table provides estimated depths of native soil that is suitable for direct foundation support (based on the recommended 4,000 psf bearing capacity) at the building-area test borings. It is important to note that suitable native soil might be at deeper depths away from the test borings, and along the existing building, where existing fill will likely be encountered; therefore, testing and approval of foundation-support soil by a geotechnical engineer during construction is critical. Without testing and approval of foundation-support soil by a geotechnical engineer, the additions could be improperly supported.

TABLE 1
ESTIMATED DEPTH/ELEVATION OF SUITABLE BEARING NATIVE SOIL

Test Boring Number	Estimated Depth of Suitable Native Soil	Estimated Elevation of Suitable Native Soil
1	±2 feet	±El. 97.6
2	±2 feet	±El. 97.6
3	±1 foot	±El. 99.0
4	±4 feet	±El. 95.3
5	±6½ feet	±El. 92.9
6	±1 foot	±El. 98.9

- For direct foundation support, or for placement of engineered fill or lean-concrete backfill; based on a 4,000 psf maximum, net, allowable soil bearing capacity.
- Referenced to the site grades during the geotechnical subsurface exploration program.
- Elevations are referenced to the elevations shown on the *Test Boring Logs*.

Foundation excavations are recommended to be dug with a smooth-edge backhoe bucket to develop a relatively undisturbed bearing grade. A toothed bucket will likely disturb foundation-bearing soil more than a smooth-edge bucket, thereby making soil at the excavation base more susceptible to saturation and instability, especially during adverse weather. It is critical that contractors protect foundation-support soil and foundation construction materials (concrete and reinforcing). Furthermore, engineered fill is recommended to be placed and compacted in benched excavations along foundation walls immediately after the foundation walls can properly support lateral pressures from backfill, compaction, and compaction equipment. Earth-formed footing construction techniques are expected to be feasible within cohesive site soil, but foundation forms might be needed within granular site soil.

Foundation Support Soil Requirements

Existing fill is unsuitable for direct or indirect support of foundations. Each footing must bear on suitable native soil or on new engineered fill or lean-concrete backfill (both discussed below) that is placed on suitable native soil. Based on the recommended 4,000 psf maximum, net, allowable soil bearing capacity, the in-situ unconfined compressive strength of cohesive native soil, such as lean clay, within foundation influence zones is recommended to be at least 2 tons per square foot (tsf). Native granular soil, such as gravelly silty sand, within foundation influence zones is recommended to have a corrected N-value (determined from SPTs and correlated from other in-situ tests) of at least 14, based on the recommended bearing capacity. It is further recommended that the strength characteristics of soil within all foundation influence zones (determined by a geotechnical engineer during construction) meet or exceed the recommended values, unless Giles approves other values.

Evaluation of foundation-support soil by a geotechnical engineer during foundation excavation and immediately before foundation construction is critical, especially considering the existing development and existing fill. The purpose of the evaluation is (1) to confirm that the foundations

will be properly supported by suitable native soil, (2) to determine where over-excavation is needed, and (3) to confirm that the support soils are similar to those described on the *Test Boring Logs*. If a firm other than Giles performs the recommended support-soil evaluation, Giles must be notified if the composition or strength characteristics of foundation-support soils differ from those shown on the *Test Boring Logs*; revision of this report might be necessary. Without evaluation and approval of foundation-support soil by a geotechnical engineer, the proposed additions could be improperly supported, which could lead to excessive settlement and other structural problems.

Unsuitable materials beneath foundation areas possibly could be replaced with engineered fill consisting of dense-graded crushed stone that meets the gradation requirements of *dense-graded base* (1¼-inch) in Section 305 of the Wisconsin Department of Transportation Standard Specifications (2019). Granular material with other gradation characteristics could possibly be used but should be approved by a geotechnical engineer before the material is placed. If engineered fill is used as backfill, lateral over-excavation of unsuitable materials will also be required, in addition to the required vertical over-excavation. The overall width of lateral over-excavation will depend on the vertical over-excavation depth. For estimating purposes, the minimum lateral over-excavation could be determined by extending an imaginary line outward and downward at a ratio of 1(horizontal):2(vertical) from the bottom edges of a footing pad, but the actual lateral extents of over-excavation are recommended to be approved by a geotechnical engineer during construction.

Lean Portland cement concrete (minimum 28-day compressive strength of 1,000 psi) could also be used to replace unsuitable materials beneath foundation areas and is Giles' preferred backfill material. Where lean concrete is used as backfill, footing construction must not begin until the lean concrete has gained sufficient strength. Also, over-excavations that are filled with lean concrete are recommended to be at least as wide (on all sides) as the footing pad that will be supported by the concrete, and excavation sidewalls are recommended to be plumb and parallel. To help control caving, lean-concrete backfill is recommended to be placed immediately after excavation. This trench-and-pour method requires close communication and scheduling between the general contractor, foundation contractor, concrete supply company, and geotechnical engineer. With a trench-and-pour method, a geotechnical engineer must observe excavations as they are made. Full-time observation by a geotechnical engineer is recommended.

Existing Construction Considerations

Precautions must be taken to protect the existing building during construction and to ensure that excavations do not undermine or otherwise compromise the existing building or other existing site improvements. If a void develops below existing footings or floor slabs, a geotechnical engineer should immediately observe the conditions and provide repair recommendations. In general, voids should be immediately filled with a concrete dry-pack, or an expansive sand-and-cement slurry (non-shrink) should be injected into the void, under appropriate pressure, to redevelop contact between the foundation and supporting soils.

Near the existing building, it is recommended that foundations for the additions bear at the same elevation as the adjacent (existing) foundations, assuming that the required 48-inch embedment depth will be met, where required. If the new and existing footings will bear at different elevations, a structural engineer should evaluate the stresses to be imposed on the lower foundation and confirm that the structural integrity of the existing building and additions will be maintained. Control joints should separate the existing building and the addition since some differential movement is expected at these junctures. Excavations must not be performed within the zone of influence (determined by a geotechnical engineer) of an existing footing; otherwise, existing footings could be undermined, possibly causing significant (and catastrophic) damage.

Where new foundations are perpendicular to an existing foundation, it might be necessary to cantilever new foundations a certain distance away from the existing building to help reduce potential settlement of the existing building due to overlapping stress from the new construction. When the existing and proposed foundation systems and depths can be confirmed, Giles should be contacted to evaluate whether our recommendations need to be updated. Care must be taken to protect the existing building during construction of the additions. The existing building should be underpinned and braced, where needed. Extra care should be exercised not to undermine existing footings during removal of unsuitable materials or during construction of the new footings.

It is assumed that the proposed additions will be self-supporting structures and that no structural load will be imposed on the existing building due to the additions. If load is added to the existing building, it will likely undergo some settlement. The amount and location of settlement will partly depend on the magnitude and location of the load increase. Differential settlement should be expected between the existing building and the additions, even if additional load will not be imposed on the existing building.

Estimated Foundation Settlement

The post-construction total and differential settlements of a spread-footing foundation designed and constructed based on this report are estimated to be less than about 1 inch and ½ inch, respectively. The post-construction angular distortion is estimated to be less than about 0.002 inch per inch across 20 feet. Estimated settlements assume that the recommendations provided in this report will be followed and that foundation-support soil will be evaluated and approved by a geotechnical engineer during construction.

8.3. At-Grade Floor Slab Recommendations

With proper subgrade preparation, existing soil (including existing fill) is expected to be suitable to support at-grade floor slabs for the proposed addition; new engineered fill that is placed on properly prepared existing soil is also expected to be suitable. However, subgrade improvement

might be necessary to develop proper slab support, considering the existing fill. Consequently, all at-grade floor areas are recommended to be thoroughly evaluated and approved by a geotechnical engineer immediately before fill placement and before floor construction. Without a thorough evaluation of floor slab support materials, at-grade floor slabs might be improperly supported, which could lead to excessive settlement and other structural problems.

From a geotechnical perspective and based on a maximum 100 psf floor load, at-grade floor slabs for the proposed addition is recommended to be at least 4 inches thick; this thickness assumes that the 28-day compressive strength of concrete will be at least 3,500 pounds per square inch (psi). Assuming proper site preparation, the floor slabs may be designed using a *Modulus of Subgrade Reaction* (K_{v1}) value of 125 pounds per square inch per inch (psi/in). It is recommended and assumed that a structural engineer will specify the actual floor slab thickness, reinforcing, joint details, and other parameters.

A minimum 4-inch-thick base course is recommended to be below the floor slabs to serve as a capillary break and for support considerations. It is recommended that the base course consist of free-draining aggregate that has been tested and approved by a geotechnical engineer. Depending on aggregate gradation and the subgrade conditions, geotextile might need to be below the base material to serve as a separator. The need for geotextile should be determined during construction with the assistance of a geotechnical engineer.

A minimum 10-mil vapor retarder is recommended to be directly above or below the base course throughout all at-grade floor areas. The position (above or below the base course) of the vapor retarder should be specified by the project structural engineer or architect. Abutting vapor retarder sheets are recommended to be overlapped at least 6 inches, and the overlaps are recommended to be fully taped. Also, it is recommended that vapor barriers extend to all foundation walls. Vapor retarders are recommended to be in accordance with ASTM E 1745, entitled *Standard Specification for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs*, and other relevant documents. If the base course has sharp, angular aggregate, protecting the retarder with geotextile (or by other means) is recommended.

Due to the frost-susceptible site soil, areas of the at-grade floor slabs (such as near exterior doors and entrance-exit vestibules) might be susceptible to freeze-thaw related movement. Installation of insulation (or other protective measures against freeze-thaw movement) should be considered for these areas. Pavement and ground grades are recommended to be sloped away from the existing building, proposed additions, and sidewalks to reduce water infiltration and potential freeze-thaw problems.

Estimated Floor Slab Settlement

The post-construction total and differential settlements of an isolated floor slab constructed in accordance with this report are estimated to be less than about ⅝ inch and ⅓ inch, respectively,

over about 20 feet. Estimated settlements assume that the addition areas will be prepared per this report, and that floor slab support materials will be thoroughly tested and approved by a geotechnical engineer during construction.

8.4. Pavement Recommendations

Giles was not given specific traffic-related information regarding the proposed project; therefore, the recommendations provided herein are based on an arbitrarily assumed traffic condition consisting of ten 18-kip Equivalent Single Axle Loads (ESALs) per day. The recommended pavement sections given below are intended for pavement areas subject to passenger vehicles, with infrequent traffic from heavier vehicles due to occasional deliveries and due to removal of refuse and recyclables. The recommended pavement sections assume no increase in traffic volume and no changes in vehicle type or traffic pattern. Also, it is assumed that the ESALs noted above will be in one direction for each lane.

It is important that the project owner, developer, civil engineer, and other design professionals involved with the project confirm that the ESALs noted above are appropriate for the expected traffic conditions, vehicle types, and axle loadings. If requested, Giles can provide supplemental pavement recommendations based upon other traffic conditions, vehicle types, and axle loads. The recommended pavement sections could underperform or fail prematurely if the design ESALs are exceeded.

Based on the test borings, it is expected that pavement support materials will consist of lean clay. Therefore, the recommended pavement sections were developed based on a lean clay subgrade, with an assumed field CBR value of 3 and a *Modulus of Subgrade Reaction* (K_{V1}) value of 100 psi/in. Engineered fill that is placed in proposed pavement areas is recommended to have a field CBR value and a *Modulus of Subgrade Reaction* (K_{V1}) value at least equivalent to the design values. Fill is recommended to be placed and compacted per this report.

Because of the moisture conditions and strength characteristics of the lean clay and sandy clay, the subgrade will likely need to be improved, especially if construction is during or after adverse weather. The need for subgrade improvement should be determined during construction with the assistance of a geotechnical engineer.

Asphalt-Concrete Pavement

The following table shows the recommended thicknesses for hot-mix asphalt (HMA) pavement with an aggregate base-course. State specifications are also included in the table. The recommended pavement section is based on the traffic condition described above.

TABLE 2 RECOMMENDED HMA PAVEMENT SECTION		
Materials	**Pavement Thickness**	**Wisconsin DOT Standard Specifications**
Hot-Mix Asphalt Surface Course	1.5 inches	Section 460
Hot Mix Asphalt Binder Course	2.5 inches	Section 460
Dense-Graded Aggregate Base Course	8.0 inches	Section 305, 1¼-inch Crushed Stone

Portland Cement Concrete Pavement

Portland cement concrete pavement is recommended in high-stress areas, such as entrance and exit aprons and in areas where trucks will turn or will be parked. Concrete pavement is recommended to be at least 6 inches thick and is recommended to be underlain by a minimum 4-inch-thick aggregate base-course. It is recommended that concrete pavement have load-transfer reinforcement, where appropriate. Control-joint spacing should be determined in accordance with the current ACI code. Expansion joints should be provided where pavement abuts fixed objects, such as the buildings and light poles. The 28-day compressive strength of concrete is recommended to be at least 4,000 psi and the concrete should be properly air-entrained. It is recommended and assumed that a civil engineer will provide specific recommendations for concrete pavement, including reinforcing details and control-joint spacing. Materials and construction procedures for concrete pavement and the aggregate base are recommended to be in accordance with Wisconsin DOT specifications.

General Pavement Considerations

The pavement recommendations assume that the pavement subgrade will be prepared according to this report, the base course will be properly drained, and a geotechnical engineer will observe and test pavement construction. Pavement was designed based on AASHTO design parameters for a twenty-year design period, but the actual service life may be less. More frequent pavement maintenance should be expected in areas of perched groundwater. Local codes may require specific testing to determine soil support characteristics and minimum pavement section thickness might be required.

8.5. Preliminary Stormwater Infiltration Screening

Stormwater management devices are planned to be constructed in the areas of Test Borings 7, 8, and 9; approximate locations of these test borings are shown on the *Test Boring Location Plan*. Because of clay loam soils and moderately shallow groundwater that was encountered at Test Borings 7, 8, and 9, Giles considers this area to be exempt from stormwater infiltration

requirements per section NR 151.124(4)(c) of the Wisconsin Administrative Code and WDNR 1002 guidelines.

8.6. Generalized Site Preparation Recommendations

This section deals with site preparation, including preparation of floor slab, pavement, and engineered fill areas. The means and methods of site preparation will greatly depend on the weather conditions before and during construction, the subsurface conditions that are exposed during earthwork operations, and the finalized details of the proposed development. Therefore, only generalized site preparation recommendations are given.

In addition to being generalized, the following site preparation recommendations are abbreviated; the *Guide Specifications* in Appendix D gives further recommendations. The *Guide Specifications* should be read along with this section. Also, the *Guide Specifications* are recommended to be used as an aid to develop the project specifications.

Removal and Stripping

Pavement, surface vegetation, trees and bushes (including root-balls), topsoil, and other unsuitable materials are recommended to be removed from the proposed addition areas, pavement area, and other structural areas. Stripping and removal should extend at least several feet beyond the proposed development areas, where feasible. Existing pavement should remain in place as long and possible to protect the underlying soil.

Proof-Rolling and Fill Placement

After the recommended removal and stripping, and once the improvement areas are cut (lowered) as needed, the subgrade within each improvement area is recommended to be proof-rolled with a fully-loaded tandem-axle dump truck (or other suitable construction equipment) to locate unstable areas based on subgrade deflection caused by the wheel loads of the proof-roll equipment. However, proof-roll equipment must be kept a sufficient distance from the existing building and other existing construction, as existing construction could be damaged during proof-rolling. Also, for safety, proof-roll equipment must be kept a sufficient distance from excavations. It is recommended that a geotechnical engineer observe proof-roll operations and evaluate subgrade stability based on those observations. Areas that cannot be proof-rolled (such as near the existing building) are recommended to be evaluated and approved by a geotechnical engineer using appropriate means and methods.

Due to the existing fill and existing developments, unsuitable materials might be encountered during proof-rolling and testing. Unsuitable materials are recommended to be removed and replaced with engineered fill or improved. Recommendations for subgrade improvement should, however, be made by a geotechnical engineer based on the site conditions during construction.

Areas requiring subgrade improvement should be defined during construction with the assistance of a geotechnical engineer. Specific improvement methods should be determined during construction on an area-by-area basis.

The improvement areas are recommended to be raised, where necessary, to the planned finished grades with engineered fill immediately after each subgrade is confirmed to be stable and suitable to support the proposed site improvements. Engineered fill is recommended to be placed in thin layers (lifts) that are uniform in elevation. Each layer of engineered fill is recommended to be compacted to at least 95 percent of the fill material's maximum dry density determined from the Standard Proctor compaction test (ASTM D 698). As an exception, the in-place dry density of engineered fill within one foot of a pavement subgrade is recommended to be compacted to at least 100 percent of the fill's maximum dry density. The water content of fill material is recommended to be uniform and within a narrow range of the optimum moisture content, also determined from the Standard Proctor compaction test. Item Nos. 4 and 5 of the *Guide Specifications* give more information pertaining to selection and compaction of engineered fill.

Care must be taken not to damage the existing building or other existing construction during compaction of engineered fill. In some areas (such as along foundation walls of the existing building), it will likely be necessary to use walk-behind vibratory compaction equipment, possibly along with imported aggregate fill material. Also, vibratory compaction equipment should not be used near groundwater (including perched groundwater), since vibratory compaction near groundwater could cause soil to become unstable.

Engineered fill that does not meet the density and water content requirements is recommended to be replaced or scarified to a sufficient depth (likely 6 to 12 inches, or more), moisture-conditioned, and compacted to the required density. A subsequent lift of fill should only be placed after a geotechnical engineer confirms that the previous lift was properly placed and compacted. Subgrade soil will likely need to be recompacted immediately before construction, since equipment traffic and adverse weather may reduce soil stability.

Use of Site Soil as Engineered Fill

Site soil that does not contain adverse organic content or other deleterious materials, as noted in the *Guide Specifications*, could be used as engineered fill. However, site soil will likely need to be moisture conditioned (uniformly moistened or dried) prior to being used as engineered fill. If construction is during adverse weather (discussed in the following section), drying site soil will likely not be feasible. In this case, fill material will likely need to be imported to the site. Additional recommendations regarding fill selection, placement, and compaction are given in the *Guide Specifications*.

8.7. Generalized Construction Considerations

Adverse Weather

Site soil is moisture sensitive and will become unstable when exposed to adverse weather, such as rain, snow, and freezing temperatures. Therefore, it might be necessary to remove or stabilize the upper 6 to 12 inches (or more) of soil due to adverse weather, which commonly occurs during late fall, winter, and early spring. At least some over-excavation or stabilization of unstable soil should be expected if construction is during or after adverse weather. Because site preparation is weather dependent, bids for site preparation and other earthwork activities should consider the time of year that construction will be conducted.

To protect soil from adverse weather, the site is recommended to be smoothly graded and contoured during construction to divert surface water away from construction areas. Contoured subgrades are recommended to be rolled with a smooth-drum compactor, before precipitation, to "seal" the surface. Furthermore, construction traffic should be restricted to certain aggregate-covered areas to control traffic-related soil disturbance. Foundation, floor slab, and pavement construction should begin immediately after suitable support is confirmed.

Dewatering

Water that accumulates in construction areas is recommended to be removed along unsuitable soil as soon as possible. Filtered sump pumps, drawing water from sump pits excavated in the bottom of construction trenches, are expected to be adequate to remove water that collects in shallow excavations. Multiple sump pumps might be necessary. Excavated sump pits should be fully lined with geotextile and filled with free-draining crushed stone, such as crushed stone that meets the gradation requirements of ASTM No. 57 aggregate.

Excavation Stability

Excavations are recommended to be made in accordance with current OSHA excavation and trench safety standards and other applicable requirements. Where required, sides of excavations must be sloped, benched, or braced to develop and maintain a safe work environment. Temporary shoring must be designed according to applicable regulatory requirements. Contractors are responsible for excavation safety. Excavations will be susceptible to caving.

Existing Utilities

All existing utilities are recommended to be identified and located, and any planned to be maintained should be relocated outside the addition areas. Utilities that are not reused should be capped-off and removed in accordance with local codes and ordinances. Excavations for the removal of utilities are recommended to be backfilled with engineered fill placed under

engineering-controlled conditions. Grading operations must be done carefully so that existing utilities are not damaged or disturbed. Utility elevations, locations, and types should be checked relative to the planned construction to identify any concerns.

8.8. Recommended Construction Materials Testing Services

This report was prepared assuming that a geotechnical engineer will perform Construction Materials Testing ("CMT") services during construction of the proposed development. Supplemental geotechnical recommendations may be needed based on the results of CMT services and specific details of the project not known at this time.

9.0 BASIS OF REPORT

This report is strictly based on the project description given in Section 3.0. Giles must be notified if the project description or our assumptions are not accurate so that this report can be amended, if needed. This report assumes that the facility will be designed and constructed according to the codes that govern construction at the site.

The conclusions and recommendations in this report are based on estimated subsurface conditions as shown on the *Test Boring Logs*. Giles must be notified if the subsurface conditions that are encountered during construction of the proposed development differ from those shown on the *Test Boring Logs*; revision of this report might be necessary. General comments and limitations of this report are given in the appendix.

The conclusions and recommendations in this report have been promulgated in accordance with generally accepted professional engineering practices in the field of geotechnical engineering. No other warranty is either expressed or implied.

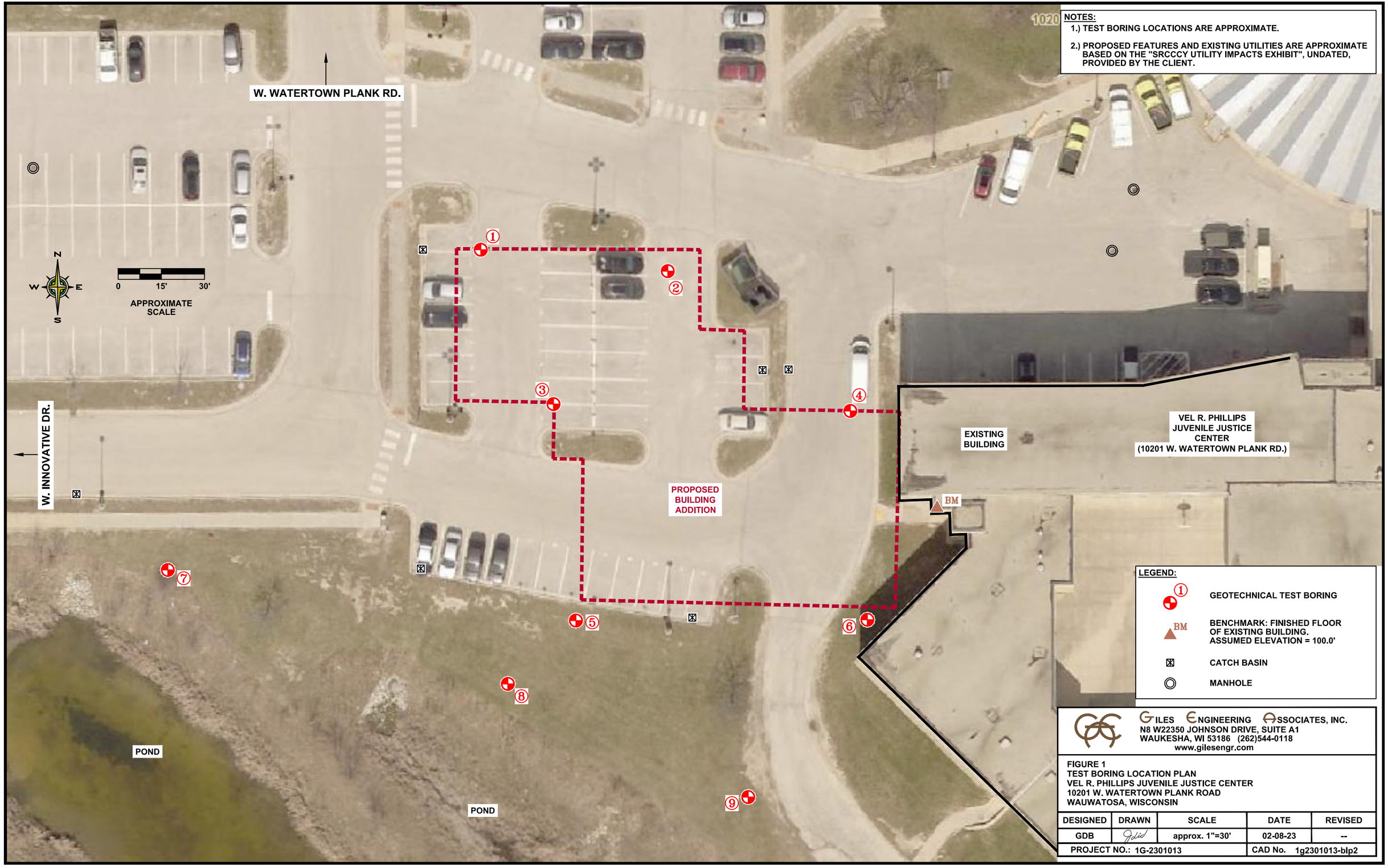
 1G-2301013report/23Geo01/ajg

APPENDIX A

FIGURES AND TEST BORING LOGS

The Test Boring Location Plan contained herein was prepared based upon information supplied by *Giles'* client, or others, along with *Giles'* field measurements and observations. The diagram is presented for conceptual purposes only and is intended to assist the reader in report interpretation.

The Test Boring Logs and related information enclosed herein depict the subsurface (soil and water) conditions encountered at the specific boring locations on the date that the exploration was performed. Subsurface conditions may differ between boring locations and within areas of the site that were not explored with test borings. The subsurface conditions may also change at the boring locations over the passage of time.



NOTES:

1.) TEST BORING LOCATIONS ARE APPROXIMATE.

2.) PROPOSED FEATURES AND EXISTING UTILITIES ARE APPROXIMATE BASED ON THE "SRCCCY UTILITY IMPACTS EXHIBIT", UNDATED, PROVIDED BY THE CLIENT.

LEGEND:

- ① GEOTECHNICAL TEST BORING
- BM BENCHMARK: FINISHED FLOOR OF EXISTING BUILDING. ASSUMED ELEVATION = 100.0'
- CATCH BASIN
- MANHOLE

GILES ENGINEERING ASSOCIATES, INC.
N8 W22350 JOHNSON DRIVE, SUITE A1
WAUKESHA, WI 53186 (262)544-0118
www.gilesengr.com

FIGURE 1
TEST BORING LOCATION PLAN
VEL R. PHILLIPS JUVENILE JUSTICE CENTER
10201 W. WATERTOWN PLANK ROAD
WAUWATOSA, WISCONSIN

DESIGNED	DRAWN	SCALE	DATE	REVISED
GDB	JDH	approx. 1"=30'	02-08-23	--
PROJECT NO.: 1G-2301013			CAD No.	1g2301013-blp2

BORING NO. & LOCATION: 1	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 99.6 feet	PROPOSED BUILDING ADDITION	
COMPLETION DATE: 02/06/23	10201 W. WATERTOWN PLANK ROAD WAUWATOSA, WISCONSIN	
FIELD REP: JAMES BLAIR	PROJECT NO: 1G-2301013	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
±6" Asphalt-Concrete										
±4" Base Course										
Fill: Dark Brown Gravelly Silty Sand-Moist			1-AU							
Brown Silty fine to medium Sand, trace Gravel-Moist			2-SS	24						
	5	95	3-SS	18						
Gray Silty fine to medium Sand, trace Gravel-Wet			4-SS	18						
	10	90	5-SS	17						
	15	85	6-SS	31						
	20	80	7-SS	16						
Gray Sandy Clay, trace Gravel-Wet	25	75	8-SS	8						
Boring Terminated at about 26 feet (EL. 73.6')										

Water Observation Data	Remarks:
Water Encountered During Drilling: 6.5 ft.	
Water Level At End of Drilling:	
Cave Depth At End of Drilling:	
Water Level After Drilling:	
Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

GILES LOG REPORT 1G2301013.GPJ GILES.GDT 2/9/23

BORING NO. & LOCATION: 2	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 99.6 feet	PROPOSED BUILDING ADDITION	
COMPLETION DATE: 02/06/23	10201 W. WATERTOWN PLANK ROAD WAUWATOSA, WISCONSIN	
FIELD REP: JAMES BLAIR	PROJECT NO: 1G-2301013	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
±6" Asphalt-Concrete										
±6" Base Course			1-AU							
Fill: Dark Brown Gravelly Silty Sand-Moist			2-SS	17		2.8		27		
Brown lean Clay, little Sand, trace Gravel-Moist										
Brown Sandy Clay, trace Gravel-Moist	5	95	3-SS	15		4.3		15		
Brown Silty fine to medium Sand, trace Gravel-Moist			4-SS	22						
Gray Silty fine to medium Sand, trace Gravel-Wet	10	90	5-SS	18						
Gray lean Clay, little Sand, trace Gravel-Wet	15	85	6-SS	19	5.2	4.5+		18		
	20	80	7-SS	34	3.7	4.3		26		
	25	75	8-SS	16	3.9	3.5		19		
Boring Terminated at about 26 feet (EL. 73.6')										

Water Observation Data	Remarks:
Water Encountered During Drilling: 9 ft.	
Water Level At End of Drilling:	
Cave Depth At End of Drilling:	
Water Level After Drilling:	
Cave Depth After Drilling:	

GILES LOG REPORT 1G2301013.GPJ GILES.GDT 2/9/23

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: 3	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 100 feet	PROPOSED BUILDING ADDITION	
COMPLETION DATE: 02/06/23	10201 W. WATERTOWN PLANK ROAD WAUWATOSA, WISCONSIN	
FIELD REP: JAMES BLAIR	PROJECT NO: 1G-2301013	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
±6" Asphalt-Concrete										
±6" Base Course			1-AU							
Brown lean Clay, little Sand, trace Gravel-Moist			2-SS	16		4.5+		11		
	5	95	3-SS	17		3.8		14		
			4-SS	18		3.3		14		
Gray lean Clay, little Sand-Moist to Wet at ±11 feet	10	90	5-SS	23				13		
	15	85	6-SS	21	8.2	4.5+		17		
	20	80	7-SS	12	4.5	4.5+		25		
Gray fine to medium Sand, trace Silt-Wet	25	75	8-SS	12						
Boring Terminated at about 26 feet (EL. 74')										

Water Observation Data	Remarks:
Water Encountered During Drilling: 11 ft.	
Water Level At End of Drilling:	
Cave Depth At End of Drilling:	
Water Level After Drilling:	
Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

GILES LOG REPORT 1G2301013.GPJ GILES.GDT 2/9/23

BORING NO. & LOCATION: 4	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 99.3 feet	PROPOSED BUILDING ADDITION	
COMPLETION DATE: 02/06/23	10201 W. WATERTOWN PLANK ROAD WAUWATOSA, WISCONSIN	
FIELD REP: JAMES BLAIR	PROJECT NO: 1G-2301013	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
±6" Asphalt-Concrete										
±6" Base Course			1-AU							
Fill: Dark Brown lean Clay, little Sand, trace Gravel-Moist			2-SS	12						
Brown lean Clay, little Sand, trace Gravel-Moist	5	95	3-SS	16		3.3		17		
			4-SS	25		4.5		13		
Gray lean Clay, little Sand, trace Gravel-Moist to Wet at ±11 feet	10 ▽	90	5-SS	21		4.5+		11		
	15	85	6-SS	54	2.6	2.5		30		
(Includes Silty fine to medium Sand seams below ±21 feet)	20	80	7-SS	11						
	25	75	8-SS	16				17		
Boring Terminated at about 26 feet (EL. 73.3')										

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: 11 ft.	
▼	Water Level At End of Drilling:	
	Cave Depth At End of Drilling:	
▼	Water Level After Drilling:	
	Cave Depth After Drilling:	

GILES LOG REPORT 1G2301013.GPJ GILES.GDT 2/9/23

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: 5	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 99.4 feet	PROPOSED BUILDING ADDITION	
COMPLETION DATE: 02/07/23	10201 W. WATERTOWN PLANK ROAD WAUWATOSA, WISCONSIN	
FIELD REP: JAMES BLAIR	PROJECT NO: 1G-2301013	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
±6" Topsoil Fill: Dark Brown lean Clay, little Sand and Organic Matter-Moist			1-AU							
Fill: Dark Brown lean Clay, little Sand, trace Gravel-Moist			2-SS	7		3.0		13		
	5	95	3-SS	11		1.0		13		
Brown lean Clay, little Sand, trace Gravel-Moist			4-SS	30		4.5+		12		
Gray lean Clay, little Sand, trace Gravel-Moist to Wet at ±11 feet	10 ▽	90	5-SS	26		4.5+		15		
	15	85	6-SS	17	2.6	3.0		28		
(Includes Silty fine to medium Sand lenses below ±21 feet)	20	80	7-SS	11		3.0		14		
	25	75	8-SS	35						
Boring Terminated at about 26 feet (EL. 73.4')										

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: 11 ft.	
▼	Water Level At End of Drilling:	
	Cave Depth At End of Drilling:	
▼	Water Level After Drilling:	
	Cave Depth After Drilling:	

GILES LOG REPORT 1G2301013.GPJ GILES.GDT 2/9/23

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: 6	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 99.9 feet	PROPOSED BUILDING ADDITION	
COMPLETION DATE: 02/07/23	10201 W. WATERTOWN PLANK ROAD WAUWATOSA, WISCONSIN	
FIELD REP: JAMES BLAIR	PROJECT NO: 1G-2301013	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
±8" Topsoil: Dark Gray lean Clay, little Sand and Organic Matter-Moist			1-AU							
Brown Sandy Clay, trace Silt-Moist			2-SS	18		4.0		15		
Brown lean Clay, little Sand, trace Gravel-Moist	5	95	3-SS	14		2.3		13		
			4-SS	28		4.5+		13		
Gray lean Clay, little Sand, trace Gravel-Moist to Wet at ±11 feet ▽	10	90	5-SS	24	5.0	4.5+		12		
	15	85	6-SS	10				14		
(Includes Sandy Silt lenses below ±21 feet)	20	80	7-SS	9	3.3	3.0		30		
	25	75	8-SS	13						
Boring Terminated at about 26 feet (EL. 73.9')										

Water Observation Data	Remarks:
▽ Water Encountered During Drilling: 11 ft.	
Water Level At End of Drilling:	
Cave Depth At End of Drilling:	
Water Level After Drilling:	
Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

GILES LOG REPORT 1G2301013.GPJ GILES.GDT 2/9/23

BORING NO. & LOCATION: 7	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 99.7 feet	PROPOSED BUILDING ADDITION	
COMPLETION DATE: 02/07/23	10201 W. WATERTOWN PLANK ROAD WAUWATOSA, WISCONSIN	
FIELD REP: JAMES BLAIR	PROJECT NO: 1G-2301013	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
±24" Topsoil Fill: Dark Brown lean Clay, little Sand and Organic Matter-Moist			1-AU							
Fill: Brown Gravelly Clay, little Sand-Moist			2-SS	31		4.5+		11		
Brown lean Clay, little Sand, trace Gravel-Moist	5	95	3-SS	20		4.5+		13		
			4-SS	18		4.5		15		
Gray lean Clay, little Sand, trace Gravel-Moist to Wet at ±11 feet ▽	10	90	5-SS	52	4.3	4.5+		21		
(Includes Sandy Silt lenses below ±16 feet)	15	85	6-SS	13						
	20	80	7-SS	17						
Boring Terminated at about 21 feet (EL. 78.7')										

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: 11 ft.	
▼	Water Level At End of Drilling:	
	Cave Depth At End of Drilling:	
▼	Water Level After Drilling:	
	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

GILES LOG REPORT 1G2301013.GPJ GILES.GDT 2/9/23

BORING NO. & LOCATION: 8	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 99.4 feet	PROPOSED BUILDING ADDITION	
COMPLETION DATE: 02/07/23	10201 W. WATERTOWN PLANK ROAD WAUWATOSA, WISCONSIN	
FIELD REP: JAMES BLAIR	PROJECT NO: 1G-2301013	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
±14" Topsoil: Dark Gray lean Clay, little Sand and Organic Matter-Moist			1-AU							
Brown lean Clay, little Sand, trace Gravel-Moist			2-SS	17		4.5+		10		
	5	95	3-SS	12	1.8	2.0		17		
Gray lean Clay, little Sand, trace Gravel-Moist to Wet at ±11 feet			4-SS	26		4.0		16		
	10	90	5-SS	29				15		
	15	85	6-SS	10		2.0		27		
(Includes Sandy Silt lenses below ±16 feet)										
	20	80	7-SS	72		3.5		14		
Boring Terminated at about 21 feet (EL. 78.4')										

Water Observation Data	Remarks:
Water Encountered During Drilling: 11 ft.	
Water Level At End of Drilling:	
Cave Depth At End of Drilling:	
Water Level After Drilling:	
Cave Depth After Drilling:	

GILES LOG REPORT 1G2301013.GPJ GILES.GDT 2/9/23

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: 9	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 98.6 feet	PROPOSED BUILDING ADDITION	
COMPLETION DATE: 02/07/23	10201 W. WATERTOWN PLANK ROAD WAUWATOSA, WISCONSIN	
FIELD REP: JAMES BLAIR	PROJECT NO: 1G-2301013	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
±14" Topsoil: Dark Gray lean Clay, little Sand and Organic Matter-Moist			1-AU							
Brown lean Clay, little Sand, trace Gravel-Moist			2-SS	10		2.5		14		
(Includes fine to medium Sand lenses below ±4 feet)	5	95	3-SS	24				15		
Gray lean Clay, little Sand, trace Gravel-Moist to Wet at ±9 feet			4-SS	25	9.1	4.5+		18		
	▽ 10	90	5-SS	63						
	15	85	6-SS	16	3.3	3.0		29		
	20	80	7-SS	17				16		
Boring Terminated at about 21 feet (EL. 77.6')										

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: 9 ft.	
▼	Water Level At End of Drilling:	
	Cave Depth At End of Drilling:	
▼	Water Level After Drilling:	
	Cave Depth After Drilling:	

GILES LOG REPORT 1G2301013.GPJ GILES.GDT 2/9/23

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

1002-CPS-23
Division of Industry Services
P. O. Box 2658
Madison, Wisconsin 53701
Scott Walker, Governor
Laura Gutierrez, Secretary

Attachment 2:

SOIL AND SITE EVALUATION – STORM

In accordance with SPS 382.365, 385, Wis. Adm. Code, and WDNR Standard 1002

Attach a complete site plan on paper not less than 8 ½ x 11 inches in size. Plan must include, but not limited to: vertical and horizontal reference point (BM), direction and percent of slope, scale or dimensions, north arrow, and BM referenced to nearest road

Please print all information

Personal information you provide may be used for secondary purposes [Privacy Law, s. 15.04(1)(m)]

County: Milwaukee

Parcel I.D.:

Reviewed by:

Date:

Property Owner:

Property Location: Govt. Lot NE¼ of NW ¼ S29 T7N R21E

Property Owner' Mail Address:

Lot #: | Block #: | Subd. Name or CSM #:

City: | State: | Zip Code: | Phone Number:

[x] City [ ] Village [ ] Town | Nearest Road: W. Innovation Drive

Wauwatosa, WI

Drainage area: sq. ft. acres

Test site suitable for (check all that apply) [ ] Site not suitable:

[ ] Bioretention [ ] Subsurface Dispersal System:

[ ] Reuse: [ ] Irrigation: [ ] Other:

Hydraulic Application Test Method

___ Morphological Evaluation

___ Double Ring Infiltrometer

___ Other: (specify) _____

Soil Moisture

Date of soil borings:

USDA-NRCS WETS Value:

___ Dry = 1;

___ Normal = 2;

___ Wet = 3

7 #OBS. [ ] Pit [x] Boring Ground surface elevation 99.7 ft. Elevation of limiting factor _____ ft.

Horizon	Depth in.	Dominant Color Munsell	Redox Description Qu. Sz. Cont. Color	Texture	Structure Gr. Sz. Sh.	Consistence	Boundary	% Rock Frags.	% Fines	Hydraulic App Rate inches/Hr
FILL	0-24	10YR 3/3		CL	1, VF, SBK	M, FR	A, S	<5	85	0.03
FILL	24-48	10YR 5/4		CL	MA	M, FI	C, S	35	85	0.03
B	48-180	10YR 5/4		CL	1, F, SBK	M, FI	G, W	10	85	0.03
C	108-252	10YR 5/1		CL	1, F, SBK	M, FI	--	10	85	0.03

Comments: Water encountered at about ±11 feet below ground surface

8 #OBS. [ ] Pit [x] Boring Ground surface elevation 99.4 ft. Elevation of limiting factor _____ ft.

Horizon	Depth in.	Dominant Color Munsell	Redox Description Qu. Sz. Cont. Color	Texture	Structure Gr. Sz. Sh.	Consistence	Boundary	% Rock Frags.	% Fines	Hydraulic App Rate inches/Hr
A	0-14	10YR 3/3		CL	1, VF, SBK	M, FR	A, S	<5	85	0.03
B	14-78	10YR 5/4		CL	MA	M, FI	G, W	10	85	0.03
C	78-252	10YR 5/1		CL	1, F, SBK	M, FI	--	10	85	0.03

Comments: Water encountered at about ±11 feet below ground surface

9 #OBS. [ ] Pit [x] Boring Ground surface elevation 98.6 ft. Elevation of limiting factor _____ ft.

Horizon	Depth in.	Dominant Color Munsell	Redox Description Qu. Sz. Cont. Color	Texture	Structure Gr. Sz. Sh.	Consistence	Boundary	% Rock Frags.	% Fines	Hydraulic App Rate inches/Hr
A	0-14	10YR 3/3		CL	1, VF, SBK	M, FR	A, S	<5	85	0.03
B	14-78	10YR 5/4		CL	MA	M, FI	G, W	10	85	0.03
C	78-252	10YR 5/1		CL	1, F, SBK	M, FI	--	10	85	0.03

Comments: Water encountered at about ±9 feet below ground surface

Name (Please Print)	Signature	Credential Number
Kevin T. Bugel, P.G.		P.G. No.: 178-13
Address	**Date Evaluation Conducted**	**Telephone Number**
N8 W22350 Johnson Drive, Waukesha, WI	February 7, 2023	262-544-0118

APPENDIX B

FIELD PROCEDURES

The field operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) designation D

420 entitled “Standard Guide for Sampling Rock and Rock” and/or other relevant specifications. Soil samples were preserved and transported to *Giles’* laboratory in general accordance with the procedures recommended by ASTM designation D 4220 entitled “Standard Practice for Preserving and Transporting Soil Samples.” Brief descriptions of the sampling, testing and field procedures commonly performed by *Giles* are provided herein.

GENERAL FIELD PROCEDURES

Test Boring Elevations

The ground surface elevations reported on the Test Boring Logs are referenced to the assumed benchmark shown on the Boring Location Plan (Figure 1). Unless otherwise noted, the elevations were determined with a conventional hand-level and are accurate to within about 1 foot.

Test Boring Locations

The test borings were located on-site based on the existing site features and/or apparent property lines. Dimensions illustrating the approximate boring locations are reported on the Boring Location Plan (Figure 1).

Water Level Measurement

The water levels reported on the Test Boring Logs represent the depth of "free" water encountered during drilling and/or after the drilling tools were removed from the borehole. Water levels measured within a granular (sand and gravel) soil profile are typically indicative of the water table elevation. It is usually not possible to accurately identify the water table elevation with cohesive (clayey) soils, since the rate of seepage is slow. The water table elevation within cohesive soils must therefore be determined over a period of time with groundwater observation wells.

It must be recognized that the water table may fluctuate seasonally and during periods of heavy precipitation. Depending on the subsurface conditions, water may also become perched above the water table, especially during wet periods.

Borehole Backfilling Procedures

Each borehole was backfilled upon completion of the field operations. If potential contamination was encountered, and/or if required by state or local regulations, boreholes were backfilled with an "impervious" material (such as bentonite slurry). Borings that penetrated pavements, sidewalks, etc. were "capped" with Portland Cement concrete, asphaltic concrete, or a similar surface material. It must, however, be recognized that the backfill material may settle, and the surface cap may subside, over a period of time. Further backfilling and/or re-surfacing by *Giles'* client or the property owner may be required.

FIELD SAMPLING AND TESTING PROCEDURES

Auger Sampling (AU)

Soil samples are removed from the auger flights as an auger is withdrawn above the ground surface. Such samples are used to determine general soil types and identify approximate soil stratifications. Auger samples are highly disturbed and are therefore not typically used for geotechnical strength testing.

Split-Barrel Sampling (SS) – (ASTM D-1586)

A split-barrel sampler with a 2-inch outside diameter is driven into the subsoil with a 140-pound hammer free-falling a vertical distance of 30 inches. The summation of hammer-blows required to drive the sampler the final 12-inches of an 18-inch sample interval is defined as the "Standard Penetration Resistance" or N-value is an index of the relative density of granular soils and the comparative consistency of cohesive soils. A soil sample is collected from each SPT interval.

Shelby Tube Sampling (ST) – (ASTM D-1587)

A relatively undisturbed soil sample is collected by hydraulically advancing a thin-walled Shelby Tube sampler into a soil mass. Shelby Tubes have a sharp cutting edge and are commonly 2 to 5 inches in diameter.

Bulk Sample (BS)

A relatively large volume of soils is collected with a shovel or other manually-operated tool. The sample is typically transported to *Giles'* materials laboratory in a sealed bag or bucket.

Dynamic Cone Penetration Test (DC) – (ASTM STP 399)

This test is conducted by driving a 1.5-inch-diameter cone into the subsoil using a 15-pound steel ring (hammer), free-falling a vertical distance of 20 inches. The number of hammer-blows required to drive the cone 1¾ inches is an indication of the soil strength and density, and is defined as "N". The Dynamic Cone Penetration test is commonly conducted in hand auger borings, test pits and within excavated trenches.

- Continued -

Ring-Lined Barrel Sampling – (ASTM D 3550)

In this procedure, a ring-lined barrel sampler is used to collect soil samples for classification and laboratory testing. This method provides samples that fit directly into laboratory test instruments without additional handling/disturbance.

Sampling and Testing Procedures

The field testing and sampling operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Results of the field testing (i.e. N-values) are reported on the Test Boring Logs. Explanations of the terms and symbols shown on the logs are provided on the appendix enclosure entitled “General Notes”.

APPENDIX C

LABORATORY TESTING AND CLASSIFICATION

The laboratory testing was conducted under the supervision of a geotechnical engineer in accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Brief descriptions of laboratory tests commonly performed by *Giles* are provided herein.

LABORATORY TESTING AND CLASSIFICATION

Photoionization Detector (PID)

In this procedure, soil samples are "scanned" in *Giles'* analytical laboratory using a Photoionization Detector (PID). The instrument is equipped with an 11.7 eV lamp calibrated to a Benzene Standard and is capable of detecting a minute concentration of **certain** Volatile Organic Compound (VOC) vapors, such as those commonly associated with petroleum products and some solvents. Results of the PID analysis are expressed in HNu (manufacturer's) units rather than actual concentration.

Moisture Content (w) (ASTM D 2216)

Moisture content is defined as the ratio of the weight of water contained within a soil sample to the weight of the dry solids within the sample. Moisture content is expressed as a percentage.

Unconfined Compressive Strength (qu) (ASTM D 2166)

An axial load is applied at a uniform rate to a cylindrical soil sample. The unconfined compressive strength is the maximum stress obtained or the stress when 15% axial strain is reached, whichever occurs first.

Calibrated Penetrometer Resistance (qp)

The small, cylindrical tip of a hand-held penetrometer is pressed into a soil sample to a prescribed depth to measure the soils capacity to resist penetration. This test is used to evaluate unconfined compressive strength.

Vane-Shear Strength (qs)

The blades of a vane are inserted into the flat surface of a soil sample and the vane is rotated until failure occurs. The maximum shear resistance measured immediately prior to failure is taken as the vane-shear strength.

Loss-on-Ignition (ASTM D 2974; Method C)

The Loss-on-Ignition (L.O.I.) test is used to determine the organic content of a soil sample. The procedure is conducted by heating a dry soil sample to 440°C in order to burn-off or "ash" organic matter present within the sample. The L.O.I. value is the ratio of the weight loss due to ignition compared to the initial weight of the dry sample. L.O.I. is expressed as a percentage.

Particle Size Distribution (ASTB D 421, D 422, and D 1140)

This test is performed to determine the distribution of specific particle sizes (diameters) within a soil sample. The distribution of coarse-grained soil particles (sand and gravel) is determined from a “sieve analysis,” which is conducted by passing the sample through a series of nested sieves. The distribution of fine-grained soil particles (silt and clay) is determined from a “hydrometer analysis” which is based on the sedimentation of particles suspended in water.

Consolidation Test (ASTM D 2435)

In this procedure, a series of cumulative vertical loads are applied to a small, laterally confined soil sample. During each load increment, vertical compression (consolidation) of the sample is measured over a period of time. Results of this test are used to estimate settlement and time rate of settlement.

Classification of Samples

Each soil sample was visually-manually classified, based on texture and plasticity, in general accordance with the Unified Soil Classification System (ASTM D-2488-75). The classifications are reported on the Test Boring Logs.

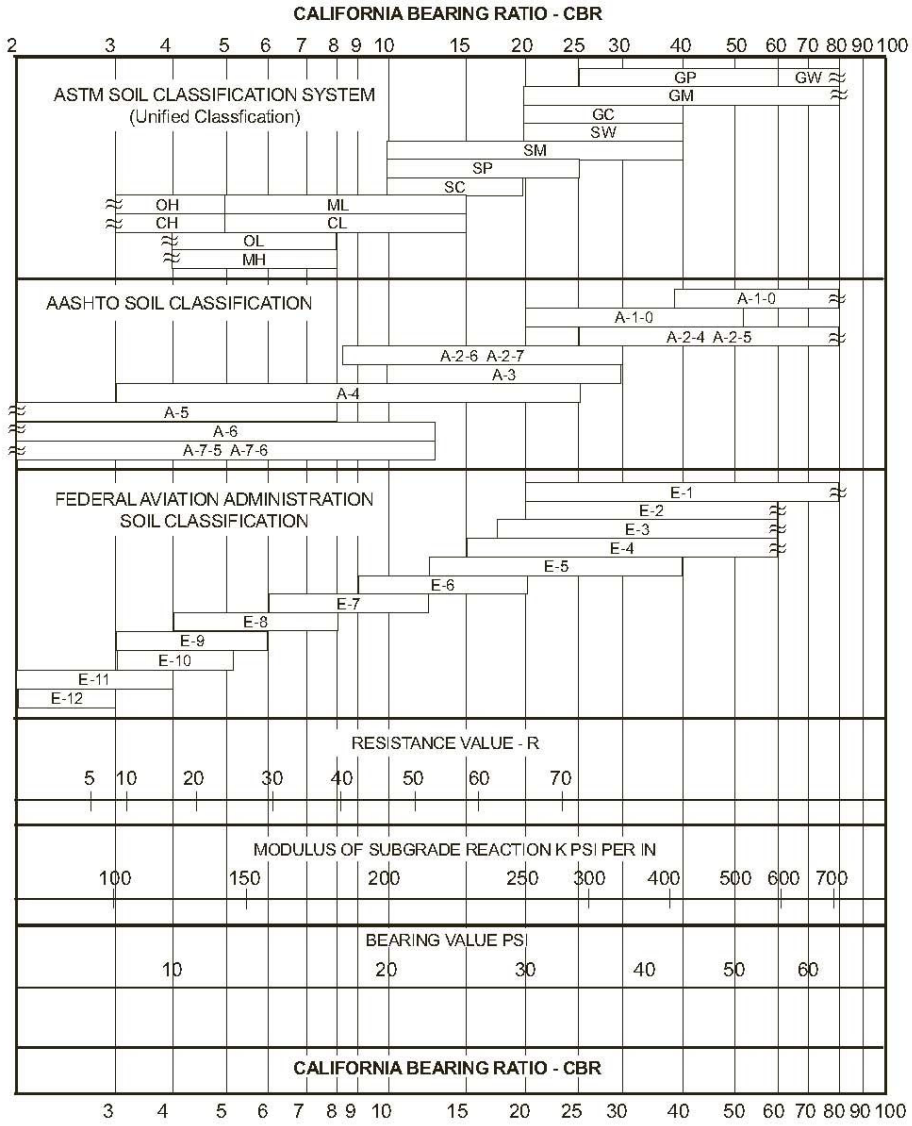
Laboratory Testing

The laboratory testing operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Results of the laboratory tests are provided on the Test Boring Logs or other appendix enclosures. Explanation of the terms and symbols used on the logs is provided on the appendix enclosure entitled “General Notes.”

California Bearing Ratio (CBR) Test ASTM D-1833

The CBR test is used for evaluation of a soil subgrade for pavement design. The test consists of measuring the force required for a 3-square-inch cylindrical piston to penetrate 0.1 or 0.2 inch into a compacted soil sample. The result is expressed as a percent of force required to penetrate a standard compacted crushed stone.

Unless a CBR test has been specifically requested by the client, the CBR is estimated from published charts, based on soil classification and strength characteristics. A typical correlation chart is below.



APPENDIX D

GENERAL INFORMATION

AND
IMPORTANT INFORMATION ABOUT
THIS GEOTECHNICAL REPORT

GENERAL COMMENTS

The soil samples obtained during the subsurface exploration will be retained for a period of thirty days. If no instructions are received, they will be disposed of at that time.

This report has been prepared exclusively for the client in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. Copies of this report may be provided to contractor(s), with contract documents, to disclose information relative to this project. The report, however, has not been prepared to serve as the plans and specifications for actual construction without the appropriate interpretation by the project architect, structural engineer, and/or civil engineer. Reproduction and distribution of this report must be authorized by the client and *Giles*.

This report has been based on assumed conditions/characteristics of the proposed development where specific information was not available. It is recommended that the architect, civil engineer and structural engineer along with any other design professionals involved in this project carefully review these assumptions to ensure they are consistent with the actual planned development. When discrepancies exist, they should be brought to our attention to ensure they do not affect the conclusions and recommendations provided herein. The project plans and specifications may also be submitted to *Giles* for review to ensure that the geotechnical related conclusions and recommendations provided herein have been correctly interpreted.

The analysis of this site was based on a subsoil profile interpolated from a limited subsurface exploration. If the actual conditions encountered during construction vary from those indicated by the borings, *Giles* must be contacted immediately to determine if the conditions alter the recommendations contained herein.

The conclusions and recommendations presented in this report have been promulgated in accordance with generally accepted professional engineering practices in the field of geotechnical engineering. No other warranty is either expressed or implied.

GUIDE SPECIFICATIONS FOR SUBGRADE AND GRADE PREPARATION FOR FILL, FOUNDATION, FLOOR SLAB AND PAVEMENT SUPPORT; AND SELECTION, PLACEMENT AND COMPACTION OF FILL SOILS USING STANDARD PROCTOR PROCEDURES

1. Construction monitoring and testing of subgrades and grades for fill, foundation, floor slab and pavement; and fill selection, placement and compaction shall be performed by an experienced soils engineer and/or his representatives.

2. All compaction fill, subgrades and grades shall be (a) underlain by suitable bearing material; (b) free of all organic, frozen, or other deleterious material, and (c) observed, tested and approved by qualified engineering personnel representing an experienced soils engineer. Preparation of subgrades after stripping vegetation, organic or other unsuitable materials shall consist of (a) proof-rolling to detect soil, wet yielding soils or other unstable materials that must be undercut, (b) scarifying top 6 to 8 inches, (c) moisture conditioning the soils as required, and (d) recompaction to same minimum in-situ density required for similar materials indicated under Item 5. Note: compaction requirements for pavement subgrade are higher than other areas. Weather and construction equipment may damage compacted fill surface and reworking and retesting may be necessary to assure proper performance.

3. In overexcavation and fill areas, the compacted fill must extend (a) a minimum 1 foot lateral distance beyond the exterior edge of the foundation at bearing grade or pavement subgrade and down to compacted fill subgrade on a maximum 0.5(H):1(V) slope, (b) 1 foot above footing grade outside the building, and (c) to floor subgrade inside the building. Fill shall be placed and compacted on a 5(H):1(V) slope or must be stepped or benched as required to flatten if not specifically approved by qualified personnel under the direction of an experienced soil engineer.

4. The compacted fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as “contaminated”, and shall be low-expansive with a maximum Liquid Limit (ASTM D-423) and Plasticity Index (ASTM D-424) of 30 and 15, respectively, unless specifically tested and found to have low expansive properties and approved by an experienced soils engineer. The top 12 inches of compacted fill should have a maximum 3-inch-particle diameter and all underlying compacted fill a maximum 6-inch-diameter unless specifically approved by an experienced soils engineer. All fill materials must be tested and approved under the direction of an experienced soils engineer prior to placement. If the fill is to provide non-frost susceptible characteristics, it must be classified as a clean GW, GP, SW or SP per the Unified Soil Classification System (ASTM D-2487).

5. For structural fill depths less than 20 feet, the density of the structural compacted fill and scarified subgrade and grades shall not be less than 95 percent of the maximum dry density as determined by Standard Proctor (ASTM-698) with the exception of the top 12 inches of pavement subgrade which shall have a minimum in-situ density of 100 percent of maximum dry density, or 5 percent higher than underlying fill materials. Where the structural fill depth is greater than 20 feet, the portions below 20 feet should have a minimum in-place density of 100 percent of its maximum dry density of 5 percent greater than the top 20 feet. The moisture content of cohesive soil shall not vary by more than -1 to +3 percent and granular soil ±3 percent of the optimum when placed and compacted or recompacted, unless specifically recommended/approved by the soils engineer monitoring the placement and compaction. Cohesive soils with moderate to high expansion potentials (PI>15) should, however, be placed, compacted and maintained prior to construction at a moisture content 3±1 percent above optimum moisture content to limit further heave. The fill shall be placed in layers with a maximum loose thickness of 8 inches for foundations and 10 inches for floor slabs and pavement, unless specifically approved by the soils engineer taking into consideration the type of materials and compaction equipment being used. The compaction equipment should consist of suitable mechanical equipment specifically designed for soil compaction. Bulldozers or similar tracked vehicles are typically not suitable for compaction.

6. Excavation, filling, subgrade and grade preparation shall be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs and seepage water encountered shall be pumped or drained to provide a suitable working platform. Springs or water seepage encountered during grading/foundation construction must be called to the soil engineer’s attention immediately for possible construction procedure revision or inclusion of an underdrain system.

7. Non-structural fill adjacent to structural fill should typically be placed in unison to provide lateral support. Backfill along walls must be placed and compacted with care to ensure excessive unbalanced lateral pressures do not develop. The type of fill material placed adjacent to below-grade walls (i.e. basement walls and retaining walls) must be properly tested and approved by an experienced soils engineer with consideration for the lateral pressure used in the wall design.

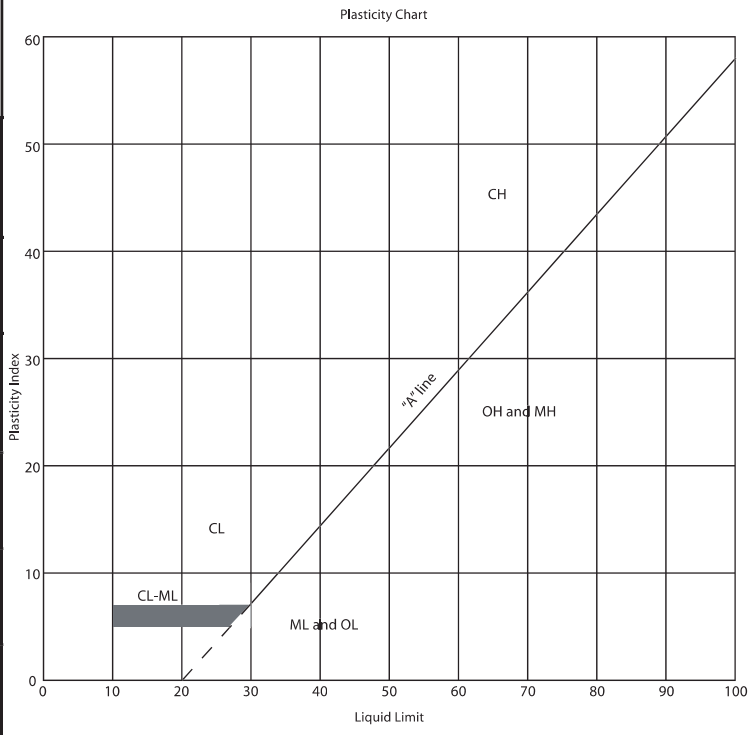
8. Whenever, in the opinion of the soils engineer or the Owner’s Representatives, an unstable condition is being created either by cutting or filling, the work shall not proceed into that area until an appropriate geotechnical exploration and analysis has been performed and the grading plan revised, if found necessary.

CHARACTERISTICS AND RATINGS OF UNIFIED SOIL SYSTEM CLASSES FOR SOIL CONSTRUCTION *									
Class	**Compaction Characteristics**	**Max. Dry Density Standard Proctor (pcf)**	**Compressibility and Expansion**	**Drainage and Permeability**	**Value as an Embankment Material**	**Value as Subgrade When Not Subject to Frost**	**Value as Base Course**	**Value as Temporary Pavement**	
								With Dust Palliative	**With Bituminous Treatment**
GW	Good: tractor, rubber-tired, steel wheel or vibratory roller	125-135	Almost none	Good drainage, pervious	Very stable	Excellent	Good	Fair to poor	Excellent
GP	Good: tractor, rubber-tired, steel wheel or vibratory roller	115-125	Almost none	Good drainage, pervious	Reasonably stable	Excellent to good	Poor to fair	Poor	
GM	Good: rubber-tired or light sheepsfoot roller	120-135	Slight	Poor drainage, semipervious	Reasonably stable	Excellent to good	Fair to poor	Poor	Poor to fair
GC	Good to fair: rubber-tired or sheepsfoot roller	115-130	Slight	Poor drainage, impervious	Reasonably stable	Good	Good to fair **	Excellent	Excellent
SW	Good: tractor, rubber-tired or vibratory roller	110-130	Almost none	Good drainage, pervious	Very stable	Good	Fair to poor	Fair to poor	Good
SP	Good: tractor, rubber-tired or vibratory roller	100-120	Almost none	Good drainage, pervious	Reasonably stable when dense	Good to fair	Poor	Poor	Poor to fair
SM	Good: rubber-tired or sheepsfoot roller	110-125	Slight	Poor drainage, impervious	Reasonably stable when dense	Good to fair	Poor	Poor	Poor to fair
SC	Good to fair: rubber-tired or sheepsfoot roller	105-125	Slight to medium	Poor drainage, impervious	Reasonably stable	Good to fair	Fair to poor	Excellent	Excellent
ML	Good to poor: rubber-tired or sheepsfoot roller	95-120	Slight to medium	Poor drainage, impervious	Poor stability, high density required	Fair to poor	Not suitable	Poor	Poor
CL	Good to fair: sheepsfoot or rubber-tired roller	95-120	Medium	No drainage, impervious	Good stability	Fair to poor	Not suitable	Poor	Poor
OL	Fair to poor: sheepsfoot or rubber-tired roller	80-100	Medium to high	Poor drainage, impervious	Unstable, should not be used	Poor	Not suitable	Not suitable	Not suitable
MH	Fair to poor: sheepsfoot or rubber-tired roller	70-95	High	Poor drainage, impervious	Poor stability, should not be used	Poor	Not suitable	Very poor	Not suitable
CH	Fair to poor: sheepsfoot roller	80-105	Very high	No drainage, impervious	Fair stability, may soften on expansion	Poor to very poor	Not suitable	Very poor	Not suitable
OH	Fair to poor: sheepsfoot roller	65-100	High	No drainage, impervious	Unstable, should not be used	Very poor	Not suitable	Not suitable	Not suitable
Pt	Not suitable		Very high	Fair to poor drainage	Should not be used	Not suitable	Not suitable	Not suitable	Not suitable

* "The Unified Classification: Appendix A - Characteristics of Soil, Groups Pertaining to Roads and Airfields, and Appendix B - Characteristics of Soil Groups Pertaining to Embankments and Foundations," Technical Memorandum 357, U.S. Waterways Ixperiment Station, Vicksburg, 1953.

** Not suitable if subject to frost.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

Major Divisions			Group Symbols	Typical Names	Laboratory Classification Criteria	
Coarse-grained soils (more than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5 percent: GW, GP, SW, SP. More than 12 percent: GM, GC, SM, SC. 5 to 12 percent: *Borderline* cases requiring dual symbols[b]	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3
			GP	Poorly graded gravels, gravel-sand mixtrues, little or no fines		Not meeting all gradation requirements for GW
		Gravels with fines (appreciable amount of fines)	GM[a] d	Silty gravels, gravel-sand-silt mixtures	Atterberg limits below "A" line or P.I. less than 4	Limits plotting within shaded area, above "A" line with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols
			GM[a] u			
			GC	Clayey gravels, gravel-sand-clay mixtures	Atterberg limits above "A" line or P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP	Poorly graded sands, gravelly sands, little or no fines	Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM[a] d	Silty sands, sand-silt mixtures	Atterberg limits below "A" line or P.I. less than 4	Limits plotting within shaded area, above "A" line with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols
			SM[a] u			
			SC	Clayey sands, sand-clay mixtures	Atterberg limits above "A" line or P.I. greater than 7	
Fine-grained soils (More than half material is smaller than No. 200 sieve size)	Silts and clays (Liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity		
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays		
			OL	Organic silts and organic silty clays of low plasticity		
	Silts and clays (Liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts		
			CH	Inorganic clays of high plasticity, fat clays		
			OH	Organic clays of medium to high plasticity, organic silts		
	Highly organic soils		Pt	Peat and other highly organic soils		

[a] Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits, suffix d used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u is used when L.L. is greater than 28.

[b] Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group sympols. For example GW-GC, well-graded gravel-sand mixture with clay binder.

GENERAL NOTES

SAMPLE IDENTIFICATION

All samples are visually classified in general accordance with the Unified Soil Classification System (ASTM D-2487-75 or D-2488-75)

DESCRIPTIVE TERM (% BY DRY WEIGHT)

Trace:	1-10%
Little:	11-20%
Some:	21-35%
And/Adjective	36-50%

PARTICLE SIZE (DIAMETER)

Boulders:	8 inch and larger
Cobbles:	3 inch to 8 inch
Gravel:	coarse - ¾ to 3 inch
	fine – No. 4 (4.76 mm) to ¾ inch
Sand:	coarse – No. 4 (4.76 mm) to No. 10 (2.0 mm)
	medium – No. 10 (2.0 mm) to No. 40 (0.42 mm)
	fine – No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt:	No. 200 (0.074 mm) and smaller (non-plastic)
Clay:	No 200 (0.074 mm) and smaller (plastic)

SOIL PROPERTY SYMBOLS

Dd:	Dry Density (pcf)
LL:	Liquid Limit, percent
PL:	Plastic Limit, percent
PI:	Plasticity Index (LL-PL)
LOI:	Loss on Ignition, percent
Gs:	Specific Gravity
K:	Coefficient of Permeability
w:	Moisture content, percent
qp:	Calibrated Penetrometer Resistance, tsf
qs:	Vane-Shear Strength, tsf
qu:	Unconfined Compressive Strength, tsf
qc:	Static Cone Penetrometer Resistance (correlated to Unconfined Compressive Strength, tsf)
PID:	Results of vapor analysis conducted on representative samples utilizing a Photoionization Detector calibrated to a benzene standard. Results expressed in HNU-Units. (BDL=Below Detection Limit)

DRILLING AND SAMPLING SYMBOLS

SS:	Split-Spoon
ST:	Shelby Tube – 3 inch O.D. (except where noted)
CS:	3 inch O.D. California Ring Sampler
DC:	Dynamic Cone Penetrometer per ASTM Special Technical Publication No. 399
AU:	Auger Sample
DB:	Diamond Bit
CB:	Carbide Bit
WS:	Wash Sample
RB:	Rock-Roller Bit
BS:	Bulk Sample
Note:	Depth intervals for sampling shown on Record of Subsurface Exploration are not indicative of sample recovery, but position where sampling initiated

N: Penetration Resistance per 12 inch interval, or fraction thereof, for a standard 2 inch O.D. (1⅜ inch I.D.) split spoon sampler driven with a 140 pound weight free-falling 30 inches. Performed in general accordance with Standard Penetration Test Specifications (ASTM D-1586). N in blows per foot equals sum of N-Values where plus sign (+) is shown.

Nc: Penetration Resistance per 1¾ inches of Dynamic Cone Penetrometer. Approximately equivalent to Standard Penetration Test N-Value in blows per foot.

Nr: Penetration Resistance per 12 inch interval, or fraction thereof, for California Ring Sampler driven with a 140 pound weight free-falling 30 inches per ASTM D-3550. Not equivalent to Standard Penetration Test N-Value.

SOIL STRENGTH CHARACTERISTICS

COHESIVE (CLAYEY) SOILS

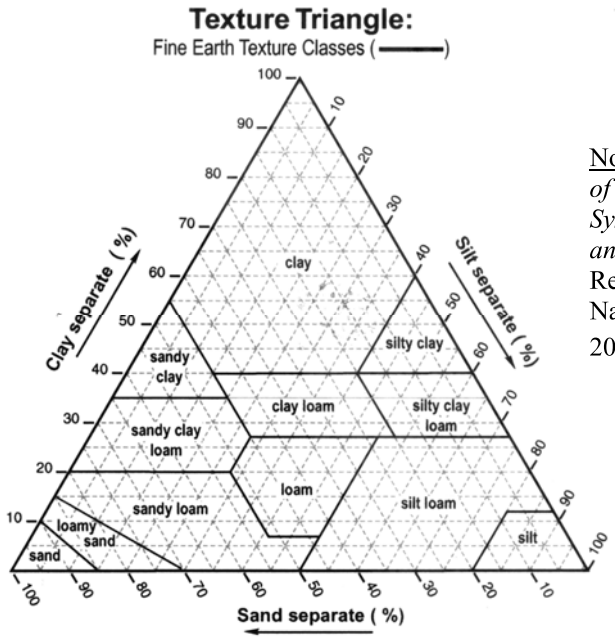
COMPARATIVE CONSISTENCY	BLOWS PER FOOT (N)	UNCONFINED COMPRESSIVE STRENGTH (TSF)
Very Soft	0 - 2	0 - 0.25
Soft	3 - 4	0.25 - 0.50
Medium Stiff	5 – 8	0.50 - 1.00
Stiff	9 – 15	1.00 - 2.00
Very Stiff	16 – 30	2.00 - 4.00
Hard	31+	4.00+

NON-COHESIVE (GRANULAR) SOILS

RELATIVE DENSITY	BLOWS PER FOOT (N)
Very Loose	0 - 4
Loose	5 - 10
Firm	11 - 30
Dense	31 - 50
Very Dense	51+

DEGREE OF PLASTICITY	PI	DEGREE OF EXPANSIVE POTENTIAL	PI
None to Slight	0 - 4	Low	0 - 15
Slight	5 - 10	Medium	15 - 25
Medium	11 - 30	High	25+
High to Very High	31+		

SOIL CLASSIFICATION NOTES



Note: *Texture Triangle* and *Comparison of Particle Size Classes in Different Systems* from *Field Book for Describing and Sampling Soil*, USDA Natural Resources Conservation Service National Soil Survey Center (September 2002).

Comparison of Particle Size Classes in Different Systems

	FINE EARTH										ROCK FRAGMENTS					
											channers			flagst.	stones	boulders
USDA [1]	Clay [2]		Silt		Sand						Gravel			Cobbles	Stones	Boulders
	fine	co.	fine	co.	v.fi.	fi.	med.	co.	v. co.		fine	medium	coarse			
millimeters:	0.0002	.002 mm		.02	.05	.1	.25	.5	1	2 mm	5	20	76	250	600 mm	
U.S. Standard Sieve No. *(opening)*:					300 [3]	140	60	35	18	10	4	*(3/4")*	*(3")*	*(10")*	*(25")*	

International [4]	Clay	Silt	Sand		Gravel	Stones	
			fine	coarse			
millimeters:	.002 mm	.02	.20		2 mm	20 mm	
U.S. Standard Sieve No. *(opening)*:					10	*(3/4")*	

Unified [5]	Silt or Clay	Sand			Gravel		Cobbles	Boulders
		fine	medium	co.	fine	coarse		
millimeters:	.074	.42	2 mm	4.8	19	76	300 mm	
U.S. Standard Sieve No. *(opening)*:	200	40	10	4	*(3/4")*	*(3")*		

AASHTO [6,7]	Clay	Silt	Sand		Gravel or Stones			Broken Rock (angular), or Boulders (rounded)
			fine	coarse	fine	med.	co.	
millimeters:	.005 mm	.074	.42	2 mm	9.5	25	75 mm	
U.S. Standard Sieve No.:		200	40	10	*(3/8")*	*(1")*	*(3")*	

phi #:	12	10	9	8	7	6	5	4	3	2	1	0	-1	-2	-3	-4	-5	-6	-7	-8	-9	-10	-12
Modified Wentworth [8]	clay			silt				sand					pebbles						Cobbles	boulders			
millimeters:	.002	.004	.008	.016	.031	.062	.125	.25	.5	1	2 mm	8	16	32	64	256	4092 mm						
U.S. Standard Sieve No.:						230	120	60	35	18	10	5											

1. Soil Survey Staff. 1995. Soil survey Laboratory information manual. USDA, Natural Resources Conservation Service, Soil Survey Investigations Report No. 45, Version 1.0, National Soil Survey Center, Lincoln, NE. 305 p.
2. Soil Survey Staff. 1995. Soil Survey Lab information manual. USDA-NRCS, Soil Survey Investigation Report #45, version 1.0, National Soil Survey Center, Lincoln, NE. Note: Mineralogy studies may subdivide clay into three size ranges; fine (<0.08µm), medium (0.08-0.2µm), and coarse (0.2-2µm); Jackson, 1969.
3. The Soil Survey Lab (Lincoln, NE) uses a no. 300 sieve (0.047 mm opening) for the USDA-sand/silt measurement. A no. 270 sieve (0.053 mm opening) is more readily available and widely used.
4. International Soil Science Society. 1951. *In*: Soil Survey Manual. Soil Survey Staff, USDA-Soil Conservation Service, Agricultural Handbook No. 18, U.S. Gov. Print. Office, Washington, D.C. 214 p.
5. ASTM. 1993. Standard classification of soils for engineering purposes (Unified Soil Classification System). ASTM designation D2487-92. *In*: Soil and rock; dimension stone; geosynthetics. Annual book of ASTM standards-Vol. 04.08.
6. AASHTO. 1986a. Recommended practice for the classification of soils and soil-aggregate mixtures for highway construction purposes. AASHTO designation M145-82. *In*: Standard specifications for transportation materials and methods of sampling and testing; Part 1: Specifications (14th ed.). American Association of State Highway and Transportation Officials, Washington, D.C.
7. AASHTO. 1986b. Standard definitions of terms relating to subgrade, soil-aggregate, and fill materials. AASHTO designation M146-70 (1980). *In*: sampling and testing; Part 1: Specifications (14th ed.). American Association of State Highway and Transportation Officials, Washington, D.C.
8. Ingram, R.L. 1982. Modified Wentworth scale. *In*: Grain-size scales. AGI Date Sheet 29.1. *In*: Dutro, J.T., Dietrich, R.V., and Foose, R.M. 1989. AGI data sheets for geology in the field, laboratory, and office, 3rd edition. American Geological Institute, Washington, D.C.

GILES ENGINEERING ASSOCIATES, INC.

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept*

responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are not final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

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