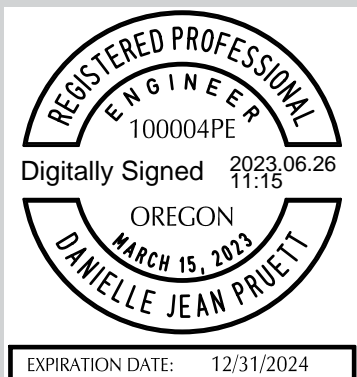EXHIBIT 3.E

Preliminary Stormwater Drainage Report

Beaverton High School Replacement Project

Prepared for: Beaverton School District 48J
Prepared by: James Sweeney, PE
Project Engineer: Danielle Pruett, PE

Junes 2023 | KPFF Project # 2100178

REGISTERED PROFESSIONAL
ENGINEER
100004PE
Digitally Signed   2023.06.26
11:15
OREGON
MARCH 15, 2023
DANIELLE JEAN PRUETT

EXPIRATION DATE: 12/31/2024

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Geotechnical Report

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Introduction

The Beaverton High School Replacement project consists of a new high school building, new parking lots with associated infrastructure and upgrades to athletic fields and roadway improvements along the fronting streets. The project is in an urban area near the center of Beaverton, south of the intersection of SW Farmington Road and SW Cedar Hills Boulevard. The Beaverton High School campus consists of six tax lots. All tax lots are adjacent to either SW Erickson Avenue or SW Stott Avenue (see Appendix A-1). The total area owned by the Beaverton School District at this campus is 27.4 acres. This project, including all phases and public work, will consist of approximately 24.7 acres. The total area of disturbance, including the 24.7 acres of new construction and staging area, totals 25.8 acres.

Existing Conditions

The main campus, which consists of the main school building, cafeteria, track, parking lots and auxiliary structures, is located between SW Erickson Avenue and SW Stott Avenue. The school has sports fields, parking lots and a building in an area located west of SW Erickson Avenue, south of SW 2nd Street and northeast of SW Fairmount Drive.

Storm runoff for the northern portion of the east campus is conveyed north, ultimately to a regional water quality facility off SW Hall Boulevard that was sized to service this area. Storm runoff for the western sports field campus and the southern portion of the east campus ultimately outfalls to Erickson Creek, which is piped underground upstream of the site. Low Impact Development (LIDA) facilities exist throughout the site and will be utilized in the proposed design. Erickson Creek currently runs northwest across the western campus entirely underground. Geotechnical investigations were performed by Geotechnical Resources, Inc. (GRI) on March 24, 2023 (see Appendix D-1).

Proposed Storm Drainage

The proposed development is divided into an east and a west campus (see Appendix A-1) with additional public right of way improvements. Proposed storm infrastructure for the west campus consists of water quality filter structures, a sports field drainage system and underground detention systems. The significant change to this area is the addition of synthetic turf fields with underdrain systems. These fields will collect and convey stormwater to a water quality filter structure prior to it entering underground storage. All storm infrastructure in the west campus ultimately connects to the pipe that conveys Erickson Creek.

Proposed drainage for the east campus includes water quality filter structures, LIDA flow-through planters per Clean Water Services (hereafter referred to as CWS), a sports field drainage system and underground detention systems. The existing condition of the north portion of the main campus which flows north is maintained in the proposed design. Stormwater management for the southern half of the main campus will utilize LIDA facilities or water quality filter structures before entering either a storm main in SW Erickson Avenue or connecting directly to the underground pipe which conveys Erickson Creek.

Proposed drainage for the public right-of-way improvements includes LIDA flow-through planters that are incorporated into the landscape strips and curb extensions. There are some roadway catchment areas that cannot accommodate a storm facility due to limited right of way or conflicts with programming needs. The proposed facilities have been designed to collect and manage runoff from existing and proposed impervious surfaces to meet the required area for stormwater management. New public storm mains will be built in the

roadways to convey runoff to the existing downstream systems and reroute existing lines where required to address conflicts or meet inverts for proposed storm facilities.

The Beaverton municipal code requires that stormwater facilities meet applicable standards in both the Engineering Design Manual and in CWS Design and Construction Standards 2019. Section 4.03.3 of CWS requires a Hydromodification Assessment using the Hydromod Planning Tool (see Appendix A-3), which places this 'Large Project' (greater than 80,000 sf) in a 'Developed Area' with a risk level of 'Low'. Table 4-2 in CWS states that Developed/Low, large projects fall under Category 2. Category 2 stipulates that peak-flow matching for detention may be used to address the hydromodification requirements is CWS 4.03.5(b).
According to CWS Table 4-7, on-site facilities are required to capture runoff such that the 2-year post-development runoff rate does not exceed ½ the 2-year pre-development runoff rate. The 5-year and 10-year post-development runoff rates must not exceed the 5-year and 10-year pre-development rates. Section 500 of the City of Beaverton Engineering Design Manual requires that the 25-year post-development runoff rate does not exceed the 25-year pre-development rate. Rainfall events have been calculated using Autodesk Storm and Sanitary Analysis 2020 (SSA). The selected computational method for runoff calculation is the Santa Barbara Urban Hydrograph (SBUH) method; based on an NRCS Type 1A rainfall distribution. Western basin 5 is unable to meet the above 2-year peak flow matching with the minimum allowable orifice size of 1-inch.

A summary of the basin areas and their respective impervious percentages is below in Table 1.

Table 1: Proposed Private Western Basin Areas

Basin	Impervious Basin Area (sf)	Pervious Basin Area (sf)	Total Basin Area (sf)	Total Basin Area (ac)	Impervious Percentage (%)	2-Year Pre-Development (cfs)	2-Year Post Development (cfs)	25 Year Pre-Development (cfs)	25-Year Post Development (cfs)	Required Volume (cf)
1	72,734	7763	80,497	1.85	90.4	0.19	0.08	0.54	0.35	9600
2	75,019	7916	82,935	1.90	90.5	0.16	0.08	0.56	0.28	11500
3	65,204	11688	76,892	1.77	84.8	0.18	0.08	0.52	0.29	9500
4	18,721	0	39,805	0.91	47.0	0.09	0.04	0.27	0.21	2400
5	12,370	2631	15,001	0.34	82.5	0.03	0.02	0.10	0.07	1600
6	19,369	0	19,369	0.44	100.0	0.04	0.02	0.13	0.10	2500
Total:	263,417	29,998		7.22						

Table 2: Proposed Private Eastern Basin Areas

Basin	Impervious Basin Area (sf)	Pervious Basin Area (sf)	Total Basin Area (sf)	Total Basin Area (ac)	Impervious Percentage (%)	2-Year Pre-Development (cfs)	2-Year Post Development (cfs)	25 Year Pre-Development (cfs)	25-Year Post Development (cfs)	Required Volume (cf)
1	64,759	11811	76,570	1.76	84.6	0.15	0.08	0.49	0.44	8500
2	53,673	20146	73,819	1.69	72.7	0.15	0.08	0.47	0.41	7000
3	72,087	16138	88,225	2.03	81.7	0.13	0.07	0.50	0.40	10500
4	118,871	17,678	136,548	3.13	87.1	0.21	0.11	0.77	0.47	18750
5	73,574	8468	82,042	1.88	89.7	0.12	0.06	0.46	0.30	10000
6	99,524	15,232	114,756	2.63	86.7	0.23	0.10	0.74	0.37	15600
Total:	482,488	89,472		13.13						

Table 3: Proposed Public Northern Basin Areas

Basin	Impervious Basin Area (sf)	Total Basin Area (sf)	Total Area (ac)	Impervious Percentage (%)	Ownership	Treatment Facility	Required Facility Size (sf)	Provided Facility Size (sf)
A01	5,965	7,140	0.16	83.5	Public	Infiltration Planter	358	390
A02	5,900	7,280	0.17	81.0	Public	Infiltration Planter	354	392
A03	4,210	5,475	0.13	76.9	Public	Infiltration Planter	253	300
A04	4,400	5,430	0.12	81.0	Public	Infiltration Planter	264	322
A05	3,695	4,515	0.10	81.8	Public	Infiltration Planter	222	268
A06	5,380	7,140	0.16	75.4	Public	Infiltration Planter	323	391
A07	15,820	24,780	0.57	63.8	Public	No Planter Proposed		
A08	6,160	11,025	0.25	55.9	Public	No Planter Proposed		
A09	3,085	3,085	0.07	100.0	Public	No Planter Proposed		
Total:	54,615	75,870	1.74					

Table 4: Proposed Public Southern Basin Areas

Basin	Impervious Basin Area (sf)	Total Basin Area (sf)	Total Area (ac)	Impervious Percentage (%)	Ownership	Treatment Facility	Required Facility Size (sf)	Provided Facility Size (sf)
B01	6,885	8,885	0.20	77.5	Public	Infiltration Planter	413	440
B02	7,900	9,480	0.22	83.3	Public	Infiltration Planter	474	495
B03	2,375	3,130	0.07	75.9	Public	WQCB, See Table 5		
B04	4,627	6,005	0.14	77.1	Public	Infiltration Planter	278	296
B05	3,025	3,920	0.09	77.2	Public	Infiltration Planter	182	192
B06	4,630	6,005	0.14	77.1	Public	Infiltration Planter	278	296
B07	5,650	6,755	0.16	83.6	Public	Infiltration Planter	339	358
B08	2,715	3,115	0.07	87.2	Public	WQCB, See Table 5		
B09	4,990	5,995	0.14	83.2	Public	Infiltration Planter	299	322
B10	11,230	11,630	0.27	96.6	Public	WQCB, See Table 5		
B11	7,230	8,710	0.20	83.0	Public	WQCB, See Table 5		
B12	2,255	2,580	0.06	87.4	Public	Infiltration Planter	135	143
B13	15,000	17,350	0.40	86.5	Public	Infiltration Planter	900	900
B14	3,770	3,915	0.09	96.3	Public	No Planter Proposed		
B15	1,475	1,925	0.04	76.6	Public	No Planter Proposed		
Total:	83,757	99,400	2.28					

Detention

According to CWS (5.04.2-b-2-C), soil types shall be derived from the NRCS Soil Survey for Washington County (see Appendix C-4).

According to CWS (5.04.2-b-2-B), curve numbers (CN) for the SBUH method shall be found in the Technical Release 55 (TR-55)-Urban Hydrology for Small Watersheds manual. Aerial photography (found in Appendix A-1) indicates that undeveloped areas immediately adjacent to the site consist of a "woods-grass combination". This cover type with a composite hydrologic soil group (see Appendix C-5) results in a pre-development CN of 77 for the eastern site area and 79 for the western site area. According to CWS 4.08.6-d, a CN of 75 shall be used as the pre-developed condition for all modified impervious surfaces.

New impervious areas including paved parking lots, roofs, driveways, etc. shall have a CN of 98. Pervious areas that will be tributary to the proposed storm system and will not be disturbed during construction fall under the meadow cover type with a CN of 78. Excerpted pages of the TR-55 manual showing these selections can be found in Appendix C-2.

The time of concentration (T_c) for the pre-developed condition of the site was calculated using TR-55-based calculation built into the SSA modeling program. Tables 1 and 2 show basin areas and their respective pre-developed and post-developed peak flow for the 2-year and 25-year storm events. Estimates for detention provided by proprietary technology for this site can be found in Appendix B-1.

Water Quality

All water quality facilities are designed for a dry weather storm event totaling 0.36 inches of precipitation falling in 3-hours per the City of Beaverton. Three methods of water quality are utilized for this project.

Eastern Basins 1 through 4 are treated by a regional water quality facility designed and constructed by the City of Beaverton. Runoff from basin 5 will enter either a trapped catch basin or a curb cut before entering a flow-through planter. The method of treatment for these planters is a growing medium (18-inches minimum) through which impervious runoff will vertically infiltrate per LIDA design criteria. The perforated pipe will convey infiltrated water from the facility to the storm drainage network before it outfalls to the storm main in SW Erickson Avenue. Runoff from Basin 6 will be treated with a water-quality treatment structure.

Western Basins 1 through 6 will be treated with water-quality treatment structures.

Stormwater facilities in the right of way are all flow-through planters that provide water quality treatment as summarized above.

Conveyance

Information on stormwater conveyance will be provided with the final site design.

Stormwater Operations and Maintenance Plan

Information on stormwater system operations and maintenance will be provided with the final site design.

Conclusion

The proposed stormwater management plan will achieve pollutant removal to the maximum extent practicable via LIDA planters, water quality structures and a regional water quality facility designed to target pollutants associated with urban development. Stormwater quantity requirements will be met with the installation of LIDA planters and underground storm detention systems. Proposed private water quality and water quantity facilities satisfy the City of Beaverton and CWS water quality and water quantity requirements. As designed, this project shall not create any adverse impacts to the downstream storm system.

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Appendix A

Maps and Exhibits

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APPENDIX A-1

SW TUALATIN VALLEY HWY

U.S. HWY 217

BEAVERTON HIGH SCHOOL REPLACEMENT PROJECT AREA

SW FARMINGTON ROAD

PROPOSED PROJECT AREA

SW STOTT AVENUE

SW FAIRMOUNT DRIVE
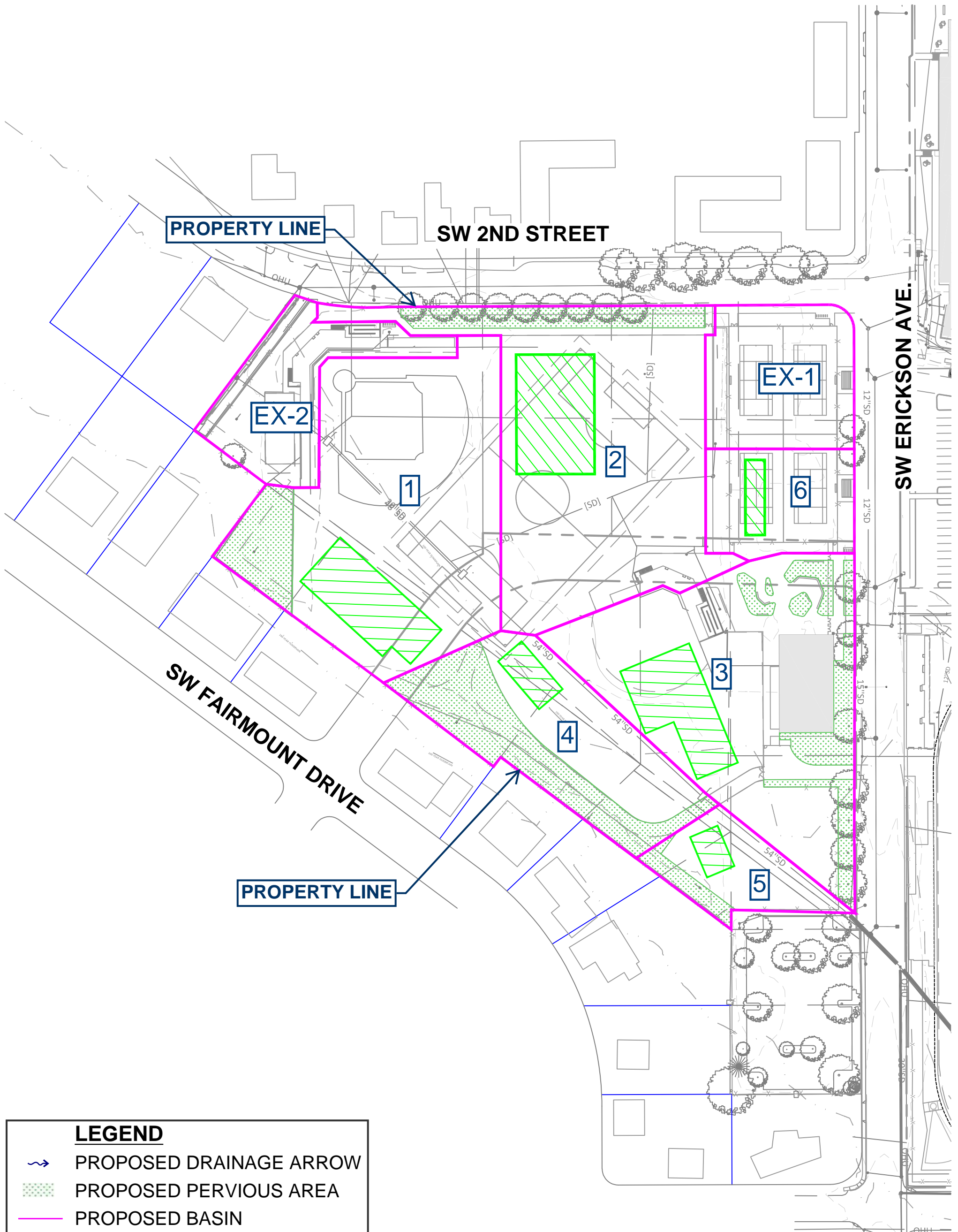
N

kpff

BEAVERTON HIGH SCHOOL REPLACEMENT
VICINITY MAP

A-1

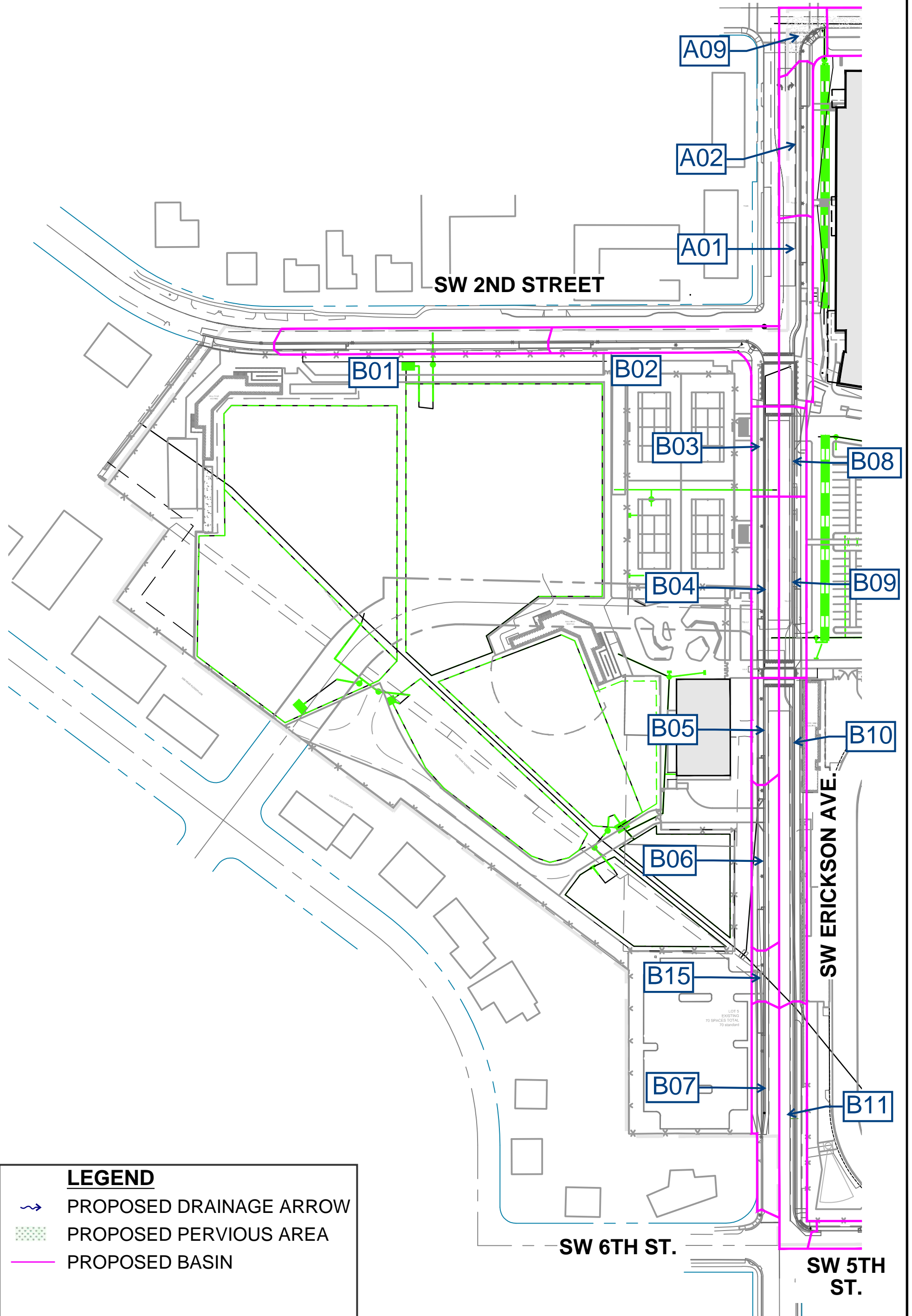#2100178

JUNE 2023

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BEAVERTON HIGH SCHOOL
ON-SITE BASIN MAP
Appendix A-2

SW FARMINGTON RD.

PROPERTY LINE

SW 2ND STREET

SW ERICKSON AVE.

EX-2

EX-1

1

2

6

3

SW FAIRMOUNT DRIVE

4

5

PROPERTY LINE

LEGEND
⤳ PROPOSED DRAINAGE ARROW
▒ PROPOSED PERVIOUS AREA
— PROPOSED BASIN

BEAVERTON HIGH SCHOOL
ON-SITE BASIN MAP
Appendix A-2

PROPERTY LINE

SW FARMINGTON RD.

SW ERICKSON AVE.

1

2

3

5

4

EX-4

EX-1

EX-3

6

SW 5TH ST.

BEAVERTON HIGH SCHOOL
OFF-SITE BASIN MAP
Appendix A-2

SW FARMINGTON RD.

A09

A02

A01

SW 2ND STREET

B01

B02

B03

B08

B04

B09

B05

B10

B06

SW ERICKSON AVE.

B15

B07

B11
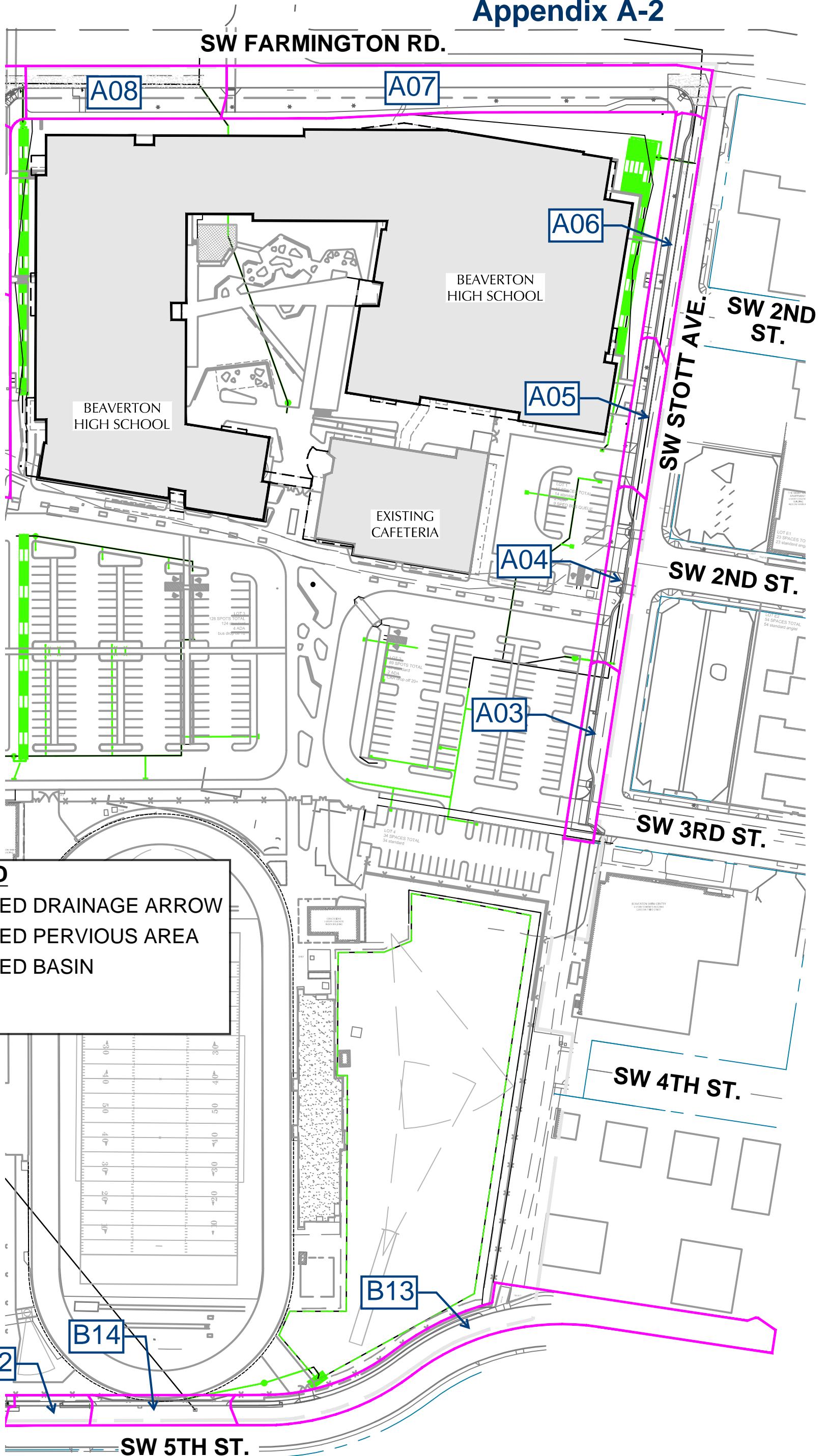
SW 6TH ST.

SW 5TH ST.

LEGEND
- ⤳ PROPOSED DRAINAGE ARROW
- ▒ PROPOSED PERVIOUS AREA
- — PROPOSED BASIN

BEAVERTON HIGH SCHOOL
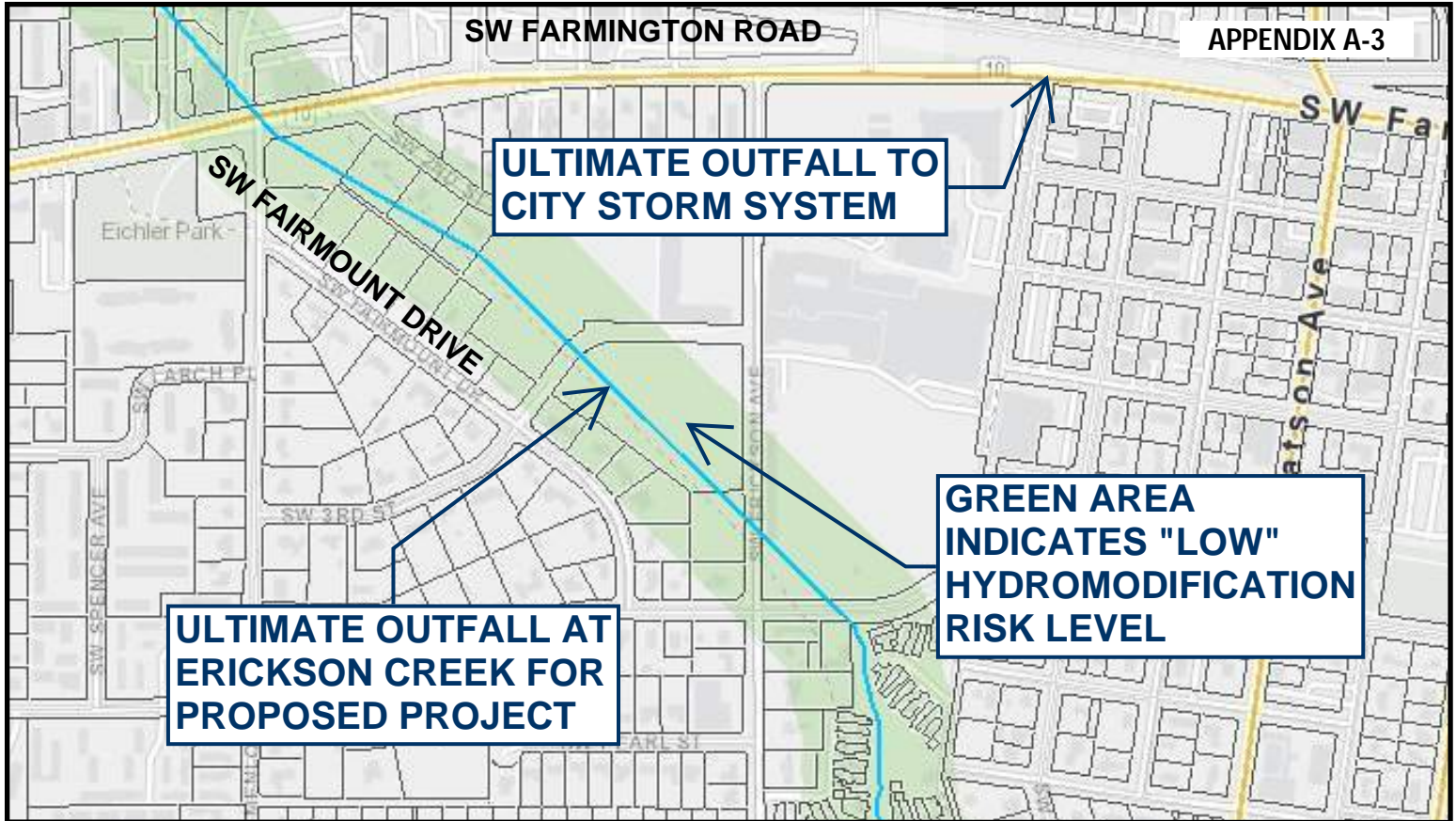
OFF-SITE BASIN MAP
Appendix A-2

SW FARMINGTON RD.

A08

A07

A06

BEAVERTON
HIGH SCHOOL

A05

BEAVERTON
HIGH SCHOOL

SW STOTT AVE.

SW 2ND
ST.

EXISTING
CAFETERIA

A04

SW 2ND ST.

A03

SW 3RD ST.

LEGEND

↝ PROPOSED DRAINAGE ARROW

▦ PROPOSED PERVIOUS AREA

—— PROPOSED BASIN

SW 4TH ST.

B13

B14

B12

SW 5TH ST.
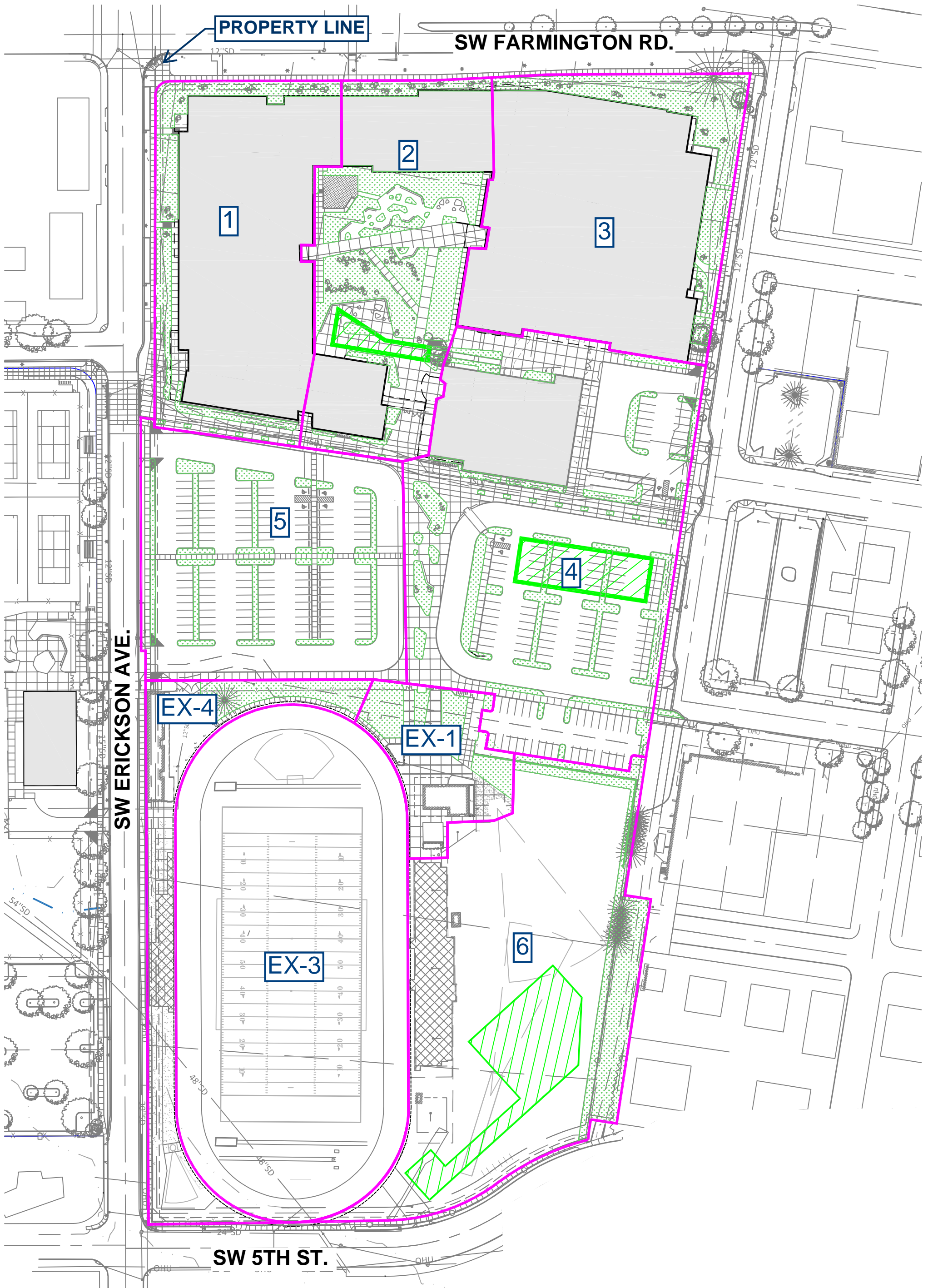
SW FARMINGTON ROAD

ULTIMATE OUTFALL TO CITY STORM SYSTEM

GREEN AREA INDICATES "LOW" HYDROMODIFICATION RISK LEVEL
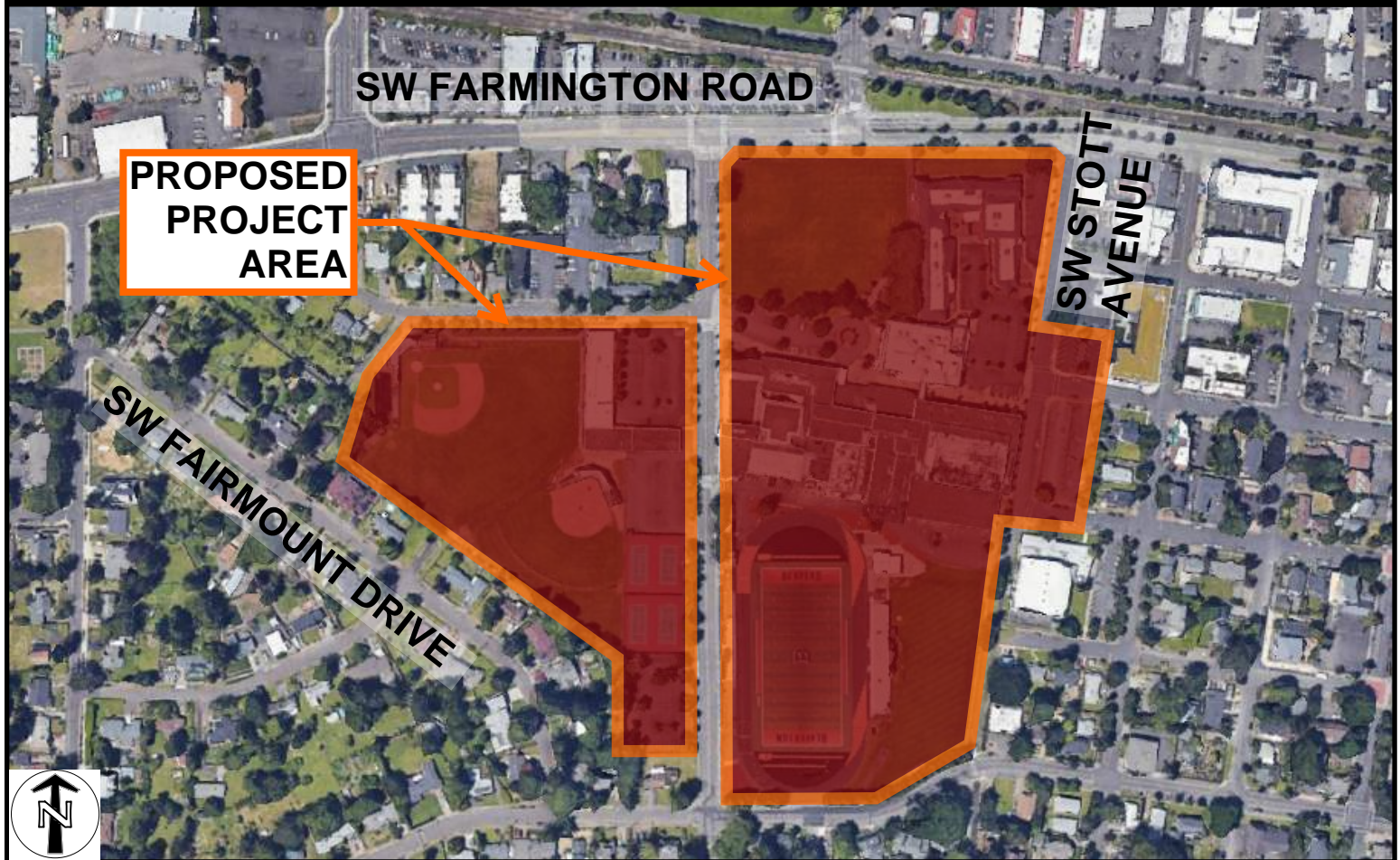
ULTIMATE OUTFALL AT ERICKSON CREEK FOR PROPOSED PROJECT

SW FAIRMOUNT DRIVE

Eichler Park

SW FARMINGTON ROAD

PROPOSED PROJECT AREA

SW STOTT AVENUE

SW FAIRMOUNT DRIVE

BEAVERTON HIGH SCHOOL REPLACEMENT HYDROMOD PLANNING TOOL MAP

A-3

#2100178

MAR 2023

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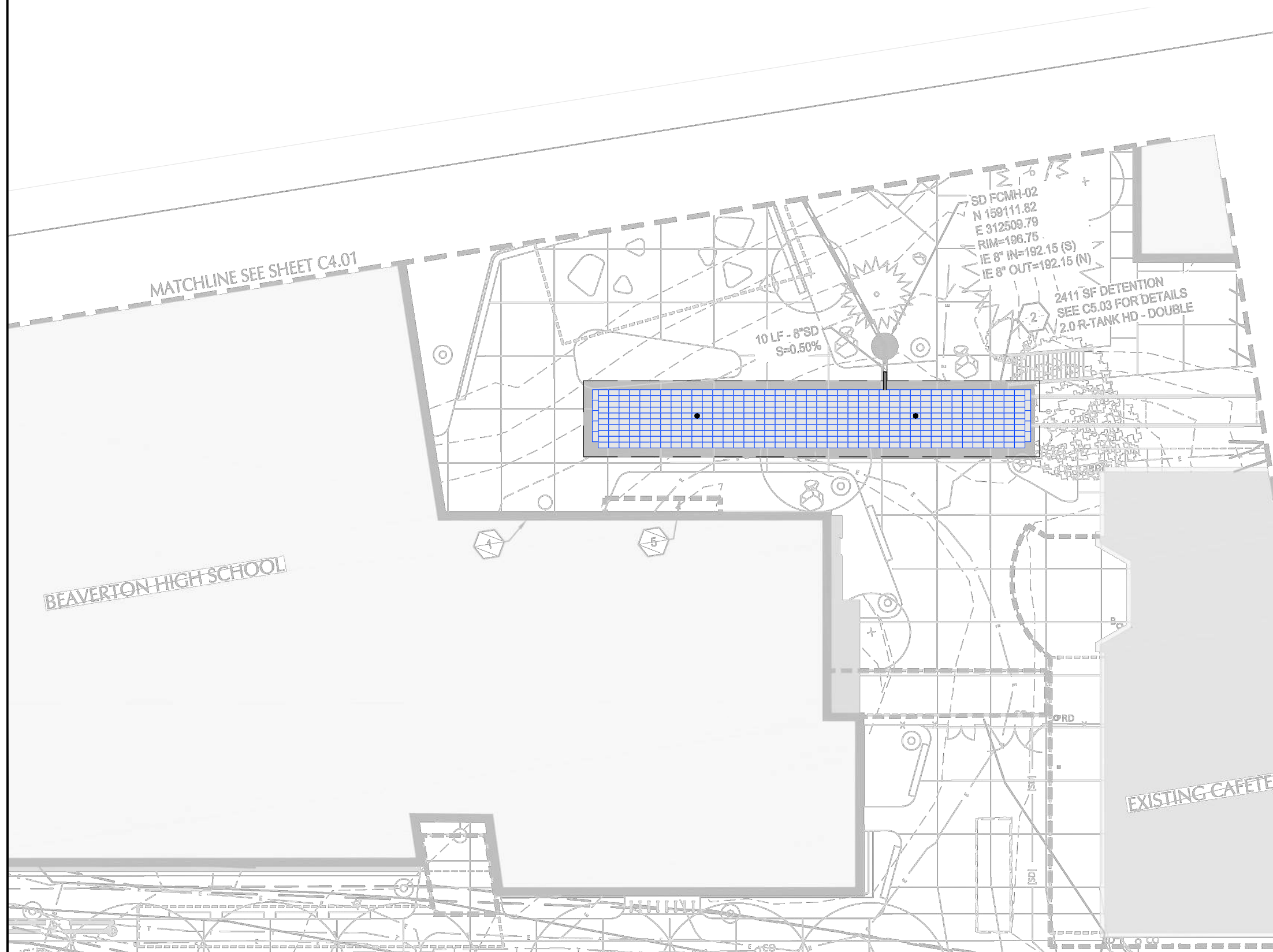
Appendix B

Detention Calcs and Hydrograph

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ACKNOWLEDGEMENT:
AN APPROVAL OF THE SUBMITTAL PLANS IS REQUIRED PRIOR TO MATERIAL ORDER. AS PART OF THE SUBMITTAL APPROVAL, THE ENGINEER OF RECORD HEREBY ACKNOWLEDGES THAT THE R-TANK SYSTEM IS NOT DESIGNED TO SUPPORT LOADS FROM BUILDINGS OR STRUCTURES. THEREFORE, THE ENGINEER OF RECORD HAS COORDINATED WITH THE PROPER DISCIPLINES TO ENSURE NO STRUCTURAL LOADS ARE IMPARTED UPON THE SYSTEM AND ANY INFILTRATION FROM THE SYSTEM SITE HAS BEEN ACCOUNTED FOR IN THE FOUNDATION DESIGN.

NOTES:
- THE CONTRACTOR SHALL PARTICIPATE IN A PRECONSTRUCTION MEETING AND SIGN THE PRECONSTRUCTION CHECKLIST PRIOR TO MATERIAL INSTALLATION.
- DOCUMENTATION SHALL BE RECORDED BY THE CONTRACTOR OR ENGINEER OF RECORD SHOWING PROPER INSTALLATION OF THE SYSTEM AND ALL CONNECTIONS, IN ACCORDANCE WITH MANUFACTURER SPECIFICATIONS.
- IT IS HEREBY RECOMMENDED THAT THE R-TANK SYSTEM BE INSTALLED AFTER THE FOUNDATIONS HAVE BEEN INSTALLED TO ENSURE PROPER SEPARATION DISTANCES ARE MAINTAINED.
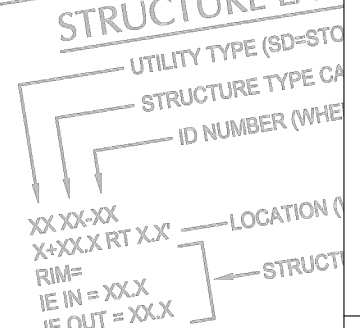
SHEET NOTES
1. PIPE BEDDING AND BACKFILL FOR ALL UTILITIES SH... DETAIL 300/C505.
2. STATIONS AND OFFSETS SHOWN ON STRUCTURE... CENTER OF STRUCTURE.
3. NORTHINGS & EASTINGS SHOWN ON STRUCTURE... CENTER OF STRUCTURE.

KEY NOTES

NOTE	DESCRIPTION
1	DOWNSPOUT WITH CLEANOUT
2	UNDERGROUND STORMWATER MANAGEME... FACILITIES. LAYOUT SHOWN FOR REFEREN... FINAL LAYOUT SHALL BE PER CONTRACTO... SHOP DRAWINGS APPROVED BY THE ENGI... CONTRACTOR TO PROVIDE INSPECTION A... MAINTENANCE STRUCTURES EVERY 100' ... MANUFACTURERS RECOMMENDATION
3	ROUTE STORM DRAIN LINE BELOW BUILD...
4	CONNECT PRIVATE STORM LINE TO PUB... SYSTEM
5	CONNECT CANOPY DRAIN TO STORM D... WITH CLEANOUT
6	CONSTRUCT NEW LIDA PLANTER
7	MAINTAIN FLOW TO EXISTING LIDA FA...
8	CONSTRUCT NEW, PRIVATE STORM ... EXISTING PUBLIC STORM MAIN
9	CONNECT SPORT FIELD UNDERDRA... TO SITE STORM SYSTEM
10	CONNECT PROPOSED STORM SYS... CONVEYANCE OF ERICKSON CRE... APPROXIMATE. FIELD VERIFY POIN...

UTILITY LABEL LEGEN...
STRUCTURE LAB...

UTILITY TYPE (SD=STO...
STRUCTURE TYPE CA...
ID NUMBER (WHE...

XX XX-XX
X+XX.X RT X.X' ——— LOCATION (...
RIM= ——— STRUCT...
IE IN = XX.X
IE OUT = XX.X

PIPE LABEL

SD FCMH-02
N 159111.82
E 312509.79
RIM=196.75
IE 8" IN=192.15 (S)
IE 8" OUT=192.15 (N)

2411 SF DETENTION
SEE C5.03 FOR DETAILS
2.0 R-TANK HD - DOUBLE

10 LF - 6"SD
S=0.50%

MATCHLINE SEE SHEET C4.01

BEAVERTON HIGH SCHOOL

EXISTING CAFETERIA

SEE SHEET C4.06

R-TANK^HD SYSTEM OVERLAY
SCALE: 1" = 25'

0   25'   50'
SCALE 1" = 25'

DRAWN BY
EDQ
DATE
06/23/2023
ACF WEST PROJECT NUMBER
23-017OR
SHEET NO.

R-TANK^HD SYSTEM OVERLAY
BEAVERTON HIGH SCHOOL REBUILD
BEAVERTON, OR
SITE DESIGNATION: RT-E02

1 of 6

DOUBLE R-TANK^HD - ELEVATION

FINISHED SURFACE

MAX. ALLOW. FINAL GRADE=202.01
MIN. ALLOW. FINAL GRADE=196.02

COVER MIN. 12"
MAX. 6.99'

TOP OF TANK=195.02

33.86"

TANK INV=192.20

3"

STONE BASE INV=191.95

DOUBLE R-TANK^HD - MODULE DETAIL

28.15"   15.75"

33.86"   33.86"

SIDE   END

28.15"

TOP   15.75"

(5) SMALL PLATES

ISOMETRIC

MODULE DATA

GEOMETRY:
LENGTH = 28.15 IN. (715 MM)
WIDTH = 15.75 IN. (400 MM)
HEIGHT = 33.86 IN. (860 MM)
TANK VOLUME = 8.69 CF
STORAGE VOLUME = 8.25 CF
VOID INTERNAL VOLUME: 95%
VOID SURFACE AREA: 90%

LOAD RATING:
33.4 PSI, (MODULE ONLY)
HS20/HS25 - SEE SPEC FOR COVER REQUIREMENTS
MATERIAL:
100% RECYCLED POLYPROPYLENE
SMALL PLATES REQUIRED:
5/SEGMENT, 10/MODULE

R-TANK^HD QUANTITIES

R-TANK^HD MODULE TYPE	DOUBLE
# OF DOUBLE R-TANKS	420
TOTAL SYSTEM STORAGE	4,697 CF
R-TANK STORAGE VOLUME	3,465 CF
STONE STORAGE VOLUME (40% VOID RATIO)	1,232 CF
STONE BED FOOTPRINT	1,761 SF
STONE QUANTITY	130 CY
TENCATE MIRAFI 180N NON-WOVEN TANK WRAP	2,214 SF (246 SY)
30 MIL. GEOMEMBRANE LINER EXCAVATION WRAP	5,173 SF (575 SY)
TENCATE MIRAFI 180N NON-WOVEN LINER PROTECTION	10,346 SF (1,150 SY)
12" MAINTENANCE PORTS	2
8" PIPE BOOTS	1
TRASHGUARD PLUS UNITS (RECOMMENDED)	TBD
NOTE: STONE QUANTITY INCLUDES 12" OF COVER AND 3" OF BASE.	
NOTE: GEOTEXTILE / LINER QUANTITIES INCLUDE A 15% WASTE FACTOR.	

NOTE:
BASE STONE STORAGE NOT INCLUDED IN VOLUMES.

R-TANK^HD TYPICAL TANK INLET/OUTLET W/ GEOTEXTILE PIPE BOOT DETAIL

CUT AN "X" IN THE FABRIC ENVELOPE THAT IS SLIGHTLY LARGER THAN THE PIPE. PULL THE FABRIC FLAPS AROUND THE PIPE, AND SEAL WITH A STAINLESS STEEL BAND.

GEOTEXTILE FABRIC OVER R-TANK

INLET/OUTLET PIPE

"X" CUT IN THE FABRIC TO ALLOW PIPE/TANK INTERFACE

AFTER TANK WRAP IS SECURED TO PIPE, SLIDE BOOT AGAINST R-TANK AND SECURE WITH SECOND STAINLESS STEEL BAND, THEN ATTACH BOOT FLAP TO TANK ENVELOPE FABRIC WITH DUCT TAPE OR OTHER ADHESIVE.

END VIEW OF PIPE/FABRIC CONNECTION

GEOTEXTILE FABRIC OVER R-TANK

NOTE: PIPE MUST BUTT DIRECTLY AGAINST R-TANK. PIPE EFFLUENT SHALL NOT PASS THROUGH FABRIC ENVELOPE

STAINLESS STEEL BAND USED TO FASTEN FABRIC TO PIPES TO PREVENT BACKFILL FROM ENTERING STRUCTURE

SIDE VIEW OF PIPE/FABRIC CONNECTION

FABRIC COLLAR TO FIT OUTSIDE DIAMETER OF INLET/OUTLET PIPE

PIPE

12 OZ/SY NON-WOVEN GEOTEXTILE, TRIM AS NEEDED

STAINLESS STEEL BAND

GEOTEXTILE BOOT

± 5.0'

± 5.0'

PIPE O.D.

FRONT VIEW OF GEOTEXTILE BOOT

GEOTEXTILE FABRIC OVER R-TANK

2'-0"

PIPE O.D.

STAINLESS STEEL BAND

SIDE VIEW OF GEOTEXTILE BOOT

N 158895.??
E 312743.55
IE 8" IN=195.28 (NE)
IE 8" OUT=195.28 (S)

SD MH-04
N 158890.39
E 312847.29
RIM=201.50
IE 12" IN=191.80 (S)
IE 12" OUT=191.60 (N)

70 LF - 8"SD
S=0.50%

61 LF - 12"SD
S=0.63%

22 LF - 8"SD
S=2.00%

SD CB-07
N 158901.97
E 312716.51
RIM=202.42
IE 8" OUT=195.54 (E)

SD WYE-10
N 158898.69
E 312737.95
IE 8" IN=195.10 (W)

7500 SF DETENTION
SEE C5.03 FOR DETAILS
2.0 R-TANK HD - DOUBLE

32 LF - 12"SD
S=0.50%

55 LF - 8"SD
S=3.28%

SD FCMH-06
N 158835.43
E 312806.53
RIM=203.01
IE 12" IN=192.33 (W)
IE 12" OUT=192.33 (E)

14 LF - 12"SD
S=0.50%

SD CB-12
N 158793.05
E 312730.57
RIM=201.86
IE 8" OUT=195.23 (W)

SD WYE-05
N 158799.20
E 312692.10
IE 8" IN=195.03 (E)

39 LF - 8"SD
S=0.50%

SD CB-14
N 158802.60
E 312623.81
RIM=200.46
IE 8" OUT=197.02 (N)

MATCHLINE SEE SHEET C4.10

200

R-TANK HD SYSTEM OVERLAY
BEAVERTON HIGH SCHOOL REBUILD
BEAVERTON, OR
SITE DESIGNATION: RT-E04

R-TANK HD SYSTEM OVERLAY
SCALE: 1" = 20'

0   20'   40'
SCALE 1" =   20'

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DOUBLE R-TANK^HD - ELEVATION

FINISHED SURFACE

MAX. ALLOW. FINAL GRADE=202.21
MIN. ALLOW. FINAL GRADE=196.89

GEOGRID ELEV=196.22

TOP OF TANK=195.22

TANK INV=192.40

STONE BASE INV=192.15

COVER MIN. 20" MAX. 6.99'
12"
33.86"
3"

DOUBLE R-TANK^HD - MODULE DETAIL

28.15"
15.75"
33.86"
33.86"

SIDE
END

(5) SMALL PLATES

28.15"
15.75"

TOP

ISOMETRIC

MODULE DATA

GEOMETRY:
LENGTH = 28.15 IN. (715 MM)
WIDTH = 15.75 IN. (400 MM)
HEIGHT = 33.86 IN. (860 MM)
TANK VOLUME = 8.69 CF
STORAGE VOLUME = 8.25 CF
VOID INTERNAL VOLUME: 95%
VOID SURFACE AREA: 90%

LOAD RATING:
33.4 PSI, (MODULE ONLY)
HS20/HS25 - SEE SPEC FOR
COVER REQUIREMENTS
MATERIAL:
100% RECYCLED POLYPROPYLENE
SMALL PLATES REQUIRED:
5/SEGMENT, 10/MODULE

R-TANK^HD QUANTITIES

R-TANK^HD MODULE TYPE	DOUBLE
# OF DOUBLE R-TANKS	2,446
# OF DOUBLE R-TANK ACCESS MODULES	30
TOTAL SYSTEM STORAGE	24,758 CF
R-TANK STORAGE VOLUME	20,427 CF
STONE STORAGE VOLUME (40% VOID RATIO)	4,331 CF
STONE BED FOOTPRINT	8,460 SF
STONE QUANTITY	479 CY
TENCATE MIRAFI 180N NON-WOVEN TANK WRAP	10,101 SF (1,122 SY)
30 MIL. GEOMEMBRANE LINER EXCAVATION WRAP	21,435 SF (2,382 SY)
TENCATE MIRAFI 180N NON-WOVEN LINER PROTECTION	42,870 SF (4,763 SY)
TENCATE MIRAFI 180N NON-WOVEN TREAT. ROW WRAP	590 SF (66 SY)
TENCATE MIRAFI FW-402 WOVEN TREAT. ROW BASE FABRIC	582 SF (65 SY)
NAUE SECUGRID 30/30 GEOGRID	11,226 SF (1,247 SY)
12" MAINTENANCE PORTS	3
12" INSPECTION PORTS	3
12" PIPE BOOTS	4
TRASHGUARD PLUS UNITS (RECOMMENDED)	9
NOTE: STONE QUANTITY INCLUDES 12" OF COVER AND 3" OF BASE.	
NOTE: GEOTEXTILE / LINER QUANTITIES INCLUDE A 15% WASTE FACTOR.	

NOTE:
1. STORAGE VOLUME FROM ELEVATION 192.55 TO 195.05 = 18,935 CF
2. BASE STONE STORAGE NOT INCLUDED IN VOLUMES.

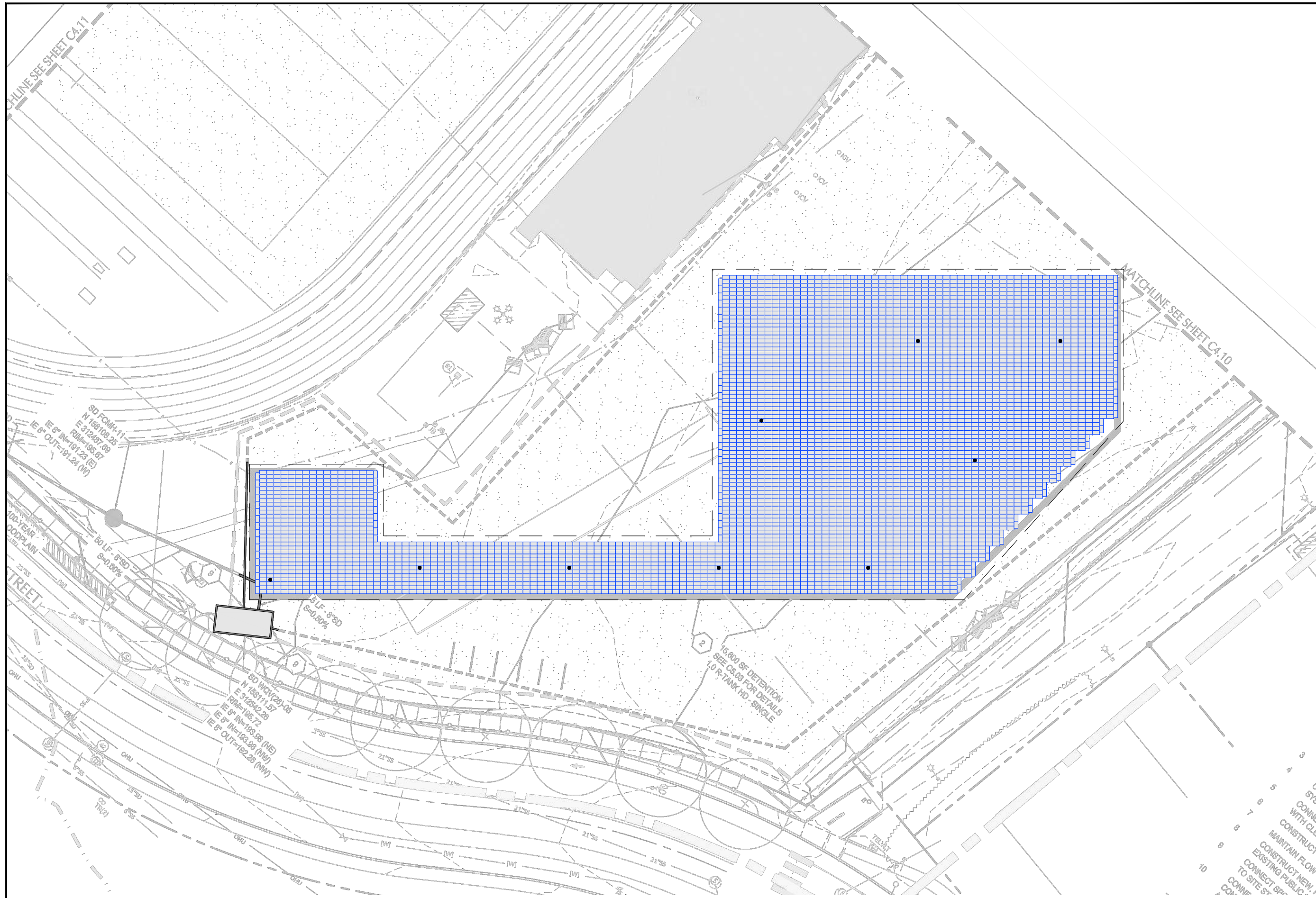### R-TANK^HD TYPICAL TANK INLET/OUTLET W/ GEOTEXTILE PIPE BOOT DETAIL

CUT AN "X" IN THE FABRIC ENVELOPE THAT IS SLIGHTLY LARGER THAN THE PIPE. PULL THE FABRIC FLAPS AROUND THE PIPE, AND SEAL WITH A STAINLESS STEEL BAND.

GEOTEXTILE FABRIC OVER R-TANK

INLET/OUTLET PIPE

"X" CUT IN THE FABRIC TO ALLOW PIPE/TANK INTERFACE

END VIEW OF PIPE/FABRIC CONNECTION

AFTER TANK WRAP IS SECURED TO PIPE, SLIDE BOOT AGAINST R-TANK AND SECURE WITH SECOND STAINLESS STEEL BAND, THEN ATTACH BOOT FLAP TO TANK ENVELOPE FABRIC WITH DUCT TAPE OR OTHER ADHESIVE.

GEOTEXTILE FABRIC OVER R-TANK

NOTE: PIPE MUST BUTT DIRECTLY AGAINST R-TANK. PIPE EFFLUENT SHALL NOT PASS THROUGH FABRIC ENVELOPE

STAINLESS STEEL BAND USED TO FASTEN FABRIC TO PIPES TO PREVENT BACKFILL FROM ENTERING STRUCTURE

SIDE VIEW OF PIPE/FABRIC CONNECTION

FABRIC COLLAR TO FIT OUTSIDE DIAMETER OF INLET/OUTLET PIPE

PIPE

12 OZ/SY NON-WOVEN GEOTEXTILE, TRIM AS NEEDED

STAINLESS STEEL BAND

GEOTEXTILE BOOT

± 5.0'
± 5.0'

PIPE O.D.

FRONT VIEW OF GEOTEXTILE BOOT

GEOTEXTILE FABRIC OVER R-TANK

PIPE O.D.

2'-0"

STAINLESS STEEL BAND

SIDE VIEW OF GEOTEXTILE BOOT

R-TANK^{HD} SYSTEM OVERLAY
SCALE: 1" = 30'

R-TANK^{HD} SYSTEM OVERLAY
BEAVERTON HIGH SCHOOL REBUILD
BEAVERTON, OR
SITE DESIGNATION: RT-E06

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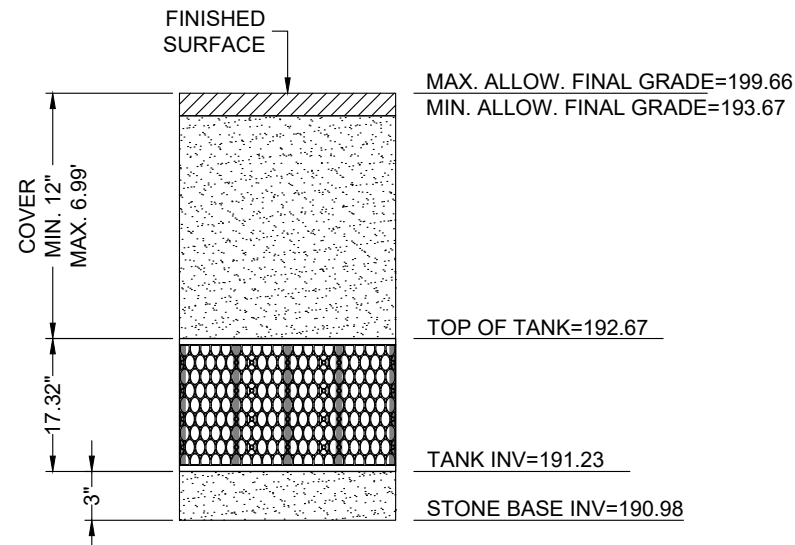DATE
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SCALE 1" = 30'

0   30'   60'
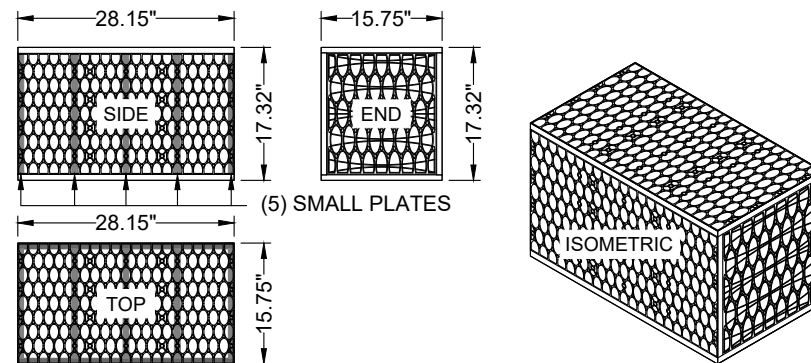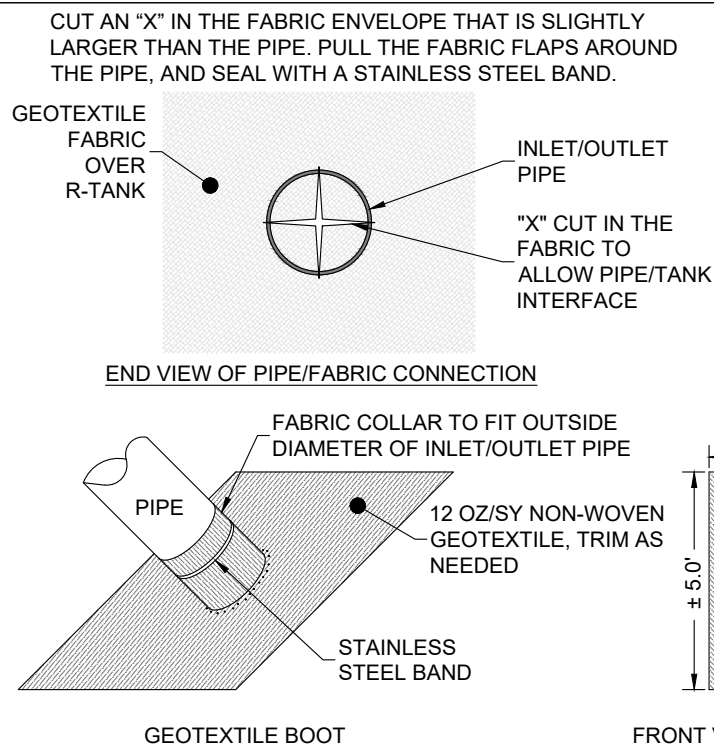
MATCHLINE SEE SHEET C4.11

MATCHLINE SEE SHEET C4.10

15,600 SF DETENTION
SEE C6.03 FOR DETAILS
1.0' R-TANK HD SINGLE

SINGLE R-TANK^HD - ELEVATION

- FINISHED SURFACE
- MAX. ALLOW. FINAL GRADE=199.66
- MIN. ALLOW. FINAL GRADE=193.67
- COVER MIN. 12" / MAX. 6.99'
- TOP OF TANK=192.67
- 17.32"
- TANK INV=191.23
- 3"
- STONE BASE INV=190.98

SINGLE R-TANK^HD - MODULE DETAIL

- SIDE — 28.15" — 17.32"
- END — 15.75" — 17.32"
- (5) SMALL PLATES
- TOP — 28.15" — 15.75"
- ISOMETRIC

MODULE DATA

GEOMETRY:
LENGTH = 28.15 IN. (715 MM)
WIDTH = 15.75 IN. (400 MM)
HEIGHT = 17.32 IN. (440 MM)
TANK VOLUME = 4.44 CF
STORAGE VOLUME = 4.22 CF
VOID INTERNAL VOLUME: 95%
VOID SURFACE AREA: 90%

LOAD RATING:
33.4 PSI, (MODULE ONLY)
HS20/HS25 - SEE SPEC FOR COVER REQUIREMENTS

MATERIAL:
100% RECYCLED POLYPROPYLENE

SMALL PLATES REQUIRED:
5/SEGMENT, 5/MODULE

R-TANK^HD QUANTITIES

R-TANK^HD MODULE TYPE	SINGLE
# OF SINGLE R-TANKS	5,111
TOTAL SYSTEM STORAGE	29,570 CF
R-TANK STORAGE VOLUME	21,568 CF
STONE STORAGE VOLUME (40% VOID RATIO)	8,002 CF
STONE BED FOOTPRINT	17,484 SF
STONE QUANTITY	903 CY
TENCATE MIRAFI 180N NON-WOVEN TANK WRAP	19,468 SF (2,163 SY)
30 MIL. GEOMEMBRANE LINER EXCAVATION WRAP	42,719 SF (4,747 SY)
TENCATE MIRAFI 180N NON-WOVEN LINER PROTECTION	85,438 SF (9,493 SY)
12" MAINTENANCE PORTS	9
8" PIPE BOOTS	2
NOTE: STONE QUANTITY INCLUDES 12" OF COVER AND 3" OF BASE.	
NOTE: GEOTEXTILE / LINER QUANTITIES INCLUDE A 15% WASTE FACTOR.	

NOTE:
1. STORAGE VOLUME FROM ELEVATION 191.23 TO 192.23 = 15,643 CF
2. BASE STONE STORAGE NOT INCLUDED IN VOLUMES.

R-TANK^HD TYPICAL TANK INLET/OUTLET W/ GEOTEXTILE PIPE BOOT DETAIL

END VIEW OF PIPE/FABRIC CONNECTION
CUT AN "X" IN THE FABRIC ENVELOPE THAT IS SLIGHTLY LARGER THAN THE PIPE. PULL THE FABRIC FLAPS AROUND THE PIPE, AND SEAL WITH A STAINLESS STEEL BAND.
- GEOTEXTILE FABRIC OVER R-TANK
- INLET/OUTLET PIPE
- "X" CUT IN THE FABRIC TO ALLOW PIPE/TANK INTERFACE

AFTER TANK WRAP IS SECURED TO PIPE, SLIDE BOOT AGAINST R-TANK AND SECURE WITH SECOND STAINLESS STEEL BAND, THEN ATTACH BOOT FLAP TO TANK ENVELOPE FABRIC WITH DUCT TAPE OR OTHER ADHESIVE.

SIDE VIEW OF PIPE/FABRIC CONNECTION
- GEOTEXTILE FABRIC OVER R-TANK
- NOTE: PIPE MUST BUTT DIRECTLY AGAINST R-TANK. PIPE EFFLUENT SHALL NOT PASS THROUGH FABRIC ENVELOPE.
- STAINLESS STEEL BAND USED TO FASTEN FABRIC TO PIPES TO PREVENT BACKFILL FROM ENTERING STRUCTURE

GEOTEXTILE BOOT
- FABRIC COLLAR TO FIT OUTSIDE DIAMETER OF INLET/OUTLET PIPE
- PIPE
- 12 OZ/SY NON-WOVEN GEOTEXTILE, TRIM AS NEEDED
- STAINLESS STEEL BAND

FRONT VIEW OF GEOTEXTILE BOOT
- ± 5.0'
- ± 5.0'
- PIPE O.D.

SIDE VIEW OF GEOTEXTILE BOOT
- GEOTEXTILE FABRIC OVER R-TANK
- PIPE O.D.
- 2'-0"
- STAINLESS STEEL BAND

NOTES:
- THE CONTRACTOR SHALL PARTICIPATE IN A PRECONSTRUCTION MEETING AND SIGN THE PRECONSTRUCTION CHECKLIST PRIOR TO MATERIAL INSTALLATION.
- DOCUMENTATION SHALL BE RECORDED BY THE CONTRACTOR OR ENGINEER OF RECORD SHOWING PROPER INSTALLATION OF THE SYSTEM AND ALL CONNECTIONS, IN ACCORDANCE WITH MANUFACTURER SPECIFICATIONS.
- IT IS HEREBY RECOMMENDED THAT THE R-TANK SYSTEM BE INSTALLED AFTER THE FOUNDATIONS HAVE BEEN INSTALLED TO ENSURE PROPER SEPARATION DISTANCES ARE MAINTAINED.

9,600 SF DETENTION.
SEE C5.03 FOR DETAILS
1.0 R-TANK HD - SINGLE

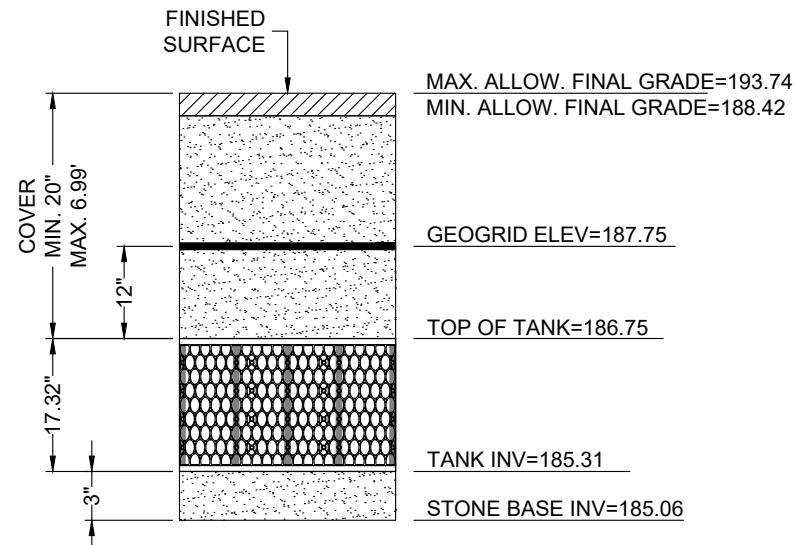R-TANK^{HD} SYSTEM OVERLAY
SCALE: 1" = 20'

R-TANK^{HD} SYSTEM OVERLAY
BEAVERTON HIGH SCHOOL REBUILD
BEAVERTON, OR
SITE DESIGNATION: RT-W01

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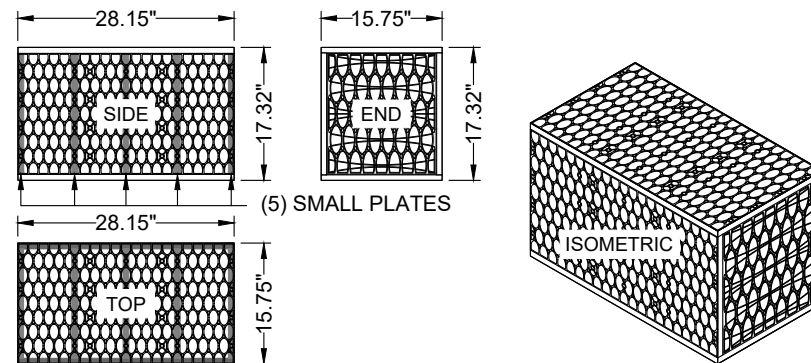DATE
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0   20'   40'
SCALE 1" = 20'

SINGLE R-TANK^HD - ELEVATION

FINISHED SURFACE

MAX. ALLOW. FINAL GRADE=193.74
MIN. ALLOW. FINAL GRADE=188.42

COVER MIN. 20" MAX. 6.99'

GEOGRID ELEV=187.75

TOP OF TANK=186.75

TANK INV=185.31

STONE BASE INV=185.06

12"

17.32"

3"

SINGLE R-TANK^HD - MODULE DETAIL

28.15"
15.75"
17.32"
17.32"
SIDE
END

28.15"
15.75"
TOP

(5) SMALL PLATES

ISOMETRIC

MODULE DATA

GEOMETRY:
LENGTH = 28.15 IN. (715 MM)
WIDTH = 15.75 IN. (400 MM)
HEIGHT = 17.32 IN. (440 MM)
TANK VOLUME = 4.44 CF
STORAGE VOLUME = 4.22 CF
VOID INTERNAL VOLUME: 95%
VOID SURFACE AREA: 90%

LOAD RATING:
33.4 PSI, (MODULE ONLY)
HS20/HS25 - SEE SPEC FOR COVER REQUIREMENTS
MATERIAL:
100% RECYCLED POLYPROPYLENE
SMALL PLATES REQUIRED:
5/SEGMENT, 5/MODULE

R-TANK^HD QUANTITIES

R-TANK^HD MODULE TYPE	SINGLE
# OF SINGLE R-TANKS	3,161
TOTAL SYSTEM STORAGE	18,116 CF
R-TANK STORAGE VOLUME	13,339 CF
STONE STORAGE VOLUME (40% VOID RATIO)	4,777 CF
STONE BED FOOTPRINT	10,637 SF
STONE QUANTITY	541 CY
TENCATE MIRAFI 180N NON-WOVEN TANK WRAP	11,926 SF (1,325 SY)
30 MIL. GEOMEMBRANE LINER EXCAVATION WRAP	25,884 SF (2,876 SY)
TENCATE MIRAFI 180N NON-WOVEN LINER PROTECTION	51,768 SF (5,752 SY)
NAUE SECUGRID 30/30 GEOGRID	13,853 SF (1,539 SY)
12" MAINTENANCE PORTS	4
12" PIPE BOOTS	2
NOTE: STONE QUANTITY INCLUDES 12" OF COVER AND 3" OF BASE.	
NOTE: GEOTEXTILE / LINER QUANTITIES INCLUDE A 15% WASTE FACTOR.	

NOTE:
1. STORAGE VOLUME FROM ELEVATION 185.31 TO 186.31 = 9,604 CF
2. BASE STONE STORAGE NOT INCLUDED IN VOLUMES.

R-TANK^HD TYPICAL TANK INLET/OUTLET W/ GEOTEXTILE PIPE BOOT DETAIL

CUT AN "X" IN THE FABRIC ENVELOPE THAT IS SLIGHTLY LARGER THAN THE PIPE. PULL THE FABRIC FLAPS AROUND THE PIPE, AND SEAL WITH A STAINLESS STEEL BAND.

GEOTEXTILE FABRIC OVER R-TANK

INLET/OUTLET PIPE

"X" CUT IN THE FABRIC TO ALLOW PIPE/TANK INTERFACE

AFTER TANK WRAP IS SECURED TO PIPE, SLIDE BOOT AGAINST R-TANK AND SECURE WITH SECOND STAINLESS STEEL BAND, THEN ATTACH BOOT FLAP TO TANK ENVELOPE FABRIC WITH DUCT TAPE OR OTHER ADHESIVE.

END VIEW OF PIPE/FABRIC CONNECTION

GEOTEXTILE FABRIC OVER R-TANK

NOTE: PIPE MUST BUTT DIRECTLY AGAINST R-TANK. PIPE EFFLUENT SHALL NOT PASS THROUGH FABRIC ENVELOPE

STAINLESS STEEL BAND USED TO FASTEN FABRIC TO PIPES TO PREVENT BACKFILL FROM ENTERING STRUCTURE

SIDE VIEW OF PIPE/FABRIC CONNECTION

FABRIC COLLAR TO FIT OUTSIDE DIAMETER OF INLET/OUTLET PIPE

PIPE

12 OZ/SY NON-WOVEN GEOTEXTILE, TRIM AS NEEDED

STAINLESS STEEL BAND

GEOTEXTILE BOOT

± 5.0'
± 5.0'
PIPE O.D.

FRONT VIEW OF GEOTEXTILE BOOT

GEOTEXTILE FABRIC OVER R-TANK

PIPE O.D.
2'-0"

STAINLESS STEEL BAND

SIDE VIEW OF GEOTEXTILE BOOT

R-TANK^HD SYSTEM DETAILS
BEAVERTON HIGH SCHOOL REBUILD
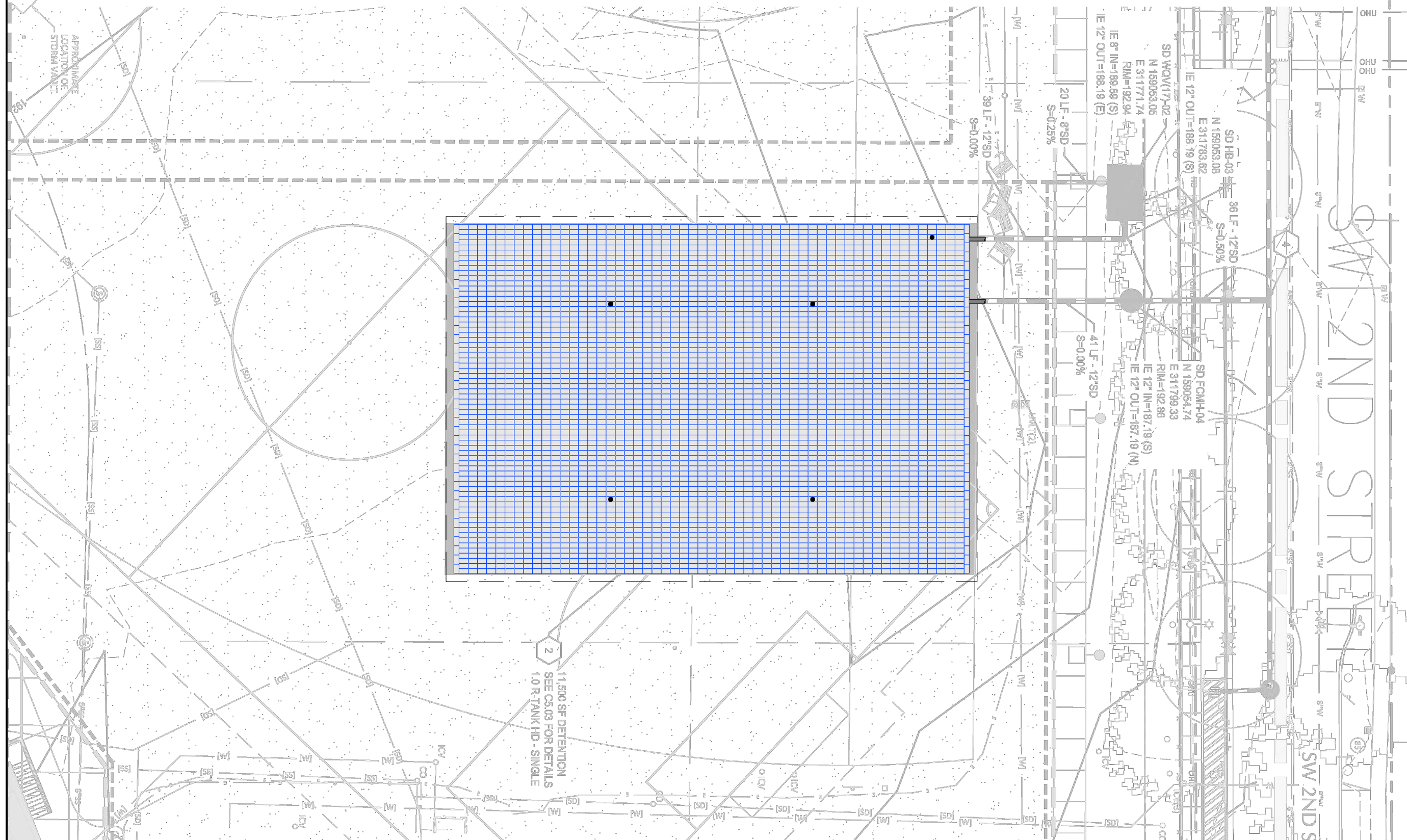BEAVERTON, OR
SITE DESIGNATION: RT-W01

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DATE
06/24/2023
ACF WEST PROJECT NUMBER
23-017OR
SHEET NO.

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ACKNOWLEDGEMENT:
AN APPROVAL OF THE SUBMITTAL PLANS IS REQUIRED PRIOR TO MATERIAL ORDER. AS PART OF THE SUBMITTAL APPROVAL, THE ENGINEER OF RECORD HEREBY ACKNOWLEDGES THAT THE R-TANK SYSTEM IS NOT DESIGNED TO SUPPORT LOADS FROM BUILDINGS OR STRUCTURES. THEREFORE, THE ENGINEER OF RECORD HAS COORDINATED WITH THE PROPER DISCIPLINES TO ENSURE NO STRUCTURAL LOADS ARE IMPARTED UPON THE SYSTEM AND ANY INFILTRATION FROM THE SYSTEM HAS BEEN ACCOUNTED FOR IN THE FOUNDATION DESIGN.
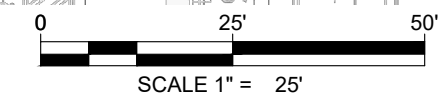
NOTES:
- THE CONTRACTOR SHALL PARTICIPATE IN A PRECONSTRUCTION MEETING AND SIGN THE PRECONSTRUCTION CHECKLIST PRIOR TO MATERIAL INSTALLATION.
- DOCUMENTATION SHALL BE RECORDED BY THE CONTRACTOR OR ENGINEER OF RECORD SHOWING PROPER INSTALLATION OF THE SYSTEM AND ALL CONNECTIONS, IN ACCORDANCE WITH MANUFACTURER SPECIFICATIONS.
- IT IS HEREBY RECOMMENDED THAT THE R-TANK SYSTEM BE INSTALLED AFTER THE FOUNDATIONS HAVE BEEN INSTALLED TO ENSURE PROPER SEPARATION DISTANCES ARE MAINTAINED.

ENGINEER OF RECORD TO REVIEW, APPROVE AND ENDORSE FINAL SITE SPECIFIC DESIGN.

FOR ADDITIONAL INFORMATION PLEASE CONTACT:
ACF WEST, 1-800-423-4567, www.acfwest.com

11,500 SF DETENTION
SEE C5.03 FOR DETAILS
1.0' R-TANK HD - SINGLE

R-TANK^{HD} SYSTEM OVERLAY
SCALE: 1" = 25'

R-TANK^{HD} SYSTEM OVERLAY
BEAVERTON HIGH SCHOOL REBUILD
BEAVERTON, OR
SITE DESIGNATION: RT-W02

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| EDQ |
| DATE |
| 06/24/2023 |
| ACF WEST PROJECT NUMBER |
| 23-017OR |
| SHEET NO. |

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0   25'   50'
SCALE 1" = 25'

SINGLE R-TANK^HD - ELEVATION

FINISHED SURFACE

MAX. ALLOW. FINAL GRADE=195.62
MIN. ALLOW. FINAL GRADE=190.30

COVER MIN. 20" MAX. 6.99'

GEOGRID ELEV=189.63

12"

TOP OF TANK=188.63

17.32"

TANK INV=187.19

3"

STONE BASE INV=186.94

SINGLE R-TANK^HD - MODULE DETAIL

28.15"
SIDE
17.32"

15.75"
END
17.32"

(5) SMALL PLATES

28.15"
TOP
15.75"

ISOMETRIC

MODULE DATA

GEOMETRY:
LENGTH = 28.15 IN. (715 MM)
WIDTH = 15.75 IN. (400 MM)
HEIGHT = 17.32 IN. (440 MM)
TANK VOLUME = 4.44 CF
STORAGE VOLUME = 4.22 CF
VOID INTERNAL VOLUME: 95%
VOID SURFACE AREA: 90%

LOAD RATING:
33.4 PSI, (MODULE ONLY)
HS20/HS25 - SEE SPEC FOR COVER REQUIREMENTS
MATERIAL:
100% RECYCLED POLYPROPYLENE
SMALL PLATES REQUIRED:
5/SEGMENT, 5/MODULE

R-TANK^HD QUANTITIES

R-TANK^HD MODULE TYPE	SINGLE
# OF SINGLE R-TANKS	3,816
TOTAL SYSTEM STORAGE	21,683 CF
R-TANK STORAGE VOLUME	16,104 CF
STONE STORAGE VOLUME (40% VOID RATIO)	5,579 CF
STONE BED FOOTPRINT	12,649 SF
STONE QUANTITY	634 CY
TENCATE MIRAFI 180N NON-WOVEN TANK WRAP	14,245 SF (1,583 SY)
30 MIL. GEOMEMBRANE LINER EXCAVATION WRAP	30,511 SF (3,390 SY)
TENCATE MIRAFI 180N NON-WOVEN LINER PROTECTION	61,022 SF (6,780 SY)
NAUE SECUGRID 30/30 GEOGRID	16,167 SF (1,796 SY)
12" MAINTENANCE PORTS	5
12" PIPE BOOTS	2
NOTE: STONE QUANTITY INCLUDES 12" OF COVER AND 3" OF BASE.	
NOTE: GEOTEXTILE / LINER QUANTITIES INCLUDE A 15% WASTE FACTOR.	

NOTE:
1. STORAGE VOLUME FROM ELEVATION 187.19 TO 188.19 = 11,517 CF
2. BASE STONE STORAGE NOT INCLUDED IN VOLUMES.

R-TANK^HD TYPICAL TANK INLET/OUTLET W/ GEOTEXTILE PIPE BOOT DETAIL

CUT AN "X" IN THE FABRIC ENVELOPE THAT IS SLIGHTLY LARGER THAN THE PIPE. PULL THE FABRIC FLAPS AROUND THE PIPE, AND SEAL WITH A STAINLESS STEEL BAND.

GEOTEXTILE FABRIC OVER R-TANK

INLET/OUTLET PIPE

"X" CUT IN THE FABRIC TO ALLOW PIPE/TANK INTERFACE

END VIEW OF PIPE/FABRIC CONNECTION

AFTER TANK WRAP IS SECURED TO PIPE, SLIDE BOOT AGAINST R-TANK AND SECURE WITH SECOND STAINLESS STEEL BAND, THEN ATTACH BOOT FLAP TO TANK ENVELOPE FABRIC WITH DUCT TAPE OR OTHER ADHESIVE.

GEOTEXTILE FABRIC OVER R-TANK

NOTE: PIPE MUST BUTT DIRECTLY AGAINST R-TANK. PIPE EFFLUENT SHALL NOT PASS THROUGH FABRIC ENVELOPE

STAINLESS STEEL BAND USED TO FASTEN FABRIC TO PIPES TO PREVENT BACKFILL FROM ENTERING STRUCTURE

SIDE VIEW OF PIPE/FABRIC CONNECTION

FABRIC COLLAR TO FIT OUTSIDE DIAMETER OF INLET/OUTLET PIPE

PIPE

12 OZ/SY NON-WOVEN GEOTEXTILE, TRIM AS NEEDED

STAINLESS STEEL BAND

GEOTEXTILE BOOT

± 5.0'
± 5.0'
PIPE O.D.

FRONT VIEW OF GEOTEXTILE BOOT

GEOTEXTILE FABRIC OVER R-TANK

PIPE O.D.

2'-0"

STAINLESS STEEL BAND

SIDE VIEW OF GEOTEXTILE BOOT

ACKNOWLEDGEMENT:
AN APPROVAL OF THE SUBMITTAL PLANS IS REQUIRED PRIOR TO MATERIAL ORDER. AS PART OF THE SUBMITTAL APPROVAL, THE ENGINEER OF RECORD HEREBY ACKNOWLEDGES THAT THE R-TANK SYSTEM IS NOT DESIGNED TO SUPPORT LOADS FROM BUILDINGS OR STRUCTURES. THEREFORE, THE ENGINEER OF RECORD HAS COORDINATED WITH THE PROPER DISCIPLINES TO ENSURE NO STRUCTURAL LOADS ARE IMPARTED UPON THE SYSTEM AND ANY INFILTRATION FROM THE SYSTEM SITE HAS BEEN ACCOUNTED FOR IN THE FOUNDATION DESIGN.

NOTES:
- THE CONTRACTOR SHALL PARTICIPATE IN A PRECONSTRUCTION MEETING AND SIGN THE PRECONSTRUCTION CHECKLIST PRIOR TO MATERIAL INSTALLATION.
- DOCUMENTATION SHALL BE RECORDED BY THE CONTRACTOR OR ENGINEER OF RECORD SHOWING PROPER INSTALLATION OF THE SYSTEM AND ALL CONNECTIONS, IN ACCORDANCE WITH MANUFACTURER SPECIFICATIONS.
- IT IS HEREBY RECOMMENDED THAT THE R-TANK SYSTEM BE INSTALLED AFTER THE FOUNDATIONS HAVE BEEN INSTALLED TO ENSURE PROPER SEPARATION DISTANCES ARE MAINTAINED.

R-TANK^HD SYSTEM OVERLAY
BEAVERTON HIGH SCHOOL REBUILD
BEAVERTON, OR
SITE DESIGNATION: RT-W03

R-TANK^HD SYSTEM OVERLAY
SCALE: 1" = 25'
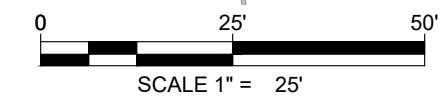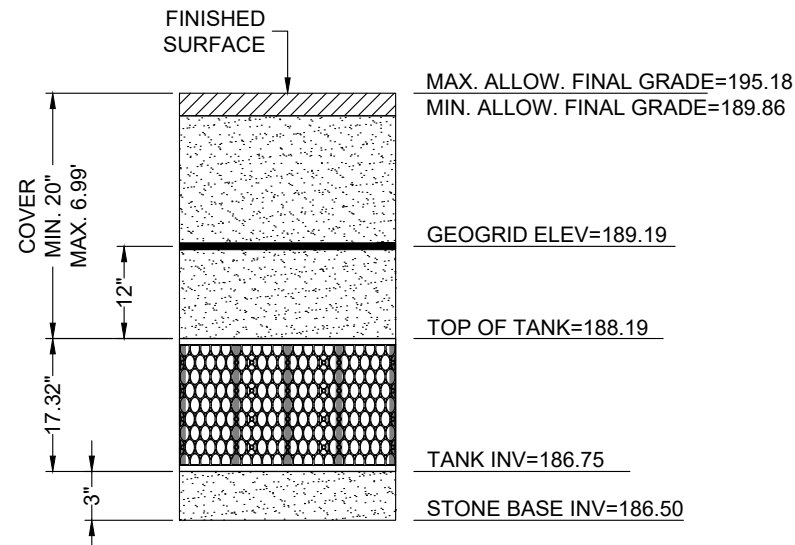
DRAWN BY	EDQ
DATE	06/24/2023
ACF WEST PROJECT NUMBER	23-017OR
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0   25'   50'
SCALE 1" = 25'

SINGLE R-TANK^HD - ELEVATION

FINISHED SURFACE

MAX. ALLOW. FINAL GRADE=195.18
MIN. ALLOW. FINAL GRADE=189.86

COVER MIN. 20" MAX. 6.99'

GEOGRID ELEV=189.19

TOP OF TANK=188.19

12"

17.32"

TANK INV=186.75

3"

STONE BASE INV=186.50

SINGLE R-TANK^HD - MODULE DETAIL

SIDE — 28.15"
END — 15.75"
17.32"
28.15"
(5) SMALL PLATES
TOP — 15.75"
ISOMETRIC

MODULE DATA

GEOMETRY:
LENGTH = 28.15 IN. (715 MM)
WIDTH = 15.75 IN. (400 MM)
HEIGHT = 17.32 IN. (440 MM)
TANK VOLUME = 4.44 CF
STORAGE VOLUME = 4.22 CF
VOID INTERNAL VOLUME: 95%
VOID SURFACE AREA: 90%

LOAD RATING:
33.4 PSI, (MODULE ONLY)
HS20/HS25 - SEE SPEC FOR COVER REQUIREMENTS

MATERIAL:
100% RECYCLED POLYPROPYLENE

SMALL PLATES REQUIRED:
5/SEGMENT, 5/MODULE

R-TANK^HD QUANTITIES

R-TANK^HD MODULE TYPE	SINGLE
# OF SINGLE R-TANKS	3,124
TOTAL SYSTEM STORAGE	17,934 CF
R-TANK STORAGE VOLUME	13,183 CF
STONE STORAGE VOLUME (40% VOID RATIO)	4,751 CF
STONE BED FOOTPRINT	10,543 SF
STONE QUANTITY	538 CY
TENCATE MIRAFI 180N NON-WOVEN TANK WRAP	11,813 SF (1,313 SY)
30 MIL. GEOMEMBRANE LINER EXCAVATION WRAP	25,702 SF (2,856 SY)
TENCATE MIRAFI 180N NON-WOVEN LINER PROTECTION	51,404 SF (5,712 SY)
NAUE SECUGRID 30/30 GEOGRID	13,781 SF (1,531 SY)
12" MAINTENANCE PORTS	4
12" PIPE BOOTS	2
NOTE: STONE QUANTITY INCLUDES 12" OF COVER AND 3" OF BASE.	
NOTE: GEOTEXTILE / LINER QUANTITIES INCLUDE A 15% WASTE FACTOR.	

NOTE:
1. STORAGE VOLUME FROM ELEVATION 186.75 TO 187.75 = 9,504 CF
2. BASE STONE STORAGE NOT INCLUDED IN VOLUMES.

R-TANK^HD TYPICAL TANK INLET/OUTLET W/ GEOTEXTILE PIPE BOOT DETAIL

CUT AN "X" IN THE FABRIC ENVELOPE THAT IS SLIGHTLY LARGER THAN THE PIPE. PULL THE FABRIC FLAPS AROUND THE PIPE, AND SEAL WITH A STAINLESS STEEL BAND.

GEOTEXTILE FABRIC OVER R-TANK

INLET/OUTLET PIPE

"X" CUT IN THE FABRIC TO ALLOW PIPE/TANK INTERFACE

END VIEW OF PIPE/FABRIC CONNECTION

AFTER TANK WRAP IS SECURED TO PIPE, SLIDE BOOT AGAINST R-TANK AND SECURE WITH SECOND STAINLESS STEEL BAND, THEN ATTACH BOOT FLAP TO TANK ENVELOPE FABRIC WITH DUCT TAPE OR OTHER ADHESIVE.

GEOTEXTILE FABRIC OVER R-TANK

NOTE: PIPE MUST BUTT DIRECTLY AGAINST R-TANK. PIPE EFFLUENT SHALL NOT PASS THROUGH FABRIC ENVELOPE

STAINLESS STEEL BAND USED TO FASTEN FABRIC TO PIPES TO PREVENT BACKFILL FROM ENTERING STRUCTURE

SIDE VIEW OF PIPE/FABRIC CONNECTION

FABRIC COLLAR TO FIT OUTSIDE DIAMETER OF INLET/OUTLET PIPE

PIPE

12 OZ/SY NON-WOVEN GEOTEXTILE, TRIM AS NEEDED

STAINLESS STEEL BAND

GEOTEXTILE BOOT

± 5.0'
± 5.0'
PIPE O.D.

FRONT VIEW OF GEOTEXTILE BOOT

GEOTEXTILE FABRIC OVER R-TANK

PIPE O.D.

2'-0"

STAINLESS STEEL BAND

SIDE VIEW OF GEOTEXTILE BOOT
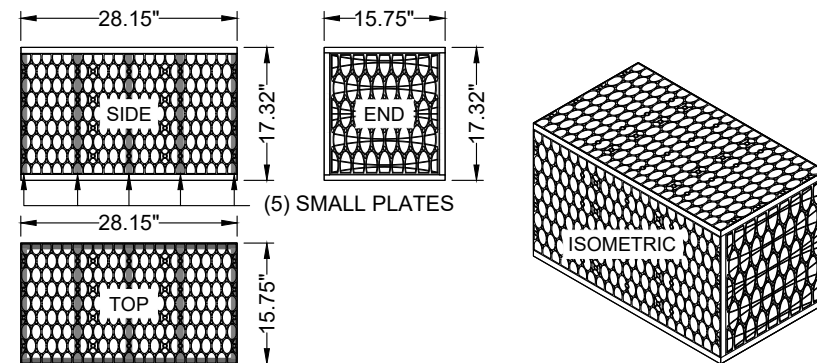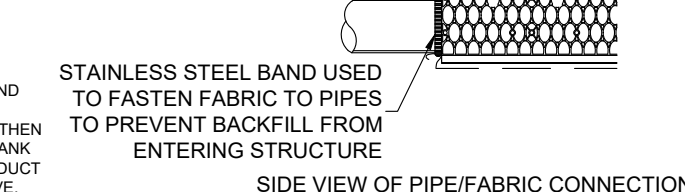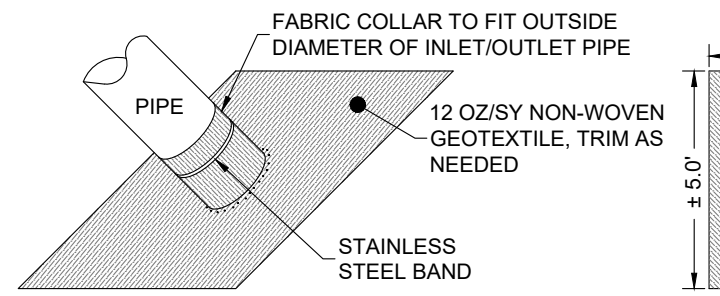
R-TANK^HD SYSTEM DETAILS
BEAVERTON HIGH SCHOOL REBUILD
BEAVERTON, OR
SITE DESIGNATION: RT-W03

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NOTES:
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- DOCUMENTATION SHALL BE RECORDED BY THE CONTRACTOR OR ENGINEER OF RECORD SHOWING PROPER INSTALLATION OF THE SYSTEM AND ALL CONNECTIONS, IN ACCORDANCE WITH MANUFACTURER SPECIFICATIONS.
- IT IS HEREBY RECOMMENDED THAT THE R-TANK SYSTEM BE INSTALLED AFTER THE FOUNDATIONS HAVE BEEN INSTALLED TO ENSURE PROPER SEPARATION DISTANCES ARE MAINTAINED.

R-TANK^HD SYSTEM OVERLAY
SCALE: 1" = 15'

ENGINEER OF RECORD TO REVIEW, APPROVE AND ENDORSE FINAL SITE SPECIFIC DESIGN.

FOR ADDITIONAL INFORMATION PLEASE CONTACT:
ACF WEST, 1-800-423-4567, www.acfwest.com

R-TANK^HD SYSTEM OVERLAY
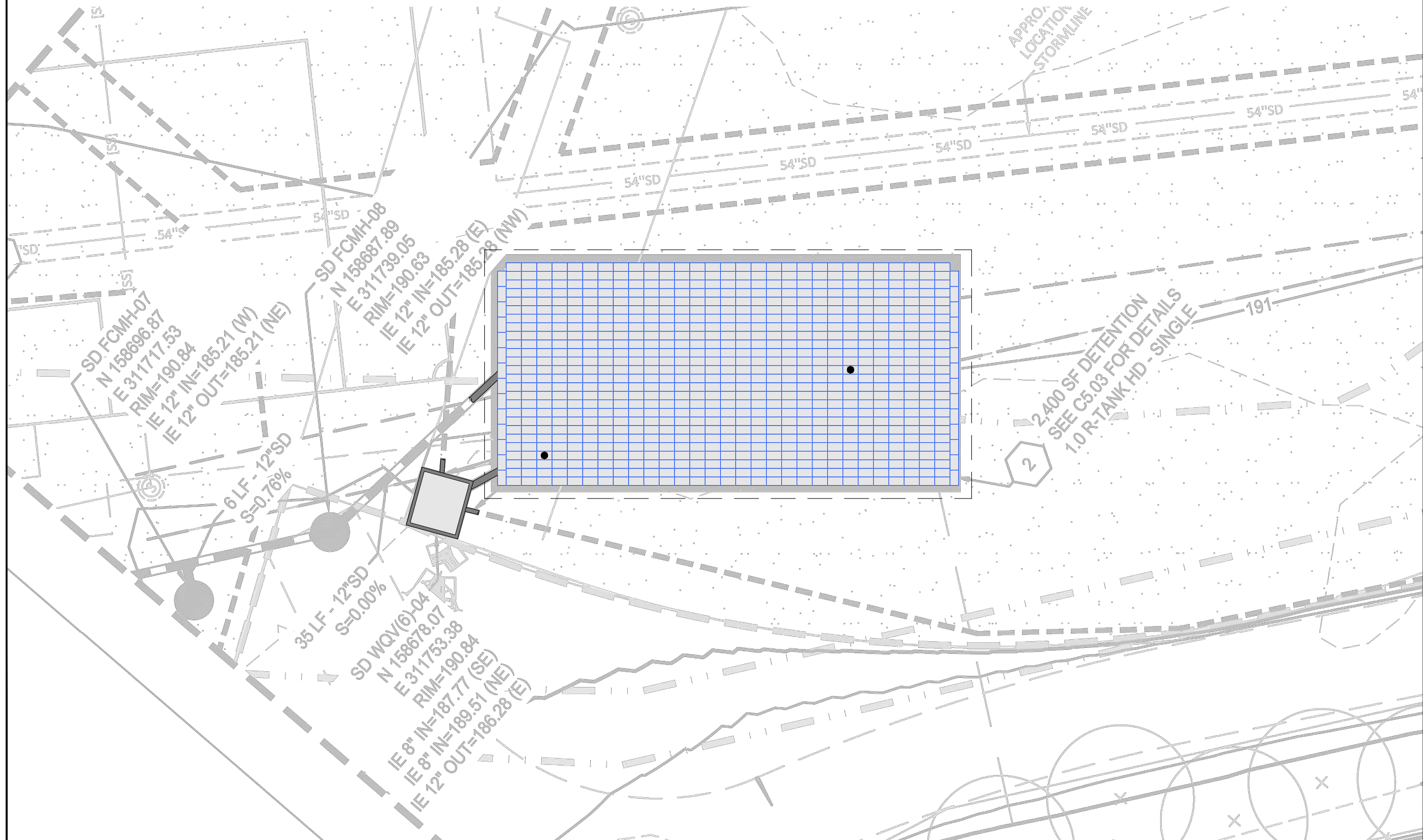BEAVERTON HIGH SCHOOL REBUILD
BEAVERTON, OR
SITE DESIGNATION: RT-W04

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SCALE 1" = 15'
0   15'   30'

SINGLE R-TANK^HD - ELEVATION

FINISHED SURFACE

MAX. ALLOW. FINAL GRADE=193.71
MIN. ALLOW. FINAL GRADE=187.72

COVER MIN. 12"
MAX. 6.99'

TOP OF TANK=186.72

17.32"

TANK INV=185.28

4"

STONE BASE INV=184.95

SINGLE R-TANK^HD - MODULE DETAIL

28.15" SIDE 17.32"
15.75" END 17.32"

(5) SMALL PLATES

28.15" TOP 15.75"

ISOMETRIC

MODULE DATA

GEOMETRY:
LENGTH = 28.15 IN. (715 MM)
WIDTH = 15.75 IN. (400 MM)
HEIGHT = 17.32 IN. (440 MM)
TANK VOLUME = 4.44 CF
STORAGE VOLUME = 4.22 CF
VOID INTERNAL VOLUME: 95%
VOID SURFACE AREA: 90%

LOAD RATING:
33.4 PSI, (MODULE ONLY)
HS20/HS25 - SEE SPEC FOR
COVER REQUIREMENTS
MATERIAL:
100% RECYCLED POLYPROPYLENE
SMALL PLATES REQUIRED:
5/SEGMENT, 5/MODULE
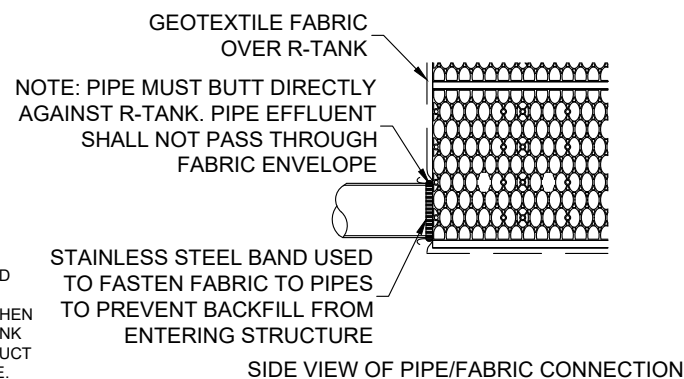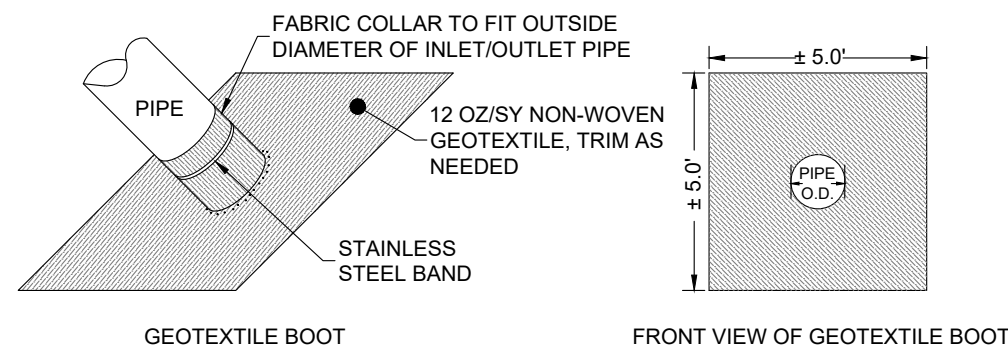
R-TANK^HD QUANTITIES

R-TANK^HD MODULE TYPE	SINGLE
# OF SINGLE R-TANKS	782
TOTAL SYSTEM STORAGE	5,071 CF
R-TANK STORAGE VOLUME	3,300 CF
STONE STORAGE VOLUME (40% VOID RATIO)	1,771 CF
STONE BED FOOTPRINT	2,846 SF
STONE QUANTITY	164 CY
TENCATE MIRAFI FW-402 WOVEN TANK WRAP	5,886 SF (654 SY)
TENCATE MIRAFI FW-402 WOVEN EXCAVATION WRAP	7,268 SF (808 SY)
12" MAINTENANCE PORTS	2
8" PIPE BOOTS	2
NOTE: STONE QUANTITY INCLUDES 12" OF COVER AND 4" OF BASE.	
NOTE: GEOTEXTILE / LINER QUANTITIES INCLUDE A 15% WASTE FACTOR.	

NOTE:
STORAGE VOLUME FROM 185.28 TO 186.28 = 2,462 CF

R-TANK^HD TYPICAL TANK INLET/OUTLET W/ GEOTEXTILE PIPE BOOT DETAIL

CUT AN "X" IN THE FABRIC ENVELOPE THAT IS SLIGHTLY
LARGER THAN THE PIPE. PULL THE FABRIC FLAPS AROUND
THE PIPE, AND SEAL WITH A STAINLESS STEEL BAND.

GEOTEXTILE FABRIC OVER R-TANK

INLET/OUTLET PIPE

"X" CUT IN THE FABRIC TO ALLOW PIPE/TANK INTERFACE

END VIEW OF PIPE/FABRIC CONNECTION

GEOTEXTILE FABRIC OVER R-TANK

NOTE: PIPE MUST BUTT DIRECTLY
AGAINST R-TANK. PIPE EFFLUENT
SHALL NOT PASS THROUGH
FABRIC ENVELOPE

AFTER TANK WRAP IS
SECURED TO PIPE, SLIDE
BOOT AGAINST R-TANK AND
SECURE WITH SECOND
STAINLESS STEEL BAND, THEN
ATTACH BOOT FLAP TO TANK
ENVELOPE FABRIC WITH DUCT
TAPE OR OTHER ADHESIVE.

STAINLESS STEEL BAND USED
TO FASTEN FABRIC TO PIPES
TO PREVENT BACKFILL FROM
ENTERING STRUCTURE

SIDE VIEW OF PIPE/FABRIC CONNECTION

FABRIC COLLAR TO FIT OUTSIDE
DIAMETER OF INLET/OUTLET PIPE

PIPE

12 OZ/SY NON-WOVEN
GEOTEXTILE, TRIM AS
NEEDED

STAINLESS STEEL BAND

GEOTEXTILE BOOT

± 5.0'
± 5.0'
PIPE O.D.

FRONT VIEW OF GEOTEXTILE BOOT

GEOTEXTILE FABRIC OVER R-TANK

2'-0"
PIPE O.D.

STAINLESS STEEL BAND

SIDE VIEW OF GEOTEXTILE BOOT

R-TANK^HD SYSTEM DETAILS
BEAVERTON HIGH SCHOOL REBUILD
BEAVERTON, OR
SITE DESIGNATION: RT-W04

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EDQ
DATE
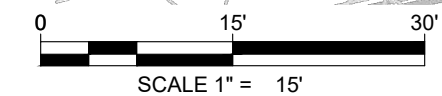06/24/2023
ACF WEST PROJECT NUMBER
23-017OR
SHEET NO.

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NOTES:
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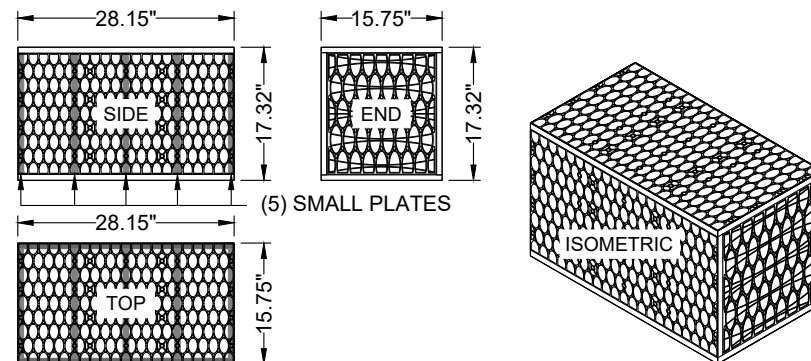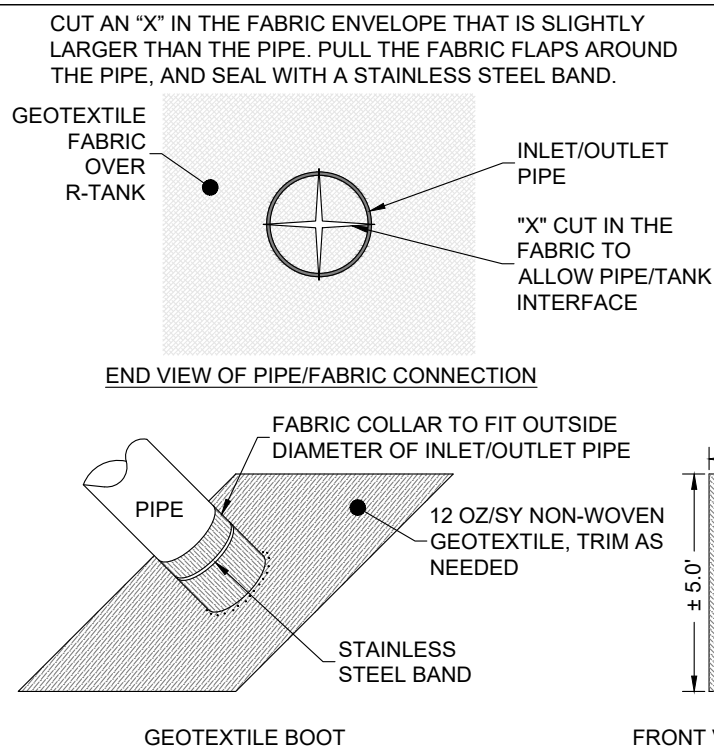
ENGINEER OF RECORD TO REVIEW, APPROVE AND ENDORSE FINAL SITE SPECIFIC DESIGN.

FOR ADDITIONAL INFORMATION PLEASE CONTACT:
ACF WEST, 1-800-423-4567, www.acfwest.com

1,600 SF DETENTION
SEE C5.03 FOR DETAILS
1.0 R-TANK HD - SINGLE

MATCHLINE SEE SHEET C4.11

SD WY
N 15853
E 312017.
RIM=190.4
IE 8" IN=189.

16 LF - 12"SD S=0.00%
16 LF - 12"SD S=0.00%
7 LF - 12"SD S=0.00%
8 LF - 12"SD S=0.00%
32 LF - 12"SD S=0.00%

54"SD
54"SD

SD FCMH-09
N 158532.48
E 311996.54
RIM=192.36
IE 12" IN=186.75 (N)
IE 12" OUT=186.75 (SW)

SD FCMH-10
N 158513.44
E 311981.80
RIM=192.22
IE 12" IN=186.71 (SE)
IE 12" OUT=186.66 (NE)

SD WQCB(3)-07
N 158491.70
E 311979.94
RIM=191.86
IE 8" IN=189.36 (NE)
IE 8" IN=189.36 (SW)
IE 12" OUT=187.71 (SE)

R-TANK^HD SYSTEM OVERLAY
SCALE: 1" = 15'

R-TANK^HD SYSTEM OVERLAY
BEAVERTON HIGH SCHOOL REBUILD
BEAVERTON, OR
SITE DESIGNATION: RT-W05

0   15'   30'
SCALE 1" = 15'

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SINGLE R-TANK^HD - ELEVATION

FINISHED SURFACE

MAX. ALLOW. FINAL GRADE=195.14
MIN. ALLOW. FINAL GRADE=189.15

COVER MIN. 12"
MAX. 6.99'

TOP OF TANK=188.15

17.32"

TANK INV=186.71

4"

STONE BASE INV=186.38

SINGLE R-TANK^HD - MODULE DETAIL

28.15"   15.75"

SIDE   END

17.32"   17.32"

(5) SMALL PLATES

ISOMETRIC

28.15"

TOP

15.75"

MODULE DATA

GEOMETRY:
LENGTH = 28.15 IN. (715 MM)
WIDTH = 15.75 IN. (400 MM)
HEIGHT = 17.32 IN. (440 MM)
TANK VOLUME = 4.44 CF
STORAGE VOLUME = 4.22 CF
VOID INTERNAL VOLUME: 95%
VOID SURFACE AREA: 90%

LOAD RATING:
33.4 PSI, (MODULE ONLY)
HS20/HS25 - SEE SPEC FOR COVER REQUIREMENTS

MATERIAL:
100% RECYCLED POLYPROPYLENE

SMALL PLATES REQUIRED:
5/SEGMENT, 5/MODULE

R-TANK^HD QUANTITIES

R-TANK^HD MODULE TYPE	SINGLE
# OF SINGLE R-TANKS	506
TOTAL SYSTEM STORAGE	3,347 CF
R-TANK STORAGE VOLUME	2,135 CF
STONE STORAGE VOLUME (40% VOID RATIO)	1,212 CF
STONE BED FOOTPRINT	1,901 SF
STONE QUANTITY	112 CY
TENCATE MIRAFI FW-402 WOVEN TANK WRAP	3,854 SF (428 SY)
TENCATE MIRAFI FW-402 WOVEN EXCAVATION WRAP	4,943 SF (549 SY)
12" MAINTENANCE PORTS	2
8" PIPE BOOTS	2
NOTE: STONE QUANTITY INCLUDES 12" OF COVER AND 4" OF BASE.	
NOTE: GEOTEXTILE / LINER QUANTITIES INCLUDE A 15% WASTE FACTOR.	

NOTE:
STORAGE VOLUME FROM 186.71 TO 187.71 = 1,617 CF

R-TANK^HD TYPICAL TANK INLET/OUTLET W/ GEOTEXTILE PIPE BOOT DETAIL

CUT AN "X" IN THE FABRIC ENVELOPE THAT IS SLIGHTLY LARGER THAN THE PIPE. PULL THE FABRIC FLAPS AROUND THE PIPE, AND SEAL WITH A STAINLESS STEEL BAND.

GEOTEXTILE FABRIC OVER R-TANK

INLET/OUTLET PIPE

"X" CUT IN THE FABRIC TO ALLOW PIPE/TANK INTERFACE

END VIEW OF PIPE/FABRIC CONNECTION

AFTER TANK WRAP IS SECURED TO PIPE, SLIDE BOOT AGAINST R-TANK AND SECURE WITH SECOND STAINLESS STEEL BAND, THEN ATTACH BOOT FLAP TO TANK ENVELOPE FABRIC WITH DUCT TAPE OR OTHER ADHESIVE.

GEOTEXTILE FABRIC OVER R-TANK

NOTE: PIPE MUST BUTT DIRECTLY AGAINST R-TANK. PIPE EFFLUENT SHALL NOT PASS THROUGH FABRIC ENVELOPE

STAINLESS STEEL BAND USED TO FASTEN FABRIC TO PIPES TO PREVENT BACKFILL FROM ENTERING STRUCTURE

SIDE VIEW OF PIPE/FABRIC CONNECTION

FABRIC COLLAR TO FIT OUTSIDE DIAMETER OF INLET/OUTLET PIPE

PIPE

12 OZ/SY NON-WOVEN GEOTEXTILE, TRIM AS NEEDED

STAINLESS STEEL BAND

GEOTEXTILE BOOT

± 5.0'
± 5.0'
PIPE O.D.

FRONT VIEW OF GEOTEXTILE BOOT

GEOTEXTILE FABRIC OVER R-TANK

PIPE O.D.
2'-0"

STAINLESS STEEL BAND

SIDE VIEW OF GEOTEXTILE BOOT
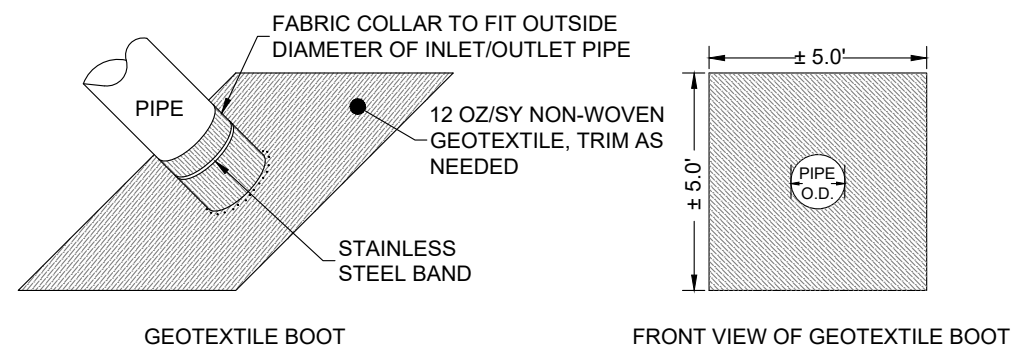
R-TANK^HD SYSTEM DETAILS
BEAVERTON HIGH SCHOOL REBUILD
BEAVERTON, OR
SITE DESIGNATION: RT-W05

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DATE
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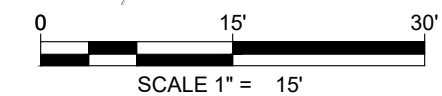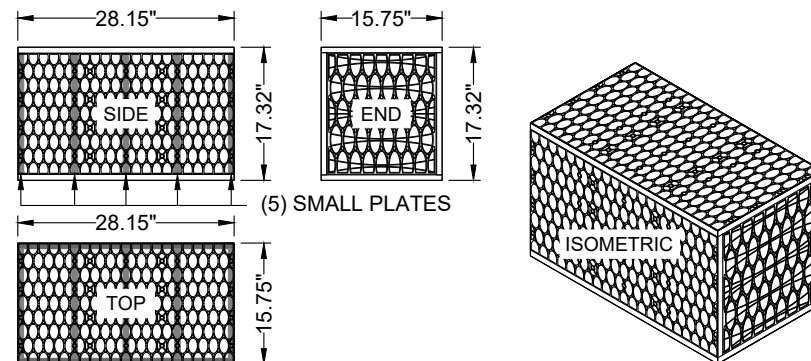**NOTES:**
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ENGINEER OF RECORD TO REVIEW, APPROVE AND ENDORSE FINAL SITE SPECIFIC DESIGN.

FOR ADDITIONAL INFORMATION PLEASE CONTACT:
ACF WEST, 1-800-423-4567, www.acfwest.com

MATCHLINE SEE SHEET C1.02

R-TANK^HD SYSTEM OVERLAY
SCALE: 1" = 25'

R-TANK^HD SYSTEM OVERLAY
BEAVERTON HIGH SCHOOL REBUILD
BEAVERTON, OR
SITE DESIGNATION: RT-W06

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SCALE 1" = 25'

0   25'   50'

SINGLE R-TANK^HD - ELEVATION

FINISHED SURFACE

MAX. ALLOW. FINAL GRADE=199.70
MIN. ALLOW. FINAL GRADE=194.38

COVER MIN. 20" MAX. 6.99'

GEOGRID ELEV=193.71

TOP OF TANK=192.71

TANK INV=191.27

STONE BASE INV=191.02

12"
17.32"
3"

SINGLE R-TANK^HD - MODULE DETAIL

28.15"
15.75"
17.32"
17.32"

SIDE
END

(5) SMALL PLATES

28.15"
15.75"

TOP

ISOMETRIC

MODULE DATA

GEOMETRY:
LENGTH = 28.15 IN. (715 MM)
WIDTH = 15.75 IN. (400 MM)
HEIGHT = 17.32 IN. (440 MM)
TANK VOLUME = 4.44 CF
STORAGE VOLUME = 4.22 CF
VOID INTERNAL VOLUME: 95%
VOID SURFACE AREA: 90%

LOAD RATING:
33.4 PSI, (MODULE ONLY)
HS20/HS25 - SEE SPEC FOR
COVER REQUIREMENTS

MATERIAL:
100% RECYCLED POLYPROPYLENE

SMALL PLATES REQUIRED:
5/SEGMENT, 5/MODULE

R-TANK^HD QUANTITIES

R-TANK^HD MODULE TYPE	SINGLE
# OF SINGLE R-TANKS	799
TOTAL SYSTEM STORAGE	4,833 CF
R-TANK STORAGE VOLUME	3,372 CF
STONE STORAGE VOLUME (40% VOID RATIO)	1,461 CF
STONE BED FOOTPRINT	2,949 SF
STONE QUANTITY	163 CY
TENCATE MIRAFI 180N NON-WOVEN TANK WRAP	3,217 SF (357 SY)
30 MIL. GEOMEMBRANE LINER EXCAVATION WRAP	7,556 SF (840 SY)
TENCATE MIRAFI 180N NON-WOVEN LINER PROTECTION	15,112 SF (1,679 SY)
NAUE SECUGRID 30/30 GEOGRID	4,295 SF (477 SY)
12" MAINTENANCE PORTS	2
8" PIPE BOOTS	3
NOTE: STONE QUANTITY INCLUDES 12" OF COVER AND 3" OF BASE.	
NOTE: GEOTEXTILE / LINER QUANTITIES INCLUDE A 15% WASTE FACTOR.	

NOTE:
1. STORAGE VOLUME FROM ELEVATION 191.27 TO 192.27 = 2,532 CF
2. BASE STONE STORAGE NOT INCLUDED IN VOLUMES.

R-TANK^HD TYPICAL TANK INLET/OUTLET W/ GEOTEXTILE PIPE BOOT DETAIL

CUT AN "X" IN THE FABRIC ENVELOPE THAT IS SLIGHTLY LARGER THAN THE PIPE. PULL THE FABRIC FLAPS AROUND THE PIPE, AND SEAL WITH A STAINLESS STEEL BAND.

GEOTEXTILE FABRIC OVER R-TANK

INLET/OUTLET PIPE

"X" CUT IN THE FABRIC TO ALLOW PIPE/TANK INTERFACE

END VIEW OF PIPE/FABRIC CONNECTION

AFTER TANK WRAP IS SECURED TO PIPE, SLIDE BOOT AGAINST R-TANK AND SECURE WITH SECOND STAINLESS STEEL BAND, THEN ATTACH BOOT FLAP TO TANK ENVELOPE FABRIC WITH DUCT TAPE OR OTHER ADHESIVE.

GEOTEXTILE FABRIC OVER R-TANK

NOTE: PIPE MUST BUTT DIRECTLY AGAINST R-TANK. PIPE EFFLUENT SHALL NOT PASS THROUGH FABRIC ENVELOPE

STAINLESS STEEL BAND USED TO FASTEN FABRIC TO PIPES TO PREVENT BACKFILL FROM ENTERING STRUCTURE

SIDE VIEW OF PIPE/FABRIC CONNECTION

FABRIC COLLAR TO FIT OUTSIDE DIAMETER OF INLET/OUTLET PIPE

PIPE

12 OZ/SY NON-WOVEN GEOTEXTILE, TRIM AS NEEDED

STAINLESS STEEL BAND

GEOTEXTILE BOOT

± 5.0'
± 5.0'

PIPE O.D.

FRONT VIEW OF GEOTEXTILE BOOT

GEOTEXTILE FABRIC OVER R-TANK

PIPE O.D.

2'-0"

STAINLESS STEEL BAND

SIDE VIEW OF GEOTEXTILE BOOT
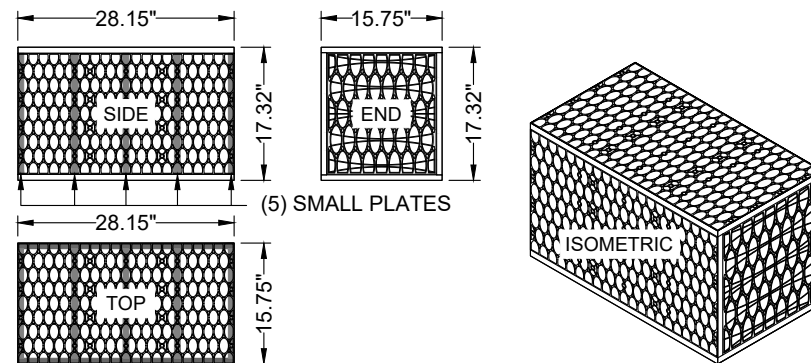
R-TANK^HD SYSTEM DETAILS
BEAVERTON HIGH SCHOOL REBUILD
BEAVERTON, OR
SITE DESIGNATION: RT-W06

DRAWN BY
EDQ
DATE
06/24/2023
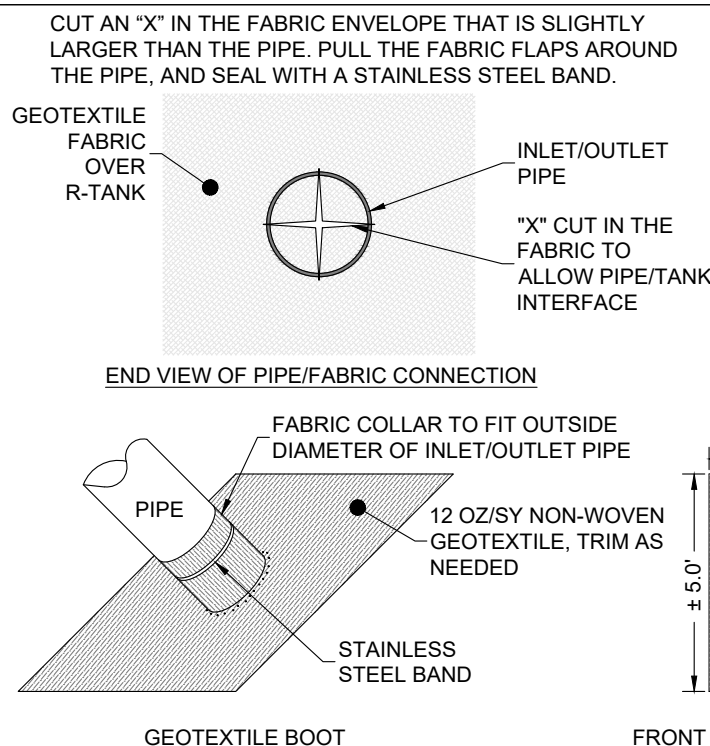ACF WEST PROJECT NUMBER
23-017OR
SHEET NO.

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Appendix C
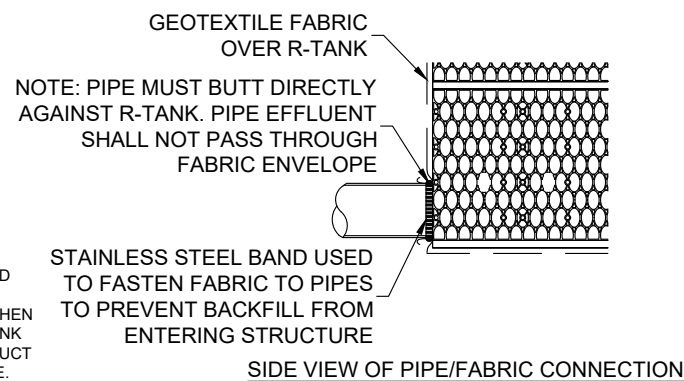
References and Codes

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Chapter 7   Hydrologic Soil Groups

630.0700   Introduction

This chapter defines four hydrologic soil groups, or HSGs, that, along with land use, management practices, and hydrologic conditions, determine a soil's associated runoff curve number (NEH630.09). Runoff curve numbers are used to estimate direct runoff from rainfall (NEH630.10).

A map unit is a collection of areas defined and named the same in terms of their soil components or miscellaneous areas or both (NSSH 627.03). Soil scientists assign map unit components to hydrologic soil groups. Map unit components assigned to a specific hydrologic soil group have similar physical and runoff characteristics. Soils in the United States, its territories, and Puerto Rico have been assigned to hydrologic soil groups. The assigned groups can be found by consulting the Natural Resources Conservation Service's (NRCS) Field Office Technical Guide; published soil survey data bases; the NRCS Soil Data Mart Web site (*http://soildatamart.nrcs.usda.gov/*); and/or the Web Soil Survey Web site (*http://websoilsurvey.nrcs.usda.gov/*).

The NRCS State soil scientist should be contacted if a soil survey does not exist for a given area or where the soils within a watershed have not been assigned to hydrologic groups.

630.0701   Hydrologic soil groups

Soils were originally assigned to hydrologic soil groups based on measured rainfall, runoff, and infiltrometer data (Musgrave 1955). Since the initial work was done to establish these groupings, assignment of soils to hydrologic soil groups has been based on the judgment of soil scientists. Assignments are made based on comparison of the characteristics of unclassified soil profiles with profiles of soils already placed into hydrologic soil groups. Most of the groupings are based on the premise that soils found within a climatic region that are similar in depth to a restrictive layer or water table, transmission rate of water, texture, structure, and degree of swelling when saturated, will have similar runoff responses. The classes are based on the following factors:

- intake and transmission of water under the conditions of maximum yearly wetness (thoroughly wet)
- soil not frozen
- bare soil surface
- maximum swelling of expansive clays

The slope of the soil surface is not considered when assigning hydrologic soil groups.

In its simplest form, hydrologic soil group is determined by the water transmitting soil layer with the lowest saturated hydraulic conductivity and depth to any layer that is more or less water impermeable (such as a fragipan or duripan) or depth to a water table (if present). The least transmissive layer can be any soil horizon that transmits water at a slower rate relative to those horizons above or below it. For example, a layer having a saturated hydraulic conductivity of 9.0 micrometers per second (1.3 inches per hour) is the least transmissive layer in a soil if the layers above and below it have a saturated hydraulic conductivity of 23 micrometers per second (3.3 inches per hour).

Water impermeable soil layers are among those types of layers recorded in the component restriction table of the National Soil Information System (NASIS) database. The saturated hydraulic conductivity of an impermeable or nearly impermeable layer may range

from essentially 0 micrometers per second (0 inches per hour) to 0.9 micrometers per second (0.1 inches per hour). For simplicity, either case is considered impermeable for hydrologic soil group purposes. In some cases, saturated hydraulic conductivity (a quantitatively measured characteristic) data are not always readily available or obtainable. In these situations, other soil properties such as texture, compaction (bulk density), strength of soil structure, clay mineralogy, and organic matter are used to estimate water movement. Table 7–1 relates saturated hydraulic conductivity to hydrologic soil group.

The four hydrologic soil groups (HSGs) are described as:

Group A—Soils in this group have low runoff potential when thoroughly wet. Water is transmitted freely through the soil. Group A soils typically have less than 10 percent clay and more than 90 percent sand or gravel and have gravel or sand textures. Some soils having loamy sand, sandy loam, loam or silt loam textures may be placed in this group if they are well aggregated, of low bulk density, or contain greater than 35 percent rock fragments.

The limits on the diagnostic physical characteristics of group A are as follows. The saturated hydraulic conductivity of all soil layers exceeds 40.0 micrometers per second (5.67 inches per hour). The depth to any water impermeable layer is greater than 50 centimeters [20 inches]. The depth to the water table is greater than 60 centimeters [24 inches]. Soils that are deeper than 100 centimeters [40 inches] to a water impermeable layer and a water table are in group A if the saturated hydraulic conductivity of all soil layers within 100 centimeters [40 inches] of the surface exceeds 10 micrometers per second (1.42 inches per hour).

Group B—Soils in this group have moderately low runoff potential when thoroughly wet. Water transmission through the soil is unimpeded. Group B soils typically have between 10 percent and 20 percent clay and 50 percent to 90 percent sand and have loamy sand or sandy loam textures. Some soils having loam, silt loam, silt, or sandy clay loam textures may be placed in this group if they are well aggregated, of low bulk density, or contain greater than 35 percent rock fragments.

The limits on the diagnostic physical characteristics of group B are as follows. The saturated hydraulic

conductivity in the least transmissive layer between the surface and 50 centimeters [20 inches] ranges from 10.0 micrometers per second (1.42 inches per hour) to 40.0 micrometers per second (5.67 inches per hour). The depth to any water impermeable layer is greater than 50 centimeters [20 inches]. The depth to the water table is greater than 60 centimeters [24 inches]. Soils that are deeper than 100 centimeters [40 inches] to a water impermeable layer and a water table are in group B if the saturated hydraulic conductivity of all soil layers within 100 centimeters [40 inches] of the surface exceeds 4.0 micrometers per second (0.57 inches per hour) but is less than 10.0 micrometers per second (1.42 inches per hour).

Group C—Soils in this group have moderately high runoff potential when thoroughly wet. Water transmission through the soil is somewhat restricted. Group C soils typically have between 20 percent and 40 percent clay and less than 50 percent sand and have loam, silt loam, sandy clay loam, clay loam, and silty clay loam textures. Some soils having clay, silty clay, or sandy clay textures may be placed in this group if they are well aggregated, of low bulk density, or contain greater than 35 percent rock fragments.

The limits on the diagnostic physical characteristics of group C are as follows. The saturated hydraulic conductivity in the least transmissive layer between the surface and 50 centimeters [20 inches] is between 1.0 micrometers per second (0.14 inches per hour) and 10.0 micrometers per second (1.42 inches per hour). The depth to any water impermeable layer is greater than 50 centimeters [20 inches]. The depth to the water table is greater than 60 centimeters [24 inches]. Soils that are deeper than 100 centimeters [40 inches] to a restriction and a water table are in group C if the saturated hydraulic conductivity of all soil layers within 100 centimeters [40 inches] of the surface exceeds 0.40 micrometers per second (0.06 inches per hour) but is less than 4.0 micrometers per second (0.57 inches per hour).

Group D—Soils in this group have high runoff potential when thoroughly wet. Water movement through the soil is restricted or very restricted. Group D soils typically have greater than 40 percent clay, less than 50 percent sand, and have clayey textures. In some areas, they also have high shrink-swell potential. All soils with a depth to a water impermeable layer less than 50 centimeters [20 inches] and all soils with a water table

within 60 centimeters [24 inches] of the surface are in this group, although some may have a dual classification, as described in the next section, if they can be adequately drained.

The limits on the physical diagnostic characteristics of group D are as follows. For soils with a water impermeable layer at a depth between 50 centimeters and 100 centimeters [20 and 40 inches], the saturated hydraulic conductivity in the least transmissive soil layer is less than or equal to 1.0 micrometers per second (0.14 inches per hour). For soils that are deeper than 100 centimeters [40 inches] to a restriction or water table, the saturated hydraulic conductivity of all soil layers within 100 centimeters [40 inches] of the surface is less than or equal to 0.40 micrometers per second (0.06 inches per hour).

Dual hydrologic soil groups—Certain wet soils are placed in group D based solely on the presence of a water table within 60 centimeters [24 inches] of the surface even though the saturated hydraulic conductivity may be favorable for water transmission. If these soils can be adequately drained, then they are assigned to dual hydrologic soil groups (A/D, B/D, and C/D) based on their saturated hydraulic conductivity and the water table depth when drained. The first letter applies to the drained condition and the second to the undrained condition. For the purpose of hydrologic soil group, adequately drained means that the seasonal high water table is kept at least 60 centimeters [24 inches] below the surface in a soil where it would be higher in a natural state.

Matrix of hydrologic soil group assignment criteria—The decision matrix in table 7–1 can be used to determine a soil's hydrologic soil group. If saturated hydraulic conductivity data are available and deemed to be reliable, then these data, along with water table depth information, should be used to place the soil into the appropriate hydrologic soil group. If these data are not available, the hydrologic soil group is determined by observing the properties of the soil in the field. Factors such as texture, compaction (bulk density), strength of soil structure, clay mineralogy, and organic matter are considered in estimating the hydraulic conductivity of each layer in the soil profile. The depth and hydraulic conductivity of any water impermeable layer and the depth to any high water table are used to determine correct hydrologic soil group for the soil. The property that is most limiting to water

movement generally determines the soil's hydrologic group. In anomalous situations, when adjustments to hydrologic soil group become necessary, they shall be made by the NRCS State soil scientist in consultation with the State conservation engineer.

Table 7–1 Criteria for assignment of hydrologic soil group (HSG)

Depth to water impermeable layer [1]	Depth to high water table [2]	K_{sat} of least transmissive layer in depth range	K_{sat} depth range	HSG [3]
<50 cm [<20 in]	—	—	—	D
50 to 100 cm [20 to 40 in]	<60 cm [<24 in]	>40.0 µm/s (>5.67 in/h)	0 to 60 cm [0 to 24 in]	A/D
		>10.0 to ≤40.0 µm/s (>1.42 to ≤5.67 in/h)	0 to 60 cm [0 to 24 in]	B/D
		>1.0 to ≤10.0 µm/s (>0.14 to ≤1.42 in/h)	0 to 60 cm [0 to 24 in]	C/D
		≤1.0 µm/s (≤0.14 in/h)	0 to 60 cm [0 to 24 in]	D
	≥60 cm [≥24 in]	>40.0 µm/s (>5.67 in/h)	0 to 50 cm [0 to 20 in]	A
		>10.0 to ≤40.0 µm/s (>1.42 to ≤5.67 in/h)	0 to 50 cm [0 to 20 in]	B
		>1.0 to ≤10.0 µm/s (>0.14 to ≤1.42 in/h)	0 to 50 cm [0 to 20 in]	C
		≤1.0 µm/s (≤0.14 in/h)	0 to 50 cm [0 to 20 in]	D
>100 cm [>40 in]	<60 cm [<24 in]	>10.0 µm/s (>1.42 in/h)	0 to 100 cm [0 to 40 in]	A/D
		>4.0 to ≤10.0 µm/s (>0.57 to ≤1.42 in/h)	0 to 100 cm [0 to 40 in]	B/D
		>0.40 to ≤4.0 µm/s (>0.06 to ≤0.57 in/h)	0 to 100 cm [0 to 40 in]	C/D
		≤0.40 µm/s (≤0.06 in/h)	0 to 100 cm [0 to 40 in]	D
	60 to 100 cm [24 to 40 in]	>40.0 µm/s (>5.67 in/h)	0 to 50 cm [0 to 20 in]	A
		>10.0 to ≤40.0 µm/s (>1.42 to ≤5.67 in/h)	0 to 50 cm [0 to 20 in]	B
		>1.0 to ≤10.0 µm/s (>0.14 to ≤1.42 in/h)	0 to 50 cm [0 to 20 in]	C
		≤1.0 µm/s (≤0.14 in/h)	0 to 50 cm [0 to 20 in]	D
	>100 cm [>40 in]	>10.0 µm/s (>1.42 in/h)	0 to 100 cm [0 to 40 in]	A
		>4.0 to ≤ 10.0 µm/s (>0.57 to ≤1.42 in/h)	0 to 100 cm [0 to 40 in]	B
		>0.40 to ≤4.0 µm/s (>0.06 to ≤0.57 in/h)	0 to 100 cm [0 to 40 in]	C
		≤0.40 µm/s (≤0.06 in/h)	0 to 100 cm [0 to 40 in]	D

1/ An impermeable layer has a K_{sat} less than 0.01 µm/s [0.0014 in/h] or a component restriction of fragipan; duripan; petrocalcic; orstein; petrogypsic; cemented horizon; densic material; placic; bedrock, paralithic; bedrock, lithic; bedrock, densic; or permafrost.

2/ High water table during any month during the year.

3/ Dual HSG classes are applied only for wet soils (water table less than 60 cm [24 in]). If these soils can be drained, a less restrictive HSG can be assigned, depending on the K_{sat}.

Table 2-2c   Runoff curve numbers for other agricultural lands [1]

Cover description		Curve numbers for hydrologic soil group			
Cover type	Hydrologic condition	A	B	C	D
Pasture, grassland, or range—continuous forage for grazing. [2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow—continuous grass, protected from grazing and generally mowed for hay.	—	30	58	71	78
Brush—brush-weed-grass mixture with brush the major element. [3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30 [4]	48	65	73
Woods—grass combination (orchard or tree farm). [5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods. [6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30 [4]	55	70	77
Farmsteads—buildings, lanes, driveways, and surrounding lots.	—	59	74	82	86

Pre-Development

[1]   Average runoff condition, and $I_a = 0.2S$.
[2]   *Poor:*   <50%) ground cover or heavily grazed with no mulch.
      Fair: 50 to 75% ground cover and not heavily grazed.
      Good: > 75% ground cover and lightly or only occasionally grazed.
[3]   *Poor*:   <50% ground cover.
      Fair: 50 to 75% ground cover.
      Good: >75% ground cover.
[4]   Actual curve number is less than 30; use CN = 30 for runoff computations.
[5]   CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.
[6]   *Poor:*   Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
      Fair: Woods are grazed but not burned, and some forest litter covers the soil.
      Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

Table 2-2a   Runoff curve numbers for urban areas [1/]

	Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition		Average percent impervious area [2/]	A	B	C	D

Fully developed urban areas (vegetation established) **Post-Development Pervious**

Open space (lawns, parks, golf courses, cemeteries, etc.) [3/]:

Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80

Impervious areas:

Paved parking lots, roofs, driveways, etc.
(excluding right-of-way) 98   98   98   98

Post-Development Impervious

Streets and roads:

Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way).........................		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89

Western desert urban areas:

Natural desert landscaping (pervious areas only) [4/]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96

Urban districts:

Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93

Residential districts by average lot size:

1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres...	12	46	65	77	82

Developing urban areas

Newly graded areas
(pervious areas only, no vegetation) [5/]... 77   86   91   94

Idle lands (CN's are determined using cover types
    similar to those in table 2-2c).

[1] Average runoff condition, and $I_a = 0.2S$.

[2] The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.

[3] CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.

[4] Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.

[5] Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4 based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

DESIGN STORM DISTRIBUTION CHART

THE FOLLOWING TABLE CONTAINS THE NRCS TYPE 1A PRECIPITATION DIDTRIBUTION. THE TABLE IS FRM
THE "SUB BASIN HYDROLOGIC MODLELING CRITERIA" BY KRAMER, CHIN, & MAYO INC., 1991

HOUR	PERCENT RAINFALL INCREMENTAL	PERCENT RAINFALL CUMULATIVE	RAINFALL DEPTH (INCHES) 2 YEAR 2.50	5 YEAR 3.10	10 YEAR 3.45	25 YEAR 3.90	50 YEAR 4.20	100 YEAR 4.50
1	2.40	2.40	0.06	0.07	0.08	0.09	0.10	0.11
2	2.60	5.00	0.07	0.08	0.09	0.10	0.11	0.12
3	3.20	8.20	0.08	0.10	0.11	0.12	0.13	0.14
4	3.80	12.00	0.10	0.12	0.13	0.15	0.16	0.17
5	4.44	16.44	0.11	0.14	0.15	0.17	0.19	0.20
6	5.18	21.62	0.13	0.16	0.18	0.20	0.22	0.23
7	6.48	28.10	0.16	0.20	0.22	0.25	0.27	0.29
8	16.44	44.54	0.41	0.51	0.57	0.64	0.69	0.74
9	7.58	52.12	0.19	0.23	0.26	0.30	0.32	0.34
10	5.28	57.40	0.13	0.16	0.18	0.21	0.22	0.24
11	4.96	62.36	0.12	0.15	0.17	0.19	0.21	0.22
12	4.32	66.68	0.11	0.13	0.15	0.17	0.18	0.19
13	4.02	70.70	0.10	0.12	0.14	0.16	0.17	0.18
14	3.42	74.12	0.09	0.11	0.12	0.13	0.14	0.15
15	3.28	77.40	0.08	0.10	0.11	0.13	0.14	0.15
16	3.00	80.40	0.08	0.09	0.10	0.12	0.13	0.14
17	2.80	83.20	0.07	0.09	0.10	0.11	0.12	0.13
18	2.40	85.60	0.06	0.07	0.08	0.09	0.10	0.11
19	2.40	88.00	0.06	0.07	0.08	0.09	0.10	0.11
20	2.40	90.40	0.06	0.07	0.08	0.09	0.10	0.11
21	2.40	92.80	0.06	0.07	0.08	0.09	0.10	0.11
22	2.40	95.20	0.06	0.07	0.08	0.09	0.10	0.11
23	2.40	97.60	0.06	0.07	0.08	0.09	0.10	0.11
24	2.40	100.00	0.06	0.07	0.08	0.09	0.10	0.11

DESIGN STORM
DISTRIBUTION CHART

REVISED 12-06

CleanWater Services
Our commitment is clear.

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 1:20,000.

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: Washington County, Oregon
Survey Area Data: Version 22, Sep 14, 2022

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

Date(s) aerial images were photographed: Apr 16, 2021—Apr 18, 2021

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.

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Conservation Service

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3/8/2023
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Hydrologic Soil Group

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
1	Aloha silt loam	C/D	0.0	0.2%
15	Dayton silt loam	D	4.1	20.6%
45A	Woodburn silt loam, 0 to 3 percent slopes	C	15.4	77.4%
45B	Woodburn silt loam, 3 to 7 percent slopes	C	0.3	1.7%
Totals for Area of Interest			**19.9**	**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.

45A

15

45B

45A

Sw 2nd St

Sw Fairmount Dr

Sw Erickson Ave

BEAVERS

Soil Map may not be valid at this scale.

122° 48' 57" W

45° 29' 11" N

45° 29' 3" N

122° 48' 39" W

514390 514430 514470 514510 514550 514590 514630 514670 514710 514750

5037020 5036980 5036940 5036900 5036860 5036820 5036780

N

Map Scale: 1:1,770 if printed on A landscape (11" x 8.5") sheet.

Meters
0   25   50   100   150

Feet
0   50   100   200   300

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 10N WGS84

USDA **Natural Resources**
Conservation Service

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National Cooperative Soil Survey

4/19/2023
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Hydrologic Soil Group

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
15	Dayton silt loam	D	3.5	43.2%
45A	Woodburn silt loam, 0 to 3 percent slopes	C	2.5	30.6%
45B	Woodburn silt loam, 3 to 7 percent slopes	C	2.1	26.3%
Totals for Area of Interest			**8.1**	**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.
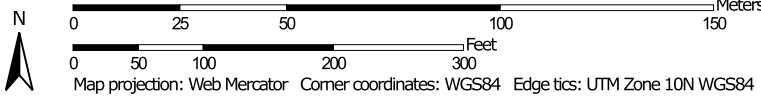
If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.

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Composite Curve Number Calculation
Appendix C-5

kpff

EAST Basins 1-6

Map Unit Symbol	Rating	CN	Area (ac)
45A	C	76	15.4
15	D	82	4.1
Composite CN:			77

WEST Basins 1-7

Map Unit Symbol	Rating	CN	Area (ac)
45A	C	76	2.5
45B	C	76	2.1
15	D	82	3.5
Composite CN:			79

*Note that modified impervious area (WEST Basin 6 & EAST Basins 3,4,5) shall have a pre-developed CN of 75

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Appendix D

Geotechnical Report

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Geotechnical Investigation and Site-Specific Seismic-Hazard Evaluation Beaverton High School Replacement

Beaverton, Oregon

March 24, 2023

Prepared for

Beaverton School District
16550 SW Merlo Road
Beaverton, OR 97003

Prepared by

GRI

16520 SW Upper Boones Ferry Road, Suite 100
Tigard, OR 97224-7661
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TABLES

FIGURES

APPENDICES

1   INTRODUCTION

As requested, GRI completed a geotechnical site evaluation for the proposed Beaverton High School replacement located in Beaverton, Oregon. The Vicinity Map, Figure 1, shows the general location of the site. The purpose of the investigation was to evaluate subsurface conditions at the site and develop geotechnical recommendations for use in the design and construction of the proposed high school replacement. The investigation included a review of available existing geotechnical information for the site and surrounding areas, subsurface explorations, laboratory testing, and engineering analysis. This report describes the work accomplished and provides conclusions and recommendations for use in the design and construction of the proposed improvements.

2   BACKGROUND

GRI reviewed the following geotechnical reports as a part of our geotechnical investigation:

"Draft Geotechnical Investigation for Beaverton High School Turf Field," prepared by Foundation Engineering for Beaverton School District, dated February 20, 2013.

"Geotechnical Engineering Services for Aloha, Beaverton and Sunset High Schools, New Bleachers," prepared by Geocon Northwest for Beaverton School District, dated April 3, 2012.

In addition, GRI completed a preliminary geotechnical evaluation which is summarized in the following report:

"Preliminary Geotechnical Investigation Beaverton High School Replacement" prepared by GRI for Beaverton School District, dated February 4, 2022

3   PROJECT DESCRIPTION

We understand the Beaverton School District proposes to design and construct new buildings at the existing high school campus to ultimately replace the existing high school building and associated structures, with the exception of the cafeteria. We understand the new building may be three stories. The proposed building will be in the northern margins of the site within the existing athletic fields and Merle Davies building areas.

4   SITE DESCRIPTION

4.1   Topography

The Beaverton High School campus is bounded by SW Farmington Road to the north, SW Stott Avenue to the east, SW Fifth Street to the south, and residential development to the west. Buildings associated with the high school occupy the central portion of the property. The Merle Davies building is located in the northeast property corner. Athletic fields occupy the northern, southern, and western property margins.

4.2 Geology

Published geologic mapping and our personnel's past experience on the project site indicate the site is mantled with Missoula flood deposits, locally referred to in the project area as the Willamette Silt Formation (Ma et al., 2009). In general, Willamette Silt is composed of beds and lenses of silt and sand. Stratification within this formation commonly consists of 4- to 6-inch-thick beds, although in some areas, the silt and sand are massive, and the bedding is indistinct or nonexistent. The Hillsboro Formation, which typically consists of stiff to very stiff, brown to gray clay, commonly underlies the Willamette Silt at depths of about 30 feet to 50 feet in this area.

5 SUBSURFACE CONDITIONS

5.1 General

Subsurface materials and conditions at the site were originally investigated on a preliminary basis with four borings, designated B-1 through B-4, and one cone penetration test (CPT) probe, designated CPT-1, from December 20 through December 22, 2021. The borings were advanced to depths ranging from 51.5 feet to 61.5 feet below existing site grades and the CPT probe was advanced to a depth of about 68.7 feet. Supplemental geotechnical investigations were completed between November 1 through 3, 2022 with ten borings including falling-head infiltration testing, designated I-1 through I-10, one hand-auger boring, designated I-9b and, three cone penetration test (CPT) probes, designated CPT-2 through CPT-4. The borings were advanced to depths ranging from 1.5 feet to 11.5 feet below existing site grades and the CPT probes were advanced to depths ranging from about 40.7 feet to 70.2 feet. Additional borings were completed between December 12 through 16, 2022, with six borings, designated B-5 through B-10, and two cone penetration test (CPT) probes, designated CPT-5 and CPT-6. The borings were advanced to depths ranging from 36.5 feet to 76.5 feet below existing site grades and the CPT probes were advanced to depths ranging from about 51.8 feet to 81.2 feet. The approximate location of the explorations completed for these investigations are shown on the Site Plan, Figure 2. Logs of the borings are provided on Figures 1A through 21A, and a log of the CPT explorations are provided on Figures 22A through 28A. The field and laboratory programs conducted to evaluate the physical engineering properties of the materials encountered in the borings are described in Appendix A. The terms and symbols used to describe the materials encountered in the borings and CPT probes are defined in Tables 2A and 3A, respectively, and on the attached legend.

5.2 Soils

For the purpose of discussion, the materials disclosed by our investigation have been grouped into the following categories based on their physical characteristics and engineering properties and listed as they were encountered below the ground surface:

a. Asphalt Concrete PAVEMENT
    b. SILT (Fill and Possible Fill)
    c. Sandy GRAVEL (Fill)
    d. SILT (Willamette Silt)
    e. SILT and CLAY (Hillsboro Formation)

The following paragraphs provide a description of the soil units and a discussion of the groundwater conditions at the site.

a. Asphalt Concrete PAVEMENT
Asphalt concrete pavement was encountered at the ground surface in borings I-3 through I-9, I-10, CPT-5, and CPT-6. In general, the asphalt concrete pavement thickness ranged from about 2 inches to 6 inches and base course thickness ranged from 3 inches to 7 inches. Boring I-8 encountered refusal in the base course at a depth of about 0.8 foot.

b. SILT (Fill and Possible Fill)
Silt fill was encountered below the base course in borings I-4 through I-7, I-10, and CPT-5, and at the ground surface in borings I-1, I-2, I-9b, B-3, B-5, B-6, B-8, B-9, and CPT-2. The silt fill is brown to dark brown, the sand content ranges from a trace of fine- to medium-grained sand to sandy and the clay content ranges from a trace of clay to clayey. The relative consistency of the silt fill is soft to medium stiff and is generally medium stiff. Additional details regarding field and laboratory testing are available in Appendix A.

c. Sandy GRAVEL (Fill)
Sandy gravel fill was encountered below asphalt concrete pavement in exploration I-9. The sandy gravel fill is gray to brown and consists of fine- to coarse-grained sand, angular gravels, and contains concrete debris. The relative consistency of the sandy gravel fill is medium-dense based on field and laboratory testing. Additional details regarding field and laboratory testing are available in Appendix A. Exploration I-9 was terminated in the sandy gravel fill at a depth of about 4 feet.

d. SILT (Willamette Silt)
Silt of the Willamette Silt Formation was encountered at the ground surface in borings I-3, B-1, B-2, B-4, I-3, B-7, B-10, CPT-1, CPT-3, CPT-4, and CPT-6 and at a depth of approximately 5 feet in borings I-1, I-2, I-4 through I-7, B-6, B-8, B-9, CPT-2, and CPT-5, at a depth of approximately 6 feet in B-3, and at a depth of approximately 7.5 feet in borings I-10 and B-5. In general, the silt is brown with scattered rust mottling in the upper 20 feet and typically grades to gray below a depth of about 20 feet. In general, the silt ranges in clay content from up to a trace of clay to clayey and ranges in sand content from a trace of fine-grained sand to sandy. Gray mottled black silty clay to clayey silt of the Willamette Silt formation was encountered in boring B-3. The relative consistency of the silt ranges

from soft to very stiff and is generally medium stiff to stiff. Additional details regarding field and laboratory testing are available in Appendix A.

Explorations I-1 through I-7, I-9b, I-10, B-5, B-6 and B-10 were terminated in the Willamette Silt at depths ranging from about 3 feet to 36.5 feet.

One-dimensional consolidation testing was completed on four samples of silt obtained at depths of about 5.3 feet in boring B-1, 16.5 feet in boring B-2, 11.1 feet in boring B-7 and 8.6 feet in boring B-9. Test results indicate the silt is heavily overconsolidated and exhibits a relatively low compressibility in the preconsolidated range of pressures and moderate compressibility in the normally consolidated range of pressures, see Figures 31A through 34A.

e. SILT and CLAY (Hillsboro Formation)

Silt and Clay of the Hillsboro Formation were encountered beneath the Willamette Silt at a depth of about 40 feet in boring B-1, about 55 feet in borings B-2 and B-8, about 50 feet in borings B-3, B-4, B-9, and in CPT-1, about 60 feet in boring B-7, about 42 feet in CPT-1, and about 48 feet in CPT-5. The silt and clay are typically gray to gray mottled rust and brown in color. In general, the silt contains some clay to clayey and the clay contains some silt to silty. The silt and clay soils typically contain a trace of fine-grained sand. The relative consistency of the clay ranges from stiff to very stiff. Additional details regarding field and laboratory testing are available in Appendix A.

Explorations B-1 through B-4 and B-7 through B-9 were terminated in the Hillsboro Formation at depths ranging from about 51.5 feet to 76.5 feet.

5.3   Groundwater

Borings B-1 through B-10 were completed using mud-rotary drilling techniques, which do not allow direct measurement of groundwater levels at the time of drilling. Groundwater measurements were taken at the time of completing CPT-1 on December 21, 2021, at the time of completing CPT-2 through CPT-4 on November 2, 2022, and at the time of completing CPT-5 and CPT-6 on December 16, 2022. To allow measurement and periodic monitoring of groundwater levels at the site, vibrating-wire piezometers were installed at a depth of about 55 feet below the ground surface in boring B-2, about 34 feet below the ground surface in borings B-5 and B-10, and at a depth of about 20.4 feet below the ground surface in boring B-6. Measurements from the various methods noted above indicate groundwater depths generally range from approximately 5 feet to 10 feet below the ground surface. We anticipate groundwater may approach the ground surface during the wet winter and spring months or following periods of heavy or prolonged rainfall.

5.4 On-Site Stormwater Infiltration

Falling-head infiltration testing was completed at the site on November 1 through 3, 2022, in general conformance with the City of Portland 2020 *Stormwater Management Manual* (SMM) using the encased falling-head method outlined in Section 2.3.2 of the manual. The test locations were designated I-1 through I-9, I-9b, and I-10 and completed in shallow boreholes at depths of about 1.5 feet to 10 feet below existing site grades. The average unfactored, field-measured infiltration rates are tabulated below in Table 4-1.

Table 5-1: INFILTRATION TEST RESULTS

Test No.	Depth of Infiltration Test, feet	Average Field Infiltration Rate, inches/hour	Soil Classification
I-1	2.5	0.5	SILT, some clay and fine-grained sand
I-2	5.0	0.5	SILT, some fine-grained sand, trace clay
I-3	4.0	0.5	SILT, some fine-grained sand
I-4	5.0	< 0.25	Sandy SILT, fine grained sand
I-5	3.0	<0.25	SILT, some fine-grained sand, trace clay
I-6	5.0	< 0.25	Sandy SILT, fine grained sand
I-7	N/A	Groundwater Encountered at approximately 5 ft	SILT, some fine-grained sand, trace clay
I-8	N/A	Drilling Refusal at 1.5 ft	Gravel and Cobbles (Fill)
I-9	N/A	Relocated to I-9b due to access conflict	N/A
I-9b	1.5	< 0.25	SILT, trace fine-grained sand
I-10	10.0	Groundwater Encountered at approximately 10 ft	SILT, some fine-grained sand, trace clay

As noted in section 4.3, relatively shallow groundwater was encountered at the project site. Based on the observed groundwater levels and very low measured infiltration rates disclosed by our explorations, we do not recommend considering on-site infiltration for stormwater disposal. Additional details regarding the infiltration testing are included in Appendix A.

6 CONCLUSIONS AND RECOMMENDATIONS

6.1 General

Subsurface explorations completed for this investigation encountered up to 7.5 feet of fill, underlain by Willamette Silt extending to depths of about 42 feet to 60 feet below the ground surface. The relative consistency of the silt is generally medium stiff to stiff. The Willamette Silt is underlain by typically very stiff silt and clay soils of the Hillsboro

Formation. The local groundwater level varies from depths of about 5 feet to 10 feet below the ground surface at the time of the geotechnical investigation but will fluctuate in response to seasonal rainfall. In addition, groundwater may approach the ground surface in localized areas during the wet winter and spring months or following periods of prolonged or intense precipitation.

In our opinion, foundation support for new structural loads can be provided by conventional spread and strip footings established in firm, undisturbed, native soil or compacted structural fill. The primary geotechnical considerations associated with the construction of the proposed building include the presence of fine-grained soils at the ground surface that are moisture sensitive and the potential for shallow, perched groundwater conditions. The following sections of this report provide our conclusions and recommendations for use in the design and construction of the proposed new Beaverton High School.

6.2   Seismic Considerations

6.2.1   *Design Acceleration Parameters*

We understand seismic design for the project is being completed in accordance with the 2022 OSSC and ASCE 7-16. A site-specific seismic-hazard study was completed for the project to fulfill the requirements of amended Section 1803 of the 2022 OSSC for special occupancy structures. Details of the site-specific seismic-hazard study and development of the recommended response spectrum are provided in Appendix B.

A ground-motion hazard analysis was completed in accordance with Section 21.2 of ASCE 7-16 to develop the site-specific ground motion values. Based on our review of available geologic and subsurface information for the project area and the results of subsurface explorations and the seismic cone penetration testing completed for this project, it is our opinion the site can generally be classified as Site Class D in accordance with Chapter 20 of ASCE 7-16. The recommended response spectra for structural design were developed by comparing the site-specific spectra based on ground motion hazard analysis with the code-based spectra based on Site Class D conditions. For dynamic analysis using the equivalent lateral force design procedure, the 0.2- and 1.0-second MCE_R and design acceleration parameters are developed in accordance with Section 21.4 of ASCE 7-16. Table 5-1 below summarizes the recommended MCE_R- and design-level spectral response parameters for the Site Class D condition developed at the site in accordance with Section 21.4 of ASCE 7-16.

Table 6-1: RECOMMENDED SEISMIC DESIGN PARAMETERS
(2022 OSSC/ASCE 7-16)

Seismic Parameter	Recommended Values*
Site Class	D
MCE$_R$ 0.2-Sec Period Spectral Response Acceleration, S$_{MS}$	1.20 g
MCE$_R$ 1.0-Sec Period Spectral Response Acceleration, S$_{M1}$	0.76 g
Design-Level 0.2-Sec Period Spectral Response Acceleration, S$_{DS}$	0.80 g
Design-Level 1.0-Sec Period Spectral Response Acceleration, S$_{D1}$	0.51 g

6.2.2 Liquefaction, Cyclic Softening, and Other Seismic Hazards

Based on the estimated depth to groundwater, the relative consistency of the fine-grained soil at the site, and cyclic direct simple shear (CDSS) testing completed as a part of our preliminary evaluation, it is our opinion the risk ground surface manifestation of liquefaction, cyclic softening, and/or significant soil strength loss at the site is low during a code-based earthquake. We estimate ground surface manifestation of seismically induced settlements will generally be less than about 1 inch to 2 inches. Based on the location of known and mapped faults in the area, the Helvetia Fault, about 8 kilometers (km) from the site, is the closest dominant crustal fault identified as a hazard to the site. We anticipate the potential for fault rupture or displacement at the site is absent unless occurring on a previously unknown or unmapped fault. The risk of damage by a tsunami and/or seiche at the site is absent.

6.3 Earthwork

6.3.1 General

The fine-grained soils that mantle the site are moisture sensitive, and perched groundwater may approach the ground surface during the wet winter months and following periods of sustained precipitation. Therefore, it is our opinion earthwork can be completed most economically during the dry summer months, typically extending from June to mid-October. It has been our experience the moisture content of the upper few feet of fine-grained soils will decrease during extended warm, dry weather. However, below this depth, the moisture content of the soil tends to remain relatively unchanged and well above the optimum moisture content for compaction. As a result, the contractor must use construction equipment and procedures that prevent disturbance and softening of the subgrade soils. To minimize disturbance of the moisture-sensitive fine-grained soils, site grading can be completed using track-mounted hydraulic excavators. The excavation

should be finished using a smooth-edged bucket to produce a firm, undisturbed surface. It may also be necessary to construct granular haul roads and work pads concurrently with excavation to minimize subgrade disturbance. If the subgrade is disturbed during construction, soft, disturbed soils should be overexcavated to firm soil and backfilled with structural fill.

If construction occurs during wet ground conditions, granular work pads will be required to protect the underlying silt subgrade and provide a firm working surface for construction activities. In our opinion, an 18-inch-thick granular work pad should be sufficient to prevent disturbance of the subgrade by lighter construction equipment and limited traffic by dump trucks. Haul roads and other high-density traffic areas will require a minimum of 18 inches to 24 inches of fragmental rock, up to 6-inch nominal size, to reduce the risk of subgrade deterioration. The use of geotextile fabric over the subgrade may reduce the need for maintenance during construction.

As an alternative to the use of a thickened section of crushed rock to support construction activities and protect the subgrade, the subgrade soils can be treated with cement. It has been our experience in this area that treating the subgrade soils to a depth of 12 inches to 16 inches with about a 6% to 8% admixture of cement overlain by 6 inches to 12 inches of crushed rock will support construction equipment and provide a good all-weather working surface. If cement treatment is being considered, GRI should be contacted prior to construction to assist with refining the preliminary cement admixture estimates noted above.

6.3.2 Site Preparation

The existing structures located within the extent of the proposed improvements, and associated foundations, if any, should be demolished as a part of site preparation activities. Any excavations necessary to remove the structures, soils disturbed during the removal of the foundations, and any soft or otherwise unsuitable soils in the footprint of the structures should be excavated and removed. The excavations should be backfilled in accordance with the Structural Fill section of this report.

The ground surface within all building areas, paved areas, walkways, and areas to receive structural fill should be stripped of existing vegetation, surface organics, and loose surface soils or fill. All trees, brush, and surficial organic material should be removed from within the limits of the proposed improvements. Excavations required to remove unsuitable soils, brush, and trees should be backfilled with structural fill. Organic strippings should be disposed of offsite or stockpiled on site for use in landscaped areas.

Following stripping or excavation to design elevation, the exposed subgrade should be evaluated by a qualified member of GRI's geotechnical engineering staff or an engineering

geologist. Proof rolling with a loaded dump truck may be part of this evaluation. Any soft areas or areas of unsuitable material disclosed by the evaluation should be overexcavated to firm native material and backfilled with structural fill.

6.3.3 Site Grading

We anticipate areal fills for the project will generally be less than 1 foot to 2 feet. If planned fills exceed this thickness, GRI should be contacted to review site grading. In general, grading across the project site should also provide for positive drainage of surface water away from buildings, adjacent properties and slopes to reduce the potential for erosion and ponding.

6.3.4 Prior Site Development

Site improvements within previously developed areas include risk of encountering undocumented or poorly documented improvements and infrastructure. Although not encountered within the subsurface explorations completed at the site, the possibility does exist to encounter existing underground improvements.

6.4 Excavation

6.4.1 General

Based on our preliminary understanding of the project, we anticipate the maximum depth of cuts to establish final site grades will generally be less than 5 feet and the depth of localized utility excavations may be on the order of 5 feet to 10 feet. Excavations completed adjacent to existing structures must be completed outside of the zone of influence of existing footings, defined by 2H:1V (Horizontal to Vertical) line extending downward from the bottom of the foundations. Alternatively, excavations that must be completed within this zone of influence may be supported with temporary shoring that includes surcharge loads from the footings, as shown on the Surcharge-Induced Lateral Pressure, Figure 3.

The method of excavation and design of excavation support are the responsibility of the contractor and are subject to applicable local, state, and federal safety regulations, including the current Occupational Safety and Health Administration (OSHA) excavation and trench safety standards. The means, methods, and sequencing of construction operations and site safety are also the contractor's responsibility. The information provided below is for the use of our client and should not be interpreted to imply we are assuming responsibility for the contractor's actions or site safety.

6.4.2 Utility Excavations

In our opinion, there are three major considerations associated with the design and construction of new utilities:

1. Provide stable excavation sideslopes or support for trench sidewalls to minimize loss of ground.

2. Provide a safe working environment during construction.

3. Minimize post-construction settlement of the utility and ground surface.

According to current OSHA regulations, the fine-grained soils encountered in the explorations may be classified as Type C. In our opinion, trench excavations should be laterally supported or alternatively provided with sideslopes of 1.5H:1V or flatter, provided static groundwater or seepage is not encountered. If groundwater is encountered, the sideslopes should be sloped at 2H:1V or flatter. In our opinion, adequate lateral support may be provided by common methods, such as the use of a trench shield or hydraulic shoring systems.

We anticipate the groundwater level may conflict with deeper trench excavations and perched groundwater may develop in utility trenches and within the near-surface fine-grained soils that mantle the site during periods of heavy or prolonged rainfall. Groundwater seepage, running-soil conditions, and unstable trench sidewalls or soft trench subgrades, if encountered during construction, will require dewatering of the excavation and trench sidewall support. The impact of these conditions can be reduced by completing trench excavation during the summer months when groundwater levels are lowest.

We anticipate groundwater inflow if encountered, can generally be controlled by pumping from sumps. To facilitate dewatering, it will be necessary to overexcavate the trench bottom to permit the installation of a granular working blanket. We estimate the required thickness of the granular working blanket will be on the order of 1 foot or as required to maintain a stable trench base. The actual required depth of overexcavation will depend on the conditions exposed in the trench and the effectiveness of the contractor's dewatering efforts. The thickness of the granular blanket must be evaluated based on field observations during construction. We recommend the use of relatively clean, free-draining material, such as 2- to 4-inch-minus crushed rock, for this purpose.

6.5 Structural Fill

In our opinion, the on-site, fine-grained soils that are free of organics and other deleterious materials and debris are suitable for use in structural fills. Fine-grained soils are moisture sensitive and can be placed and adequately compacted only during the dry summer months from June to mid-October. If silty fill soils are compacted at a moisture content that is higher than recommended, the specified densities cannot be achieved, and the fill material will be relatively weak and compressible.

On-site, fine-grained soil used as structural fill must be moisture conditioned to within 3% of optimum moisture content, as determined by ASTM International (ASTM) D698, prior to compaction. The moisture-conditioned, fine-grained soil should be placed in 9-inch-thick lifts (loose) and compacted with vibratory equipment to at least 95% of the maximum dry density determined in accordance with ASTM D698. For construction during the wet winter and spring months, fills should be constructed using imported granular materials that are relatively clean, as discussed above in Section 5.3.1.

Imported granular material would be most suitable for the construction of structural fills during wet weather. Granular material such as sand, sandy gravel, or crushed rock with a maximum size of 1.5 inches would be suitable structural fill material. Granular material that has less than 5% passing the No. 200 sieve (washed analysis) can typically be placed and effectively compacted during periods of wet weather. Granular backfill should be placed in lifts and compacted with vibratory equipment to at least 95% of the maximum dry density determined in accordance with ASTM D698. Appropriate lift thicknesses will depend on the type of compaction equipment used. For example, if hand-operated, vibratory-plate equipment is used, lift thicknesses should be limited to 6 inches to 8 inches. If smooth-drum vibratory rollers are used, lift thicknesses up to 12 inches are appropriate, and if backhoe- or excavator-mounted vibratory plates are used, lift thicknesses up to 2 feet may be acceptable. A minimum of four passes with the roller are generally required to achieve compaction. Hand-operated equipment should be used within 5 feet of building walls or retaining walls.

All utility trench excavations within building, pavement, and hardscape areas should be backfilled with relatively clean, granular material such as sand, sandy gravel, or crushed rock of up to 1½-inch maximum size and having less than 5% passing the No. 200 sieve (washed analysis). The bottom of the excavation should be thoroughly cleaned to remove loose materials and the utilities should be underlain by a minimum 6-inch thickness of bedding material. The granular backfill material should be compacted to at least 95% of the maximum dry density determined by ASTM D698 in the upper 5 feet of the trench and at least 92% of this density below a depth of 5 feet. The use of hoe-mounted, vibratory-plate compactors is usually the most efficient for this purpose. Flooding or jetting as a means of compacting the trench backfill should not be permitted.

Fill placed in landscaped areas should be compacted to a minimum of about 90% of the maximum dry density as determined by ASTM D698. The moisture content of soils placed in landscaped areas is less critical, provided construction equipment can effectively handle the materials.

6.6 Foundation Support

6.6.1 General

Structural loads are currently unknown; however, we anticipate the maximum column loads will generally be less than about 300 kips. In our opinion, the proposed structural loads can be supported on conventional spread footings in accordance with the following design criteria.

6.6.2 Foundation Design Criteria

All footings should be established in firm, undisturbed, native soil or compacted structural fill at a minimum depth of 18 inches below the lowest adjacent finished grade. Excavations for all foundations should be made with a smooth-edged bucket to reduce subgrade disturbance and a qualified member of GRI geotechnical engineering staff should observe all footing excavations. Soft or otherwise unsuitable material encountered at the foundation subgrade level should be overexcavated and backfilled with granular structural fill. Our experience indicates fine-grained soils are easily disturbed by excavation and construction activities. In this regard, we recommend installing a minimum 4-inch-thick layer of compacted crushed rock in the bottom of all footing excavations. Relatively clean, ¾-inch-minus crushed rock is suitable for this purpose and should be compacted with a lightweight vibratory compactor.

The table below includes an estimate of the nominal bearing pressure for new footings as well as recommended phi factors for the Strength and Extreme limit states.

Table 6-2: ESTIMATED NOMINAL COLUMN FOOTING RESISTANCES

Loading Type	Estimated Nominal Bearing Resistance, psf (Strength and Extreme Cases)	Service Limit Resistance, psf (Service Case)	Recommended Phi Factors	
			Strength Limit State	Extreme Limit State
Bearing Pressure	8,000	3,000	0.45	1.0

We estimate the total static settlement of spread footings designed in accordance with the recommendations presented above will be less than 1 inch for footings loaded to the service limit resistance supporting column loads of up to 300 kips. Differential static settlements between adjacent, comparably loaded footings on similar subgrade conditions are estimated to be less than half the total settlement.

Horizontal shear forces can be resisted partially or completely by frictional forces developed between the base of footings and the underlying soil and by soil passive resistance. The total frictional resistance between the footing and the soil is the normal force times the coefficient of friction between the soil and the base of the footing. We recommend an ultimate value of 0.40 for the coefficient of friction for footings cast on

granular material. The normal force is the sum of the vertical forces, i.e., dead load plus real live load. If additional lateral resistance is required, passive earth pressures against embedded footings can be computed based on an equivalent fluid having a unit weight of 250 pounds per cubic foot. This design passive earth pressure would be applicable only if the footing is cast neat against undisturbed soil or if backfill for the footings is placed as granular structural fill. This design passive earth pressure also assumes up to $0.02*t$ of lateral movement of the structure will occur in order for the soil to develop this resistance, where "t" is the thickness of the footing. This value also assumes the ground surface in front of the foundation is horizontal, i.e., does not slope downward away from the toe of the footing.

6.7   Pavement Design

6.7.1   Recommended Design

We anticipate that the new parking lot and drive over areas at the proposed Beaverton High School replacement will be paved with AC. We anticipate that portions of the parking lot and drive over areas will function as a student parking facility that will not be subjected to regular bus or heavy vehicle (delivery truck) traffic while separate portions will function as a bus yard and drive aisles that will experience regular bus and heavy vehicle traffic. For the purpose of this analysis, we have developed separate design recommendations for new parking areas that will be subjected to only passenger vehicle traffic, and new parking areas that will be subjected to regular bus and heavy vehicle traffic.

Passenger vehicle typically do not cause significant structural damage to pavements; thus, we developed our pavement design recommendations for areas subjected to only passenger vehicle traffic based on the results of the field investigation and the thickness of CRB required to support construction traffic during paving operations.

We developed our pavement design recommendations for areas subjected to regular bus and heavy vehicle traffic based the results of the field investigation and assumptions of bus and heavy vehicle loading developed through our experience with similar school projects in the greater Portland area. For our analysis, we assume that up to 24 school buses meeting the FHWA Class 4 criteria will utilize the heavy traffic parking area per school day, with half of the buses being loaded and half being unloaded. We also assume that one heavy delivery truck and one light delivery truck will utilize the parking area per school day. Traffic loading estimates were developed based on a 20-year design life and a traffic growth rate of 1%.

The pavement designs provided below do not consider any construction traffic associated with construction of new buildings and/or facilities for the proposed Beaverton High

School replacement project. Pavements subjected to construction traffic may require repair. Traffic should not be allowed on the new pavement until all lifts have been placed.

Based on our field investigation and the assumptions stated above, we recommend the following pavement sections provided in Table 6-3.

Table 6-3: RECOMMENDED PAVEMENT SECTIONS

Pavement Type	Traffic Loading	CRB Thickness, inches	Pavement Thickness, inches
AC	Areas Subject to Primarily Passenger Vehicle Traffic	10	3
AC	Areas Subject to Regular Bus and Heavy Vehicle Traffic	14	5

Note: The recommended pavement sections should be considered minimum thicknesses and underlain by a nonwoven geotextile fabric.

It should be assumed that some maintenance will be required over the life of the pavement. The recommended pavement sections are based on the assumption that pavement construction will be accomplished during the dry season and after the construction of the building has been completed. If wet-weather pavement construction is considered, it will likely be necessary to increase the thickness of the CRB course to support construction equipment and protect the subgrade from disturbance, as discussed in the Earthwork section of this report. The indicated sections are not intended to support construction traffic such as forklifts, dump trucks, or concrete trucks.

For the above-indicated sections, drainage is an essential aspect of pavement performance. We recommend all paved areas be provided positive drainage to remove surface water and water within the base course; subgrade should be sloped to a minimum of 0.5% slope to aid in drainage. This will be particularly important in cut sections or at low points within the paved areas, such as at catch basins. Effective methods to prevent saturation of the base-course materials include providing weepholes in the sidewalls of catch basins, subdrains in conjunction with utility excavations and separate trench-drain systems. To help ensure quality materials and construction practices, we recommend the pavement work conform to current Oregon Department of Transportation (ODOT) standards.

Prior to placing base-course materials, all pavement area subgrade should be proof rolled with a fully loaded dump truck. Any soft areas detected by the proof rolling should be overexcavated to firm ground and backfilled with compacted structural fill.

Provided the pavement section is installed in accordance with the above recommendations, it is our opinion the site-access areas will support infrequent traffic by an emergency vehicle having a gross vehicle weight of up to 75,000 pounds. For the

purposes of this evaluation, "infrequent" can be defined as once a month or less. If the frequency of emergency vehicle traffic exceeds this preliminary assumption, GRI should be contacted to review our pavement recommendations.

6.7.2 Standard Specifications

Construction materials and procedures should comply with the applicable sections of the current ODOT *Oregon Standard Specifications for Construction* given in Table 6-4.

Table 6-4: ODOT SPECIFICATIONS FOR PAVEMENT CONSTRUCTION

Materials/Activity	Specification
Asphalt Concrete New Construction	Section 00744. Place the AC section using a minimum lift thickness of 2 inches and maximum lift thickness of 3 inches. Lime or latex treatment of aggregate is not required.
Asphalt Binder	Use Performance Grade 64-22 Asphalt Cement in Level 2.
Aggregate Base	Section 00641 (¾ inch – 0 or 1 inch – 0).
Subgrade Geotextile	Sections 00350 and 02320. (Table 02320-4 Geotextile Property Values)

7   DESIGN REVIEW AND CONSTRUCTION SERVICES

We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed. In addition, GRI should be retained to review all geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in our report. To observe compliance with the intent of our recommendations, the design concepts, and the plans and specifications, it is our opinion all construction operations pertaining to earthwork and foundation installation should be observed by a GRI representative. Our construction-phase services will allow for timely design changes if site conditions are encountered that are different from those described in our report. If we do not have the opportunity to confirm our interpretations, assumptions, and analyses during construction, we cannot be responsible for the application of our recommendations to subsurface conditions different from those described in this report.

8   LIMITATIONS

This report has been prepared to aid the project team in the design of this project. The scope is limited to the specific project and location described within this report, and our description of the project represents our understanding of the significant aspects of the project relevant to earthwork, design and construction of the proposed improvements. If any changes in the design and location of the project elements as outlined in this report are planned, we should be given the opportunity to review the changes and modify or reaffirm the conclusions and recommendations of this report in writing.

The conclusions and recommendations in this report are based on the data obtained from the subsurface explorations at the locations shown on Figure 2 and other sources of information discussed in this report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged that variations in subsurface conditions may exist between exploration locations. This report does not reflect variations that may occur between these explorations. The nature and extent of variation may not become evident until construction. If during construction, subsurface conditions differ from those encountered in the explorations, we should be advised at once so we can observe and review these conditions and reconsider our recommendations where necessary.

Submitted for GRI,

RENEWS 12-2023

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This document has been submitted electronically.

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APPENDIX A
Field Explorations and Laboratory Testing

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

A.1　FIELD EXPLORATIONS

A.1.1　General

Subsurface materials and conditions at the site were investigated from December 20 through 22, 2021 with four borings designated B-1 through B-4, and one cone penetration test (CPT) probe, designated CPT-1. Additional investigations were performed from November 1 through 3, 2022 with ten borings, designated I-1 through I-10, one hand-auger boring, designated I-9b, and three cone penetration test probes, designated CPT-2 through CPT-4 and from December 12 through 16, 2022, with six borings, designated B-5 through B-10; and two cone penetration test probes, designated CPT-5 and CPT-6. The approximate locations of the explorations completed for this investigation are shown on the Site Plan, Figure 2. The field exploration work was coordinated and documented by an experienced member of GRI's geotechnical engineering staff, who maintained a log of the materials and conditions disclosed during the course of work.

A.1.2　Borings

Borings B-1 through B-4 were advanced to depths ranging from about 51.5 feet to 61.5 feet with mud-rotary drilling techniques using a track-mounted drill rig provided and operated by Western States Soil Conservation, Inc. of Hubbard, Oregon. Borings I-1 through I-10 were advanced to depths ranging from about 1.5 feet to about 11.5 feet with solid stem auger techniques using a trailer-mounted drill rig provided by Dan Fischer Excavating, Inc. of Forest Grove, Oregon. Borings B-5 through B-10 were advanced to depths ranging from about 36.5 feet to 76.5 feet with mud-rotary drilling techniques using a track-mounted drill rig provided and operated by Holt Services, Inc. of Vancouver, Washington. Disturbed and undisturbed soil samples were obtained from the borings at 2.5-foot intervals of depth in the upper 15 feet and 5-foot intervals below this depth. Disturbed soil samples were obtained using a standard split-spoon sampler. The standard penetration test (SPT) was completed while obtaining disturbed soil samples. This test is performed by driving a 2-inch-outside-diameter, split-spoon sampler into the soil at a distance of 18 inches using the force of a 140-pound hammer dropped 30 inches. The number of blows required to drive the sampler the last 12 inches is known as the Standard Penetration Resistance, or SPT N-value. The SPT N-values provide a measure of the relative density of granular soils and the relative consistency of cohesive soils. Samples obtained from the borings were placed in airtight sample bags and returned to our laboratory for further classification and testing. In addition, relatively undisturbed samples were collected by pushing a 3-inch-outside-diameter Shelby tube into the undisturbed soil a maximum distance of 24 inches using the hydraulic ram of the drill rig. The soil exposed in the end of the Shelby tube was examined and classified in the field. After classification, the tubes

were sealed with rubber caps and returned to our laboratory for further examination and testing.

Logs of the borings are provided on Figures 1A through 21A. The log presents a summary of the various types of materials encountered in the boring and notes the depth where the materials and/or characteristics of the materials change. To the right of the summary, the numbers and types of samples taken during the drilling operation are indicated. Farther to the right, SPT N-values, moisture contents, Atterberg limits, Torvane shear-strength values, dry unit weights, and percent material passing the No. 200 sieve are shown graphically. The terms used to describe the materials encountered in the borings are defined in Table 1A and the attached legend.

A.1.3 Cone Penetrometer Test Probe

Six CPT probes, designated CPT-1 through CPT-6, were advanced to depths ranging from about 40.7 feet to about 81.2 feet using a track- or truck-mounted CPT rig provided and operated by Oregon Geotechnical Explorations, Inc., of Salem, Oregon. During a CPT, a steel cone is forced vertically into the soil at a constant rate of penetration. The force required to cause penetration at a constant rate can be related to the bearing capacity of the soil immediately surrounding the point of the penetrometer cone. This force is measured and recorded every 2 inches. In addition to the cone measurements, measurements are obtained of the magnitude of force required to force a friction sleeve attached above the cone through the soil. The force required to move the friction sleeve can be related to the undrained shear strength of fine-grained soils. The dimensionless ratio of sleeve friction to point-bearing capacity provides an indicator of the type of soil penetrated. The cone penetration resistance and sleeve friction can be used to evaluate the relative consistency of cohesionless and cohesive soils, respectively. In addition, a piezometer fitted between the cone and the sleeve measures changes in water pressure as the probe is advanced and can also be used to measure the depth of the top of the groundwater table. The probe was also operated using an accelerometer fitted to it, which allows measurement of the arrival time of shear waves from impulses generated at the ground surface. This allows the calculation of shear-wave velocities for the surrounding soil profile.

Logs of the CPT probes are provided on Figure 22A and Figures 24A through 28A, which present a graphical summary of the tip resistance, local (sleeve) friction, friction ratio, pore pressure, and soil behavior type index. The terms used to describe the soils encountered in the probe are defined in Table 3A. Shear-wave velocity measurements recorded for the CPT probe are shown on Figure 23A.

A.1.4 Infiltration Testing

Falling-head infiltration testing was completed at the site on November 1 through 3, 2022, in general conformance with the City of Portland 2020 *Stormwater Management Manual* (SMM) using the encased falling-head method outlined in Section 2.3.2 of the manual. The test locations were designated I-1 through I-9, I-9b, and I-10 and completed in shallow boreholes at depths of about 1.5 to 5 feet below existing site grades. The boreholes designated I-1 through I-9 and I-10 were drilled to the selected depth using a trailer-mounted Buck Rogers drill rig and a 6-inch-diameter solid-stem auger. Borehole I-9b was completed with hand-auger techniques. The borehole was drilled to the depth of the infiltration test, withdrawn and a 4-inch-diameter PVC pipe was seated firmly into the base of the borehole and filled with water to a height of approximately 1 foot above the base of the hole. After soaking overnight, infiltration testing was conducted by reestablishing the water level in the pipe to the target height and recording the drop in water level over one hour or until the water completely drained, whichever occurred first. Where necessary, the infiltration test was repeated until consecutive tests showed little or no change in infiltration rate. The average unfactored, field-measured infiltration rates are tabulated below.

Table 1A: INFILTRATION TEST RESULTS

Test No.	Depth of Infiltration Test, feet	Average Field Infiltration Rate, inches/hour	Soil Classification
I-1	2.5	0.5	SILT, some clay and fine-grained sand
I-2	5.0	0.5	SILT, some fine-grained sand, trace clay
I-3	4.0	0.5	SILT, some fine-grained sand
I-4	5.0	< 0.25	Sandy SILT, fine grained sand
I-5	3.0	<0.25	SILT, some fine-grained sand, trace clay
I-6	5.0	< 0.25	Sandy SILT, fine grained sand
I-7	N/A	Groundwater Encountered at approximately 5 ft	SILT, some fine-grained sand, trace clay
I-8	N/A	Drilling Refusal at 1.5 ft	Gravel and Cobbles (Fill)
I-9	N/A	Relocated to I-9b due to access conflict	N/A
I-9b	1.5	< 0.25	SILT, trace fine-grained sand
I-10	10.0	Groundwater Encountered at approximately 10 ft	SILT, some fine-grained sand, trace clay

After the infiltration testing was completed, disturbed samples of the material were collected and examined in the field, and selected portions were saved in airtight sample

bags for further examination and physical testing in our laboratory. The City of Portland 2020 SMM, Section 2.3.2 recommends encased falling-head test methods using a minimum factor of safety of 2.0 to establish the design infiltration rate.

A.2 LABORATORY TESTING

A.2.1 General

The samples obtained from the borings were examined in our laboratory, where the physical characteristics of the samples were noted, and the field classifications modified where necessary. At the time of classification, the natural moisture content of each sample was determined. Additional testing included Torvane shear strength, dry unit weight, grain-size analyses, Atterberg limits determination, and one-dimensional consolidation. A summary of the laboratory test results has been provided in Table 4A. The following sections describe the testing program in more detail.

A.2.2 Natural Moisture Contents

Natural moisture content determinations were made in conformance with ASTM D2216. The results are summarized on Figures 1A through 21A, where applicable, and in Table 4A.

A.2.3 Torvane Shear Strength

The approximate undrained shear strength of the fine-grained soils was determined using the Torvane shear device. The Torvane is a handheld apparatus with vanes that are inserted into the soil. The torque required to fail the soil in shear around the vanes is measured using a calibrated spring. The results of the Torvane shear-strength testing are summarized on Figures 1A through 21A, where applicable.

A.2.4 Undisturbed Unit Weight

The unit weight, or density, of undisturbed soil samples was determined in the laboratory in substantial conformance with ASTM D2937. The results are summarized on Figures 1A through 21A, where applicable, and in Table 4A.

A.2.5 Atterberg Limits

Atterberg-limits determinations were performed on samples obtained from the borings in conformance with ASTM D4318. The test results are shown graphically on Figures 1A through 21A, where applicable, the Plasticity Charts 29A and 30A, and in Table 4A.

A.2.6 Grain-Size Analysis

A.2.6.1 Washed-Sieve Method

To assist in classification of the soils, samples of known dry weight were washed over a No. 200 sieve. The material retained on the sieve is oven-dried and weighed. The percentage of material passing the No. 200 sieve is then calculated. The results are summarized on Figures 1A through 21A, where applicable, and in Table 4A.

A.2.7 One-Dimensional Consolidation

Two one-dimensional consolidation tests were performed in conformance with ASTM D2435 on relatively undisturbed soil samples extruded from Shelby tubes. This test provides data on the compressibility of underlying fine-grained soils, necessary for settlement studies. The test results are summarized on Figures 31A through 34A in the form of a curve showing percent strain versus applied effective stress. The initial dry unit weight and moisture content of the samples are also shown on the figure.

Table 2A

GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

Relative Density	Standard Penetration Resistance, (N-values) blows/ft	California-Modified Penetration Resistance (SPT N*-values), blows/ft
Very Loose	0 - 4	0 – 11
Loose	4 - 10	11 – 26
Medium Dense	10 - 30	26 – 74
Dense	30 - 50	74 – 120
Very Dense	over 50	more than 120

Description of Consistency for Fine-Grained (Cohesive) Soils

Consistency	Standard Penetration Resistance (N-values), blows/ft	Torvane or Undrained Shear Strength, tsf
Very Soft	0 - 2	less than 0.125
Soft	2 - 4	0.125 - 0.25
Medium Stiff	4 - 8	0.25 - 0.50
Stiff	8 - 15	0.50 - 1.0
Very Stiff	15 - 30	1.0 - 2.0
Hard	over 30	over 2.0

Grain-Size Classification	Modifier for Subclassification		
	Adjective	Primary Constituent SAND or GRAVEL	Primary Constituent SILT or CLAY
		Percentage of Other Material (By Weight)	
Boulders: >12 in.	trace:	5 - 15 (sand, gravel)	5 - 15 (sand, gravel)
Cobbles: 3-12 in.	some:	15 - 30 (sand, gravel)	15 - 30 (sand, gravel)
Gravel: ¼ - ¾ in. (fine) ¾ - 3 in. (coarse)	sandy, gravelly:	30 - 50 (sand, gravel)	30 - 50 (sand, gravel)
Sand: No. 200 - No. 40 sieve (fine) No. 40 - No. 10 sieve (medium) No. 10 - No. 4 sieve (coarse)	trace:	<5 (silt, clay)	*Relationship of clay and silt determined by plasticity index test*
	some:	5 - 12 (silt, clay)	
Silt/Clay: Pass No. 200 sieve	silty, clayey:	12 - 50 (silt, clay)	

Table 3A

CONE PENETRATION TEST (CPT) CORRELATIONS

Cohesive Soils

Cone Tip Resistance, tsf	Consistency
<5	Very Soft
5 to 15	Soft to Medium Stiff
15 to 30	Stiff
30 to 60	Very Stiff
>60	Hard

Cohesionless Soils

Cone Tip Resistance, tsf	Relative Density
<20	Very Loose
20 to 40	Loose
40 to 120	Medium
120 to 200	Dense
>200	Very Dense

Reference

Kulhawy, F. H., and Mayne, P. W., 1990, Manual on Estimating Soil Properties for Foundation Design, Electric Power Research Institute, EL-6800.

APPENDIX B

Site-Specific Seismic-Hazard Evaluation

APPENDIX B

SITE-SPECIFIC HAZARD EVALUATION

B.1 GENERAL

GRI has completed a site-specific seismic hazard evaluation for the proposed Beaverton High School replacement located in Beaverton, Oregon. The purpose of this study was to evaluate the potential seismic hazards associated with regional and local seismicity. The site-specific seismic hazard study is intended to fulfill the requirements of amended Section 1803 of the 2022 Oregon Structural Specialty Code (OSSC) for Special occupancy structure (ORS 455.447), which references the 2016 American Society of Civil Engineers (ASCE) 7-16 document, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 7-16), for seismic design. Our site-specific seismic-hazard study was based on the potential for regional and local seismic activity, as described in the existing scientific literature, and the subsurface conditions at the site, as disclosed by the geotechnical exploration completed for the project. Specifically, our work included the following tasks:

1. A review of available literature, including published papers, maps, open-file reports, seismic histories and catalogs, and other sources of information regarding the tectonic setting, regional and local geology, and historical seismic activity that might have a significant effect on the site.

2. Compilation, examination, and evaluation of existing subsurface data gathered at the site, including classification and laboratory analyses of soil samples. This information was used to prepare a generalized subsurface profile for the site.

3. Identification of potential seismic sources appropriate for the site and characterization of those sources in terms of magnitude, distance, and acceleration response spectra.

4. Office studies based on the generalized subsurface profile and controlling seismic sources resulting in conclusions and recommendations concerning:

   a. Specific seismic events and characteristic earthquakes that might have a significant effect on the project site.

   b. The potential for ground motion amplification and liquefaction or soil-strength loss at the site.

c. Site-specific acceleration response spectra for design of structures at the site.

This appendix describes the work accomplished and summarizes our conclusions and recommendations.

B.2   GEOLOGIC SETTING

B.2.1   General

On a regional scale, the site lies within the Willamette-Puget Sound lowland trough of the Cascadia convergent tectonic system (Blakely et al., 2000). The lowland areas consist of broad north-south-trending basins in the underlying geologic structure between the Coast Range to the west and the Cascade Mountains to the east. The lowland trough is characterized by alluvial plains with areas of buttes and terraces. The site lies approximately 85 kilometers inland from the down-dip edge of the seismogenic extent of the Cascadia Subduction Zone (CSZ), an active convergent-plate boundary along which remnants of the Farallon Plate (the Gorda, Juan de Fuca, and Explorer plates) are being subducted beneath the western edge of the North American continent. The subduction zone is a broad, eastward-dipping zone of contact between the upper portion of the subducting slabs of the Gorda, Juan de Fuca, and Explorer plates and the overriding North American Plate, as shown on the Tectonic Setting Summary, Figure 1B.

On a local scale, the site is located in the Tualatin Basin, a large, southeast-trending structural basin bounded by high-angle, northwest-trending, right-lateral, strike-slip faults considered seismogenic. The distribution of faults considered active within the Quaternary Period by the U.S. Geological Survey (USGS) are shown on the Local Fault Map, Figure 2B. Information regarding the continuity and potential activity of these faults is lacking due largely to the scale at which geologic mapping in the area has been conducted and the presence of thick, geologically young, basin-filling sediments that obscure structural features of the underlying rock. Active faults may be present within the basin, but clear stratigraphic and/or geophysical evidence regarding their location and extent is not presently available.

Published geologic mapping indicates the site is mantled with Missoula flood deposits, locally referred to in the project area as the Willamette Silt Formation (Ma et al., 2009). In general, Willamette Silt is composed of beds and lenses of silt and sand. Stratification within this formation commonly consists of 4- to 6-inch-thick beds, although in some areas, the silt and sand are massive, and the bedding is indistinct or nonexistent. The Hillsboro Formation, which typically consists of stiff to very stiff, brown to gray clay, commonly underlies the Willamette Silt at depths of about 40 feet to 50 feet in this area. The depth to basalt bedrock at this site is estimated to be on the order of 600 feet

(Schlicker and Deacon, 1967). The local surface geology in close proximity to the site is shown on the Local Geologic Map, Figure 3B.

B.3 SEISMICITY

B.3.1 General

Because of the proximity of the site to the CSZ and its location within the Tualatin Basin, three seismic sources contribute to the potential for damaging earthquake motions at the site. Two of these sources are associated with tectonic activity related to the CSZ, including the interface subduction-zone events related to sudden slip between the upper surface of the Juan de Fuca Plate and lower surface of the North American Plate and subcrustal (Benioff zone) events related to deformation and volume changes within the deeper portion of the subducted Juan de Fuca Plate. The third source is associated with movement on relatively shallow faults within and adjacent to the Portland Basin. Each of these sources is considered capable of producing damaging earthquakes in the Pacific Northwest; however, there are no historical records of significant subcrustal earthquakes (M_W >6.0) in northwest Oregon and southwest Washington. Wong (2005) hypothesizes that due to subduction-zone geometry, geophysical conditions, and local geology, southwest Washington and northwest Oregon may not be subject to subcrustal earthquakes of significant magnitude.

Based on review of historical records and evaluation of U.S. Geological Survey (USGS) national seismic-hazard maps (NSHMs), the two primary types of seismic sources at the site are the CSZ interface and local crustal faults.

B.3.2 Cascadia Subduction Zone

Coastal paleoseismic evidence, offshore geological studies, and historical tsunami accounts indicate the CSZ is capable of producing large-magnitude, megathrust earthquakes (M_W 8 to M_W 9) at the interface between the Juan de Fuca and North American plates (Atwater et al., 1995; Goldfinger et al., 2012). Geological studies indicate these megathrust earthquakes have occurred repeatedly in the past 10,000 years (Walton et al., 2021). A combination of paleoseismic and geologic studies (Kelsey et al., 2005) and geodetic studies (Savage et al., 2000) indicate rate of strain accumulation consistent with the assumption that the CSZ is locked beneath offshore northern California, Oregon, Washington, and southern British Columbia (Fluck et al., 1997; Wang et al., 2001). Numerous geological and geophysical studies suggest the CSZ may be segmented (Hughes and Carr, 1980; Weaver and Michaelson, 1985; Guffanti and Weaver, 1988; Goldfinger, 1994; Kelsey and Bockheim, 1994; Mitchell et al., 1994; Personius, 1995; Nelson and Personius, 1996; Witter, 1999), but the most recent studies suggest that for the last great earthquake in 1700, most of the subduction zone ruptured in a single M_W 9.0 earthquake (Satake et al., 1996; Atwater and Hemphill-Haley, 1997; Clague et al., 2000).

There is consensus within the scientific community that the most recent great earthquake occurred along the CSZ in January 1700 (Atwater et al., 2015) based on paleoseismic evidence and historical records of an orphan tsunami in Japan. Tsunami modeling completed for the 1700 orphan tsunami indicated the 1700 earthquake ruptured the whole length of the CSZ and had a moment magnitude of about M_W 9.0 (Satake et al., 2003).

The average recurrence interval for a CSZ megathrust event is estimated to be around 350 years to 600 years based on prehistoric geologic evidence (Atwater and Hemphill-Haley 1997, Kelsey et al., 2002; Witter et al., 2003). Tsunami inundation in buried marshes along the Washington and Oregon coast and stratigraphic evidence from the Cascadia margin support these recurrence intervals (Kelsey et al., 2005; Goldfinger et al., 2003). Goldfinger et al. (2003, 2012, 2017) evaluated turbidite evidence at the heads of Cascadia submarine canyons, results of which indicated the occurrence of more than 40 great earthquakes over the past 10,000 years with partial or entire length rupture of the CSZ. About 20 of the earthquake events are associated with partial ruptures concentrated in the southern part of the margin and have estimated recurrence intervals of about 220 years to 320 years. About 19 of the events are associated with a rupture of the full CSZ, characterized by a moment magnitude (M_W) of about 8.5 to 9.1 or greater earthquake. Considering a combination of recent paleoseismic, geodetic, and geologic research, the average recurrence interval for a full-rupture CSZ earthquake is estimated to be about 500 years to 540 years (Walton et al., 2021).

The USGS probabilistic analysis assumes four potential locations (three alternative down-dip edge options and one up-dip edge option) for the eastern edge of the earthquake rupture zone for the CSZ, as shown on Figure 4B. As discussed in Petersen et al. (2014), the 2014 USGS mapping effort represents the 2014 CSZ source model with the full-CSZ ruptures with moment magnitudes from M_W 8.6 to M_W 9.3, supplemented by partial ruptures with smaller magnitudes (M_W 8.0 to M_W 9.1). There is also a possibility of serial M_W 8 earthquakes that rupture the entire CSZ over a period of a few decades or less; however, this is not implemented in the current NSHMs. The partial ruptures were accounted for using a segmented model and an unsegmented model. The magnitude-frequency distribution showing the contributions to the earthquake rates from each of the models and how the estimated rates vary along the fault is presented on Figure 5B. In general, the earthquake rates along the CSZ are dominated by the full-characteristic CSZ ruptures (i.e., from northern California to southern British Columbia), indicating the larger M_W 8.6 to M_W 9.3 earthquakes likely occur more often than the smaller, segmented ruptures.

B.3.3 Local Crustal Event

Sudden crustal movements along relatively shallow, local faults in the project area, although rare, have been responsible for local crustal earthquakes. The precise relationship between specific earthquakes and individual faults is not well understood since few of the faults in the area are expressed at the ground surface and there is a limited history of crustal events in the region. The history of local seismic activity is commonly used as a basis for determining the size and frequency to be expected of local crustal events. Although the historical record of local earthquakes is relatively short (the earliest reported seismic event in the area occurred in 1920), it can serve as a guide for estimating the potential for seismic activity in the area.

Based on fault mapping conducted by the USGS (2014 National Seismic Hazard Maps), there are about six faults within 25 km of the site the USGS identifies as contributing to the crustal seismic hazard: the Helvetia Fault at about 8 km, Portland Hills Fault at about 9.5 km, Bolton Fault at about 13 km, Newberg Fault at about 21 km, Grant Butte fault at about 21 km and the Gales Creek fault Zone at about 25 km. Based on our review of the faults, the Helvetia Fault is the closest dominant crustal fault identified as a hazard to the site, with a characteristic magnitude of M_W 6.4. In general, our review of the 2014 USGS Probabilistic Seismic Hazard Analysis (PSHA) deaggregations indicates the faults in the area contribute about 17% to the overall seismic hazard at the site. Our review of the 2014 USGS PSHA deaggregations also indicates the background-gridded seismic source is one of the sources contributing significantly to the seismicity of the site. The background-gridded seismic source is an areal source zone, which accounts for random earthquakes that are not attributed to known faults. The background seismicity is represented by a characteristic earthquake magnitude of about M_W 6.05 and contributes about 12% to the overall seismic hazard at the site.

B.4 SPECTRAL ACCELERATION VALUES

B.4.1 General

The seismic evaluation for the proposed Beaverton High School replacement is being completed in accordance with the 2022 OSSC, which references ASCE 7-16. A ground-motion hazard analysis is being completed in accordance with Section 21.2 of ASCE 7-16 to develop the recommended Risk-Targeted Maximum Considered Earthquake (MCE_R) with the intent of including the probability of structural collapse. The recommended MCE_R response spectra is generally developed by comparing a site-specific and code-based spectral value at the ground surface. The site-specific ground motion is defined as the lesser of a probabilistic and a deterministic ground motion. The code-based spectral values are developed using the mapped bedrock spectral acceleration parameters, S_S and S_1, at the site and corresponding site amplification coefficients, Fa and Fv, to account for underlying soil conditions in accordance with Chapter 11 of ASCE 7-16.

B.4.2   Site-Specific MCE$_R$ Spectral Values

As previously stated, the site-specific MCE$_R$ spectral response acceleration is defined by lesser spectral response accelerations from probabilistic ground motions and deterministic ground motions. The probabilistic ground motion represents ground motion with a targeted risk level of 1% probability of collapse within a 50-year period in the direction of maximum horizontal response with 5% damping. The site-specific probabilistic seismic hazard analysis (PSHA) was conducted using the recently released 2018 USGS NSHM Hazard tool. In accordance with guidelines of ASCE 7-16, the site-specific PSHA represents the probabilistic ground motions as spectral response acceleration values with a 2,475-year recurrence interval (i.e., 2% probability of exceedance in 50 years) in the geomean direction. The probabilistic MCE$_R$ spectral values are then derived by applying directivity factors and risk coefficients to the site-specific PSHA values. The directivity factors adjust the spectral values from geometric mean to direction of maximum horizontal response and the risk coefficients incorporate the uniform collapse risk objective of 1% in a 50-year period. The site-specific PSHA was conducted for a site condition with an average shear-wave velocity of 942 feet/second (i.e., Site Class D) based on the results of the seismic CPT probe completed for the project in the upper 100 feet. The site-specific probabilistic MCE$_R$ values are summarized in Table 1B below.

Table 1B: SITE-SPECFIC PROBABILISTIC MCE$_R$ VALUES

Period, sec	Prob. MCE$_R$ Values (g)
PGA	0.56
0.05	0.62
0.1	0.93
0.2	1.24
0.3	1.33
0.5	1.22
0.75	0.97
1	0.77
2	0.42
3	0.27
4	0.20
5	0.15

The deterministic ground motions at the site can also be developed concurrently with the site-specific PSHA in accordance with Section 21.2.2 of ASCE 7-16. However, an exception is included in Section 21.2.2 of ASCE 7-16, allowing the deterministic analysis to be disregarded when the largest spectral response acceleration from the probabilistic ground

motion is less than 1.2 F_a (i.e., F_a of 1.15 for Site Class D conditions). Therefore, the deterministic analysis was not completed for the site since the largest spectral response acceleration from the probabilistic ground motion, S_a = 1.16 g, is less than 1.2 Fa=1.38 g. Therefore, the probabilistic MCE_R spectral acceleration at any period defines the site-specific MCE_R spectrum at the site.

B.4.3 Design Acceleration Parameters

The recommended response spectra for structural design is typically developed by comparing the site-specific spectra based on the ground motion hazard analysis with the code-based spectra based on site class and generic site-amplification factors. At the project site, the site is designated Site Class D based on the V_S profile in the upper 100 feet developed from the seismic CPT prob. The code-based Site Class D spectrum was derived based on the 0.2- and 1.0-second spectral-acceleration values (S_S and S_1) at the bedrock and corresponding site coefficients, Fa and Fv, in accordance with Chapter 21 of ASCE 7-16 with proposed amendment in Subsection 1613.4.13 of 2022 OSSC. The modification typically applies to the value of Fv, suggested to be determined using straight-line interpolation between the value determined from ASCE 7-16 Section 21.3 (i.e., associated with 0% CSZ interface contribution) and the value from 2022 OSSC Table 1613.3.3(2) (i.e., associated with 100% CSZ interface contribution) based on the relative hazard contribution from the CSZ interface sources at a period of 1 sec. The relative CSZ interface hazard contribution was obtained using the 2018 USGS NSHM Hazard Tool for the 2,475-year hazard at a spectral period of 1.0 sec for an average shear-wave velocity of 942 feet/second. The USGS hazard tool shows about 74% contribution from the CSZ interface source at the site. The 0.2- and 1.0-second spectral values (S_S and S_1) for the site at bedrock are 0.89 and 0.41, respectively. The short-period site coefficient, Fa, which equals 1.15, was determined using Table 11.4-1 of ASCE 7-16. The long-period site coefficient, Fv, which equals 2.05, was determined using straight-line interpolation between the ASCE 7-16-recommended value of 2.5 and the 2022 OSSC value of 1.9 based on the relative CSZ interface hazard contribution. These site coefficients were applied in developing the Site Class D spectrum. ASCE 7-16 requires the site-specific spectral accelerations at the ground surface not be less than 80% of the spectral values determined for Site Class D.

Comparisons of the site-specific MCE_R and the code-based ground-surface spectra are shown on Figure 6B. The site-specific MCE_R response spectra was generally observed to be higher than the code-based 80% Site Class D spectra at all periods. Therefore, site-specific MCE_R spectral values are recommended for design of the structure. The design response spectral values are generally developed by taking two-thirds of the MCE_R response spectral values.

For dynamic analysis using the equivalent lateral force (ELF) design procedure, the 0.2- and 1.0-second MCE_R and design acceleration parameters are developed in accordance with Section 21.4 of ASCE 7-16. In accordance with Section 21.4, the 0.2-second MCE_R spectral value (S_{MS}) was taken as 90% of the maximum spectral acceleration obtained from the site-specific spectrum at any period within the range of 0.2 second to 5.0 seconds. The 1.0-second MCE_R spectral value (S_{M1}) was derived based on 90% of the maximum value of the product of spectral acceleration and corresponding periods for periods ranging from 1.0 to 5.0 seconds for sites with a V_{S30} value less than 1,200 feet/second (i.e., Site Class D). Table 2B summarizes the recommended MCE_R and design acceleration parameters in accordance with Section 21.4 of ASCE 7-16.

Table 2B: RECOMMENDED SEISMIC DESIGN PARAMETERS
(2022 OSSC/ASCE 7-16)

Seismic Parameter	Recommended Values*
Site Class	D
MCE_R 0.2-Sec Period Spectral Response Acceleration, S_{MS}	1.20 g
MCE_R 1.0-Sec Period Spectral Response Acceleration, S_{M1}	0.76 g
Design-Level 0.2-Sec Period Spectral Response Acceleration, S_{DS}	0.80 g
Design-Level 1.0-Sec Period Spectral Response Acceleration, S_{D1}	0.51 g

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