EXHIBIT H.

Stormwater Management Facilities

Private Stormwater Report

Sandy Campus Park

HDG Job #: LAN004

Prepared For: City of Sandy
39250 Pioneer Blvd
Sandy, OR 97055

Prepared By: Humber Design Group, Inc.

110 SE Main St. Suite 200
Portland, OR 97214
(P) 503 946 6690

REGISTERED PROFESSIONAL ENGINEER
89353
OREGON
SEPT. 12, 2017
ALLEN SCHMITZ
RENEWAL DATE 6/30/2024

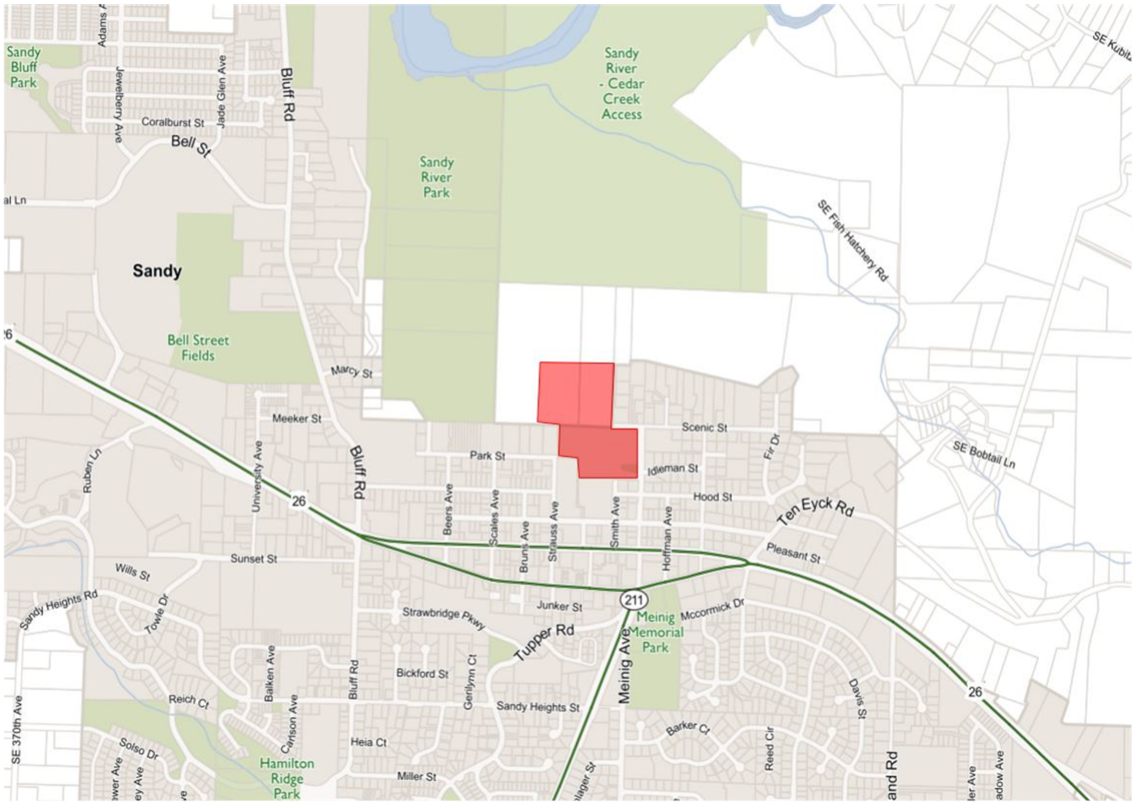
Date: July 17, 2023

Table of Contents

Project Overview and Description

Location of Project	17225 SE Meinig Ave, Sandy, OR 97055
Site Area/Acreage	10 acres
Proposed Impervious Area	
Nearest Cross Street	Scenic St
Property Zoning	Medium Density Residential & Parks and Open Space
Existing Conditions	The existing site contains concrete paving stake park, asphalt sidewalk, and parking lot swith trees and structures.
Proposed Development	The proposed site will consists of a pump track, skate park, play area, and 1 story shelter with parking lot.
Watershed Description	Sandy River
Subwatershed	Sedar Creek
Tax Map	24E13BA & 24E13BD
Tax Lot	24E13BD00101 & 24E13BA00200 & 24E13BA00300
Permits Required	Public Works Permit 1200C Erosion Control Permit

Vicinity Map



Site Location

Methodology

Existing Drainage	Stormwater on the site is currently conveyed to various area drains and catch basins where it is conveyed to as existing public storm pipe that existing 30" outfall located on west side of project site.
Infiltration Results	Pali Consulting, Inc performed (2) infiltration tests. The test were at a depth of 5ft and 15ft BFG with an infiltration rate of 1 in/hr.
PRIVATE Proposed Stormwater Management Techniques	Stormwater from the new impervious area will be managed by providing both flow control and water quantity. Stormwater will be conveyed to a water quality manhole where it treated based SWMM requirements. From there it will be conveyed to a 96" CMP detention tank with orifice control. The flow control orifice has been sized to match the post developed peak flow to pre development peak flow for one-half the 2yr, 2yr, 5yr, 10yr, and 25yr.
PUBLIC Proposed Stormwater Management Techniques	New impervious area along Scenic street will create or replace greater than 500 SF of impervious area, therefore, stormwater management will be required. This area will be managed using the water quality manhole and detention tank.
Discharge Point	Drainage Way, River, Storm Only Pipe
Stormwater Hierarchy Justificatio	Due to poor infiltration at the site, level 1 of the discharged hierarchy is not feasible. This site fall under level two of the discharge hierarchy.

Analysis

Computational Method Used HydroCAD models of a SBUH Type 1A Storm were used to calculate the stormwater management facility sizes for the catchment areas. See attached calculations. Below is a summary of the results.

Soil Types Silty Clay Loam

Table 1 – Curve Numbers

Predeveloped Pervious CN	79
Predeveloped Impervious CN	98
Post-Developed Pervious CN	79
Post-Developed Impervious CN	98

Table 2 – Design Storms

WQ Storm	0.83 inches
2-year	2.40 inches
10-year	3.40 inches
25-year	3.90 inches
100-year	4.40 inches

Table 3 – Time of Concentration

Predeveloped TOC	10 min
Post-Developed TOC	10 min

Stormwater Management Narrative Stormwater runoff from the 87,042 SF of new impervious area from private site and 6,220 SF of new impervious area from public ROW will be managed with a 96" detention tank with water quality filter manhole. Stormwater will be conveyed to existing 30" outfall located on west of property. Stormwater runoff the 10,625 SF of new impervious area from private site will be traded and managed with 96" detention tank with water quality filter manhole, since it it not practical to capture and treat stormwater from the linear pathway the areas that are being captured will be overtreated and overdetermined in order to make up for the areas not captured.

Table 4 – Catchment Areas and Facility Table

Catchment/ Facility ID	Source (roof, road, etc.)	Treatment Area (sf)	Ownership (private/ public)	Facility Type/ Function	Facility Size
A	Roof, Hardscaping	87,042	Private	Mech. Filter, Structural Detention	96"dia. X 200'
B	Road	6,220	Public	Mech. Filter, Structural Detention	96"dia. X 200'
C	Hardscaping	10,625	Private	Mech. Filter, Structural Detention	96"dia. X 200'

Engineering Conclusions

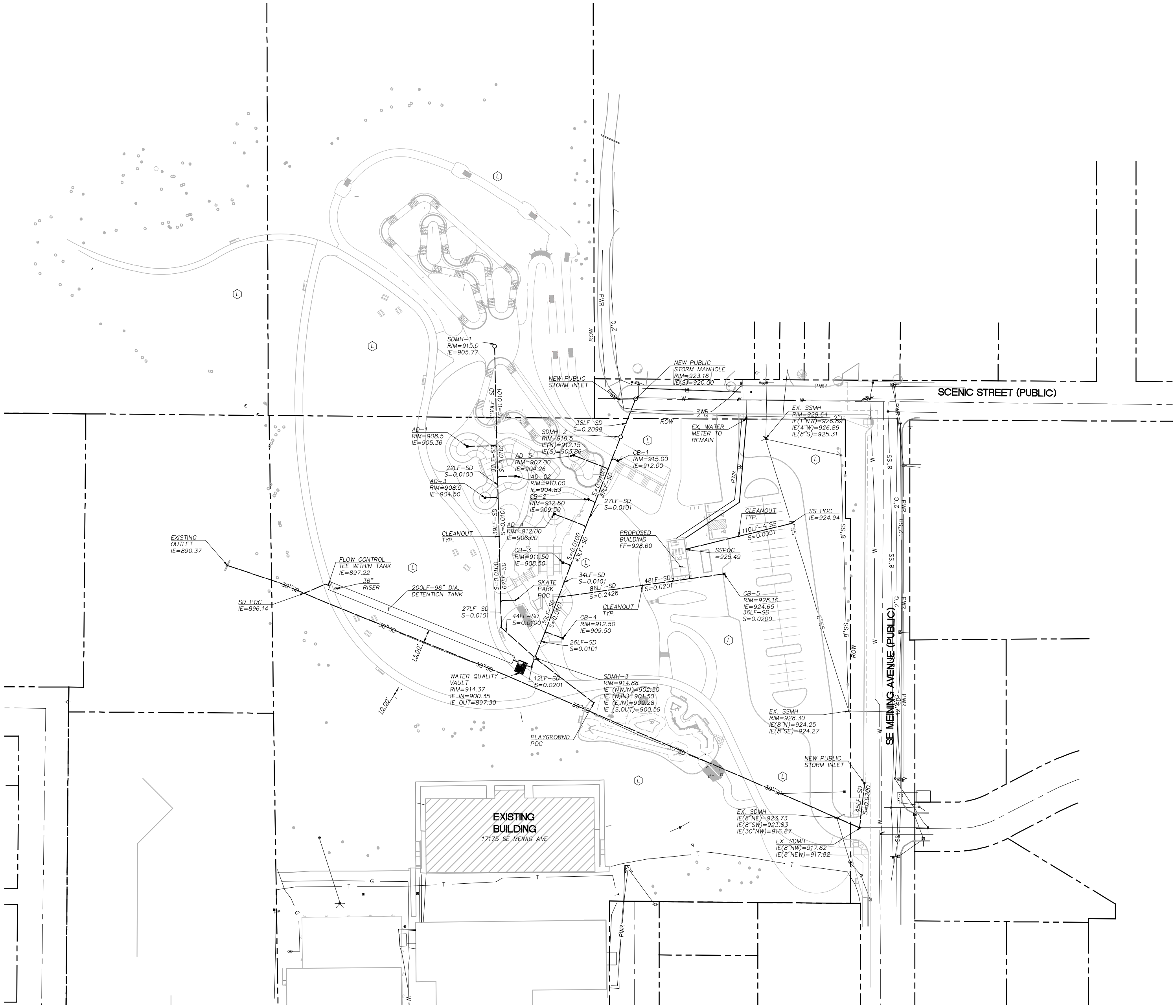
The preceding methodologies and calculations presented indicate compliance with the current jurisdictional stormwater management codes and requirements. A summarized breakdown is presented below:

Water Quality	The proposed development will meet the provisions for water quality per the 2020 Portland Stormwater Management Manual.
Water Quantity	The proposed development will meet the provisions for water quantity per the 2020 Portland Stormwater Management Manual.
Downstream / Upstream Impacts	By providing both the water quality and flow control systems to manage the stormwater runoff from this site we expect there to be no upstream or downstream impacts created by the proposed development.

Appendix A

Stormwater Facility Details / Exhibits

Utility Plan
Catchment Map
Detentaion Tank Details
Water Quaility Manhole Detail



UTILITY PLAN
SCALE: 1"=40'

SHEET LEGEND

SD	STORM	
SS	SANITARY	
W	WATER	
PWR	POWER	
■	CATCH BASIN	
•	CLEANOUT	
○	MANHOLE	
▭	DETENTION PIPE	

lango.hansen
LANDSCAPE ARCHITECTS PC
1100 nw glisan #3B portland OR 97209 T 503 295.2437

REGISTERED PROFESSIONAL ENGINEER
89352
OREGON
SEPT. 12, 2017
ALLEN SCHMITZ
PRELIMINARY
RENEWAL DATE 6/30/24

SANDY COMMUNITY CAMPUS PARK

CITY OF SANDY PARKS AND RECREATION

CITY OF SANDY
17225 SMITH AVE
SANDY, OR 97055

LAND USE

REVISIONS

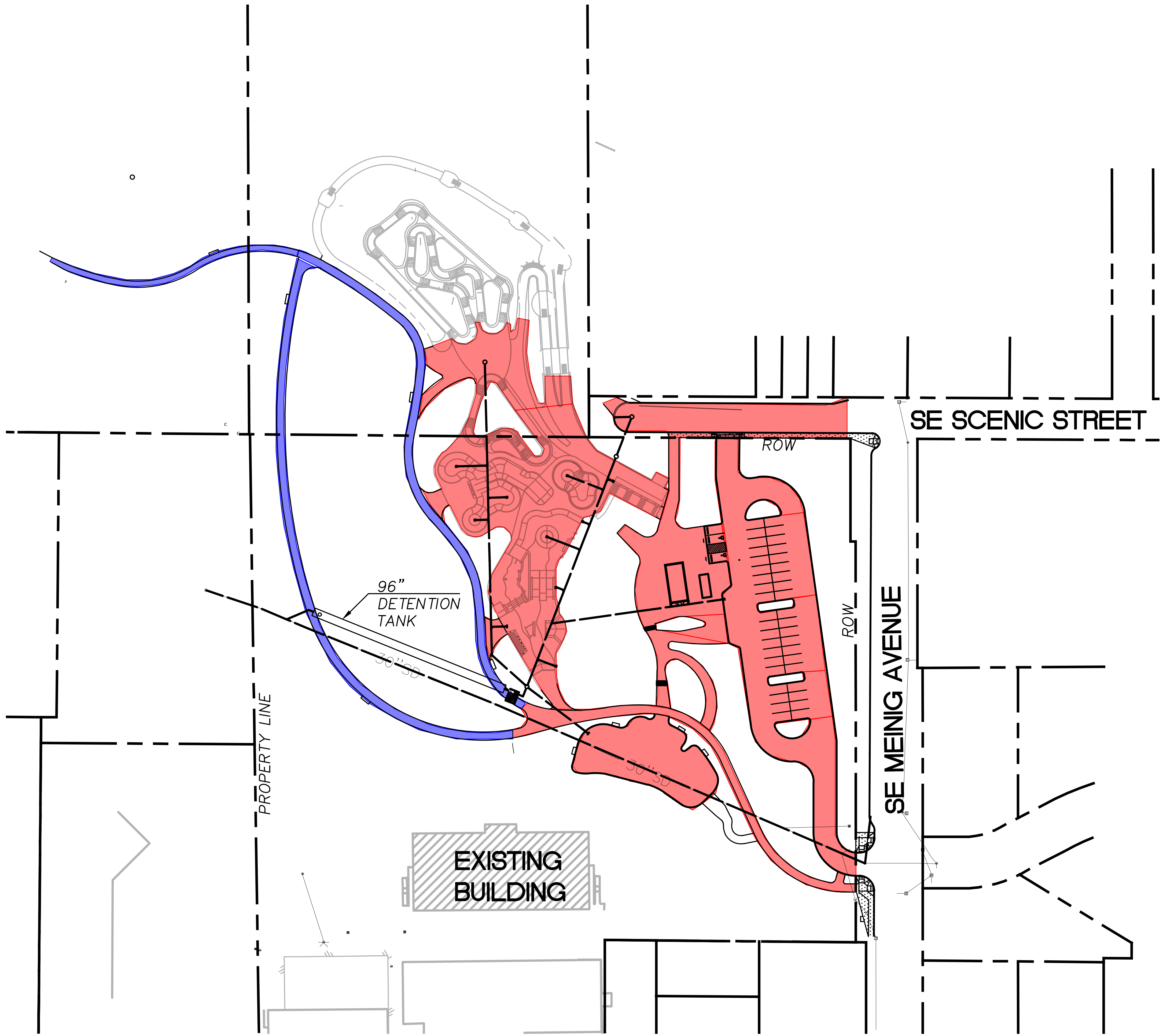
GRAPHIC SCALE
0 40 80
1 inch = 40 ft.

Humber Design Group, Inc.
Civil Engineering • 503.946.6690 • hdgpdx.com

SCALE	AS NOTED
DRAWN BY	MCS
DATE	07.17.23
PROJECT NO.	2239

UTILITY PLAN

C3.00



NEW IMPERVIOUS AREA = 87,042 SF

NEW IMPERVIOUS AREA WILL BE TRADED = 10,625 SF

TOTAL IMPERVISOUSE AREA=97,667 SF

GRAPHIC SCALE

0 100 200

1 inch =100 ft.

SANDY PARK

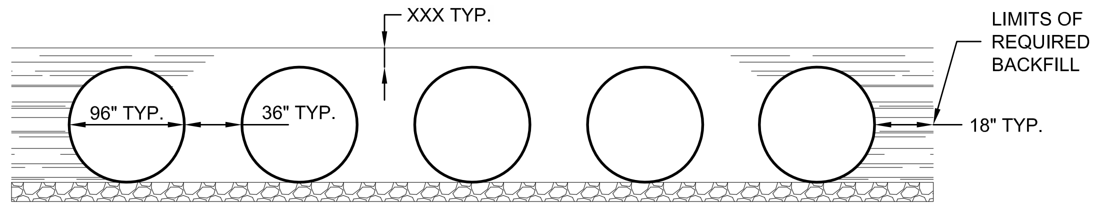
CATCHMENT MAP

PROJECT NO.:	LAN004
DRAWN BY:	MCS
DESIGN BY:	AKS
REVIEWED BY:	AKS
DATE:	07/17/2023

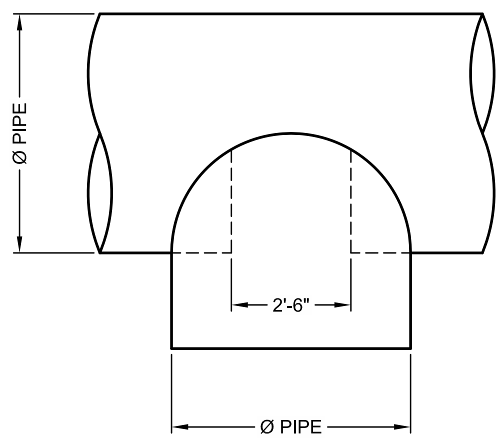
Humber Design Group, Inc.

Civil Engineering ▪ 503.946.6690 ▪ hdgpdx.com

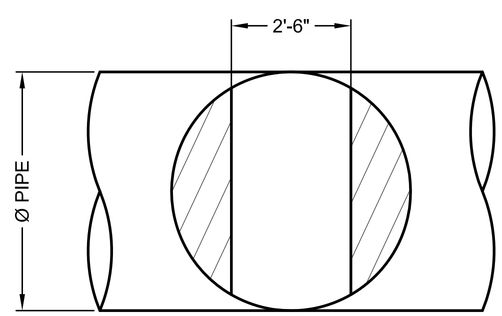
P:\LAN004 - Sandy Campus Park\7 - CAD Drawings\B - Exhibits\2023-06-07 Impervious Area Exhibit\LAN004 IMPERVIOUS AREA EXHIBIT.dwg Jul 17, 2023 - 9:23am



TYPICAL SECTION VIEW
SCALE: N.T.S.



PLAN



FRONT

TYPICAL MANWAY DETAIL
SCALE: N.T.S.

FABRICATION BoM

FITTING	TYPE	QTY	Ø	CORRUGATION	GAGE	FINISH	WALL TYPE	LENGTH	TOTAL
BAND FASTENER	12" HUGGER		96	w/BAR BOLT & STRAP	16	ALT2			
GASKETS	FLAT		96	12" WIDE					

PROJECT SPECIFIC BILL OF MATERIALS TO BE COMPLETED AT TIME OF CONTRACT DRAWINGS

MARK	DATE	REVISION DESCRIPTION	BY

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800-338-1122 513-645-7000 513-645-7993 FAX

CONTECH® CMP DETENTION SYSTEMS

CONTECH CONTRACT DRAWING

Ø96" UNDERGROUND DETENTION SYSTEM - 000000-001
SAMPLE PROJECT
ANYTOWN, USA
SITE DESIGNATION: UDS

PROJECT No.:	SEQ. No.:	DATE:
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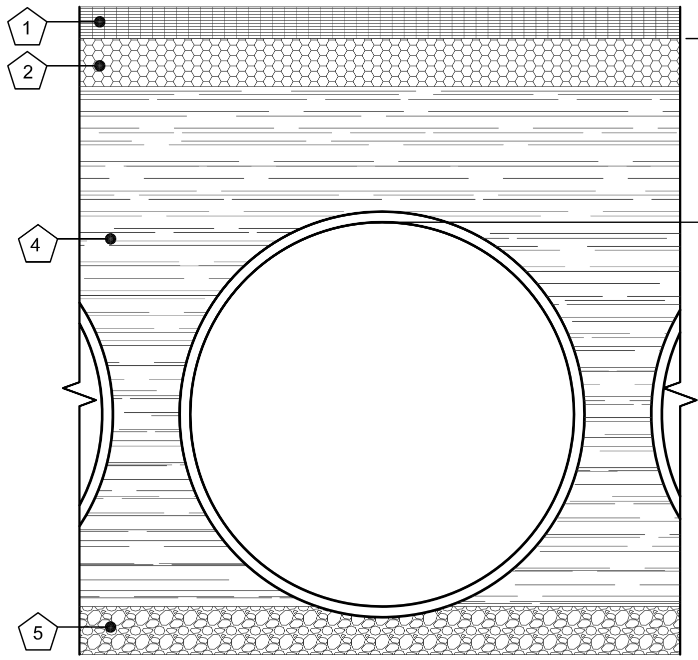
DESIGNED: XXX		DRAWN: RTF
CHECKED:		APPROVED:
SHEET NO.: C2 OF 4		



KEY

1. RIGID OR FLEXIBLE PAVEMENT
2. GRANULAR ROAD BASE
3. 12" MIN. FOR DIAMETERS THROUGH 96"
   18" MIN. FOR DIAMETERS FROM 102"
   AND LARGER MEASURED TO TOP OF RIGID
   OR BOTTOM OF FLEXIBLE PAVEMENT.
4. SELECT GRANULAR FILL PER AASHTO M145
   A1, A2 OR A3, OR APPROVED EQUAL.
   PLACED IN 8" LIFTS (COMPACTED TO MIN.
   90% STANDARD DENSITY PER AASHTO T99.)
5. GRANULAR BEDDING, ROUGHLY SHAPED TO
   FIT THE BOTTOM OF PIPE, 4" TO 6" IN DEPTH

FOUNDATION/BEDDING PREPARATION

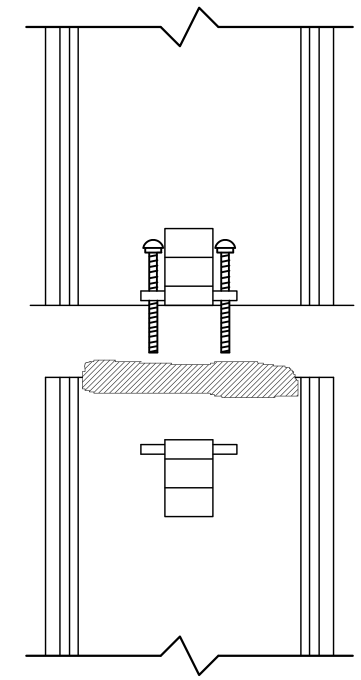
PRIOR TO PLACING THE BEDDING, THE FOUNDATION MUST BE CONSTRUCTED TO A UNIFORM AND STABLE GRADE. IN THE EVENT THAT UNSUITABLE FOUNDATION MATERIALS ARE ENCOUNTERED DURING EXCAVATION, THEY SHALL BE REMOVED AND BROUGHT BACK TO THE GRADE WITH A FILL MATERIAL AS APPROVED BY THE ENGINEER. ONCE THE FOUNDATION PREPARATION IS COMPLETE, 4" - 6" OF A WELL-GRADED GRANULAR MATERIAL SHALL BE PLACED AS THE BEDDING.

BACKFILL

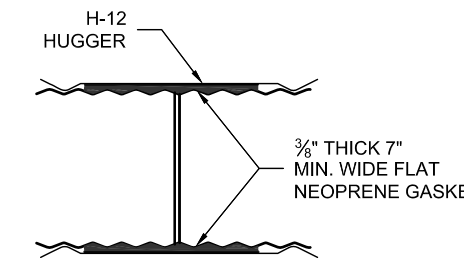
THE BACKFILL SHALL BE AN A1, A2 OR A3 GRANULAR FILL PER AASHTO M145, OR A WELL-GRADED GRANULAR FILL AS APPROVED BY THE SITE ENGINEER (SEE INSTALLATION GUIDELINES). THE MATERIAL SHALL BE PLACED IN 8" LOOSE LIFTS AND COMPACTED TO 90% AASHTO T99 STANDARD PROCTOR DENSITY. WHEN PLACING THE FIRST LIFTS OF BACKFILL IT IS IMPORTANT TO MAKE SURE THAT THE BACKFILL IS PROPERLY COMPACTED UNDER AND AROUND THE PIPE HAUNCHES. BACKFILL SHALL BE PLACED SUCH THAT THERE IS NO MORE THAN A TWO LIFT (16") DIFFERENTIAL BETWEEN ANY OF THE PIPES AT ANY TIME DURING THE BACKFILL PROCESS. THE BACKFILL SHALL BE ADVANCED ALONG THE LENGTH OF THE DETENTION SYSTEM AT THE SAME RATE TO AVOID DIFFERENTIAL LOADING ON THE PIPE.

OTHER ALTERNATE BACKFILL MATERIAL MAY BE ALLOWED DEPENDING ON SITE SPECIFIC CONDITIONS, AS APPROVED BY SITE ENGINEER.

1 / C3 BACKFILL DETAIL
SCALE: N.T.S.



CONNECTION DETAIL
SINGLE BOLT, BAR AND STRAP



FLAT GASKET

GENERAL NOTES

1. BANDS ARE NORMALLY FURNISHED AS FOLLOWS:
   12" THRU 48", 1-PIECE
   54" THRU 96", 2-PIECE
   102" THRU 144", 3-PIECES
2. BAND FASTENERS ARE ATTACHED WITH SPOT WELDS, RIVETS OR HAND WELDS
3. REROLLED ANNULAR END CORRUGATIONS ARE NORMALLY 2⅔" x ½". DIMENSIONS ARE SUBJECT TO MANUFACTURING TOLERANCES

2 / C3 H-12 HUGGER BAND DETAIL
SCALE: N.T.S.

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ANYTOWN, USA
SITE DESIGNATION: UDS

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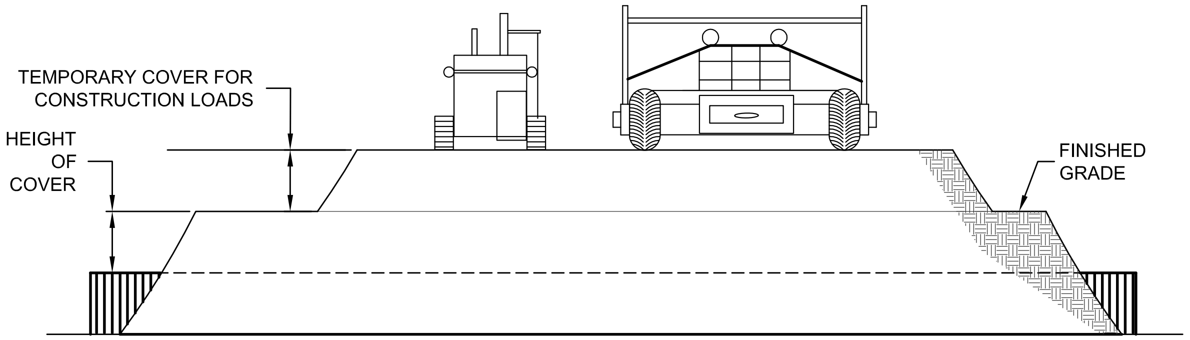


CONSTRUCTION LOADS

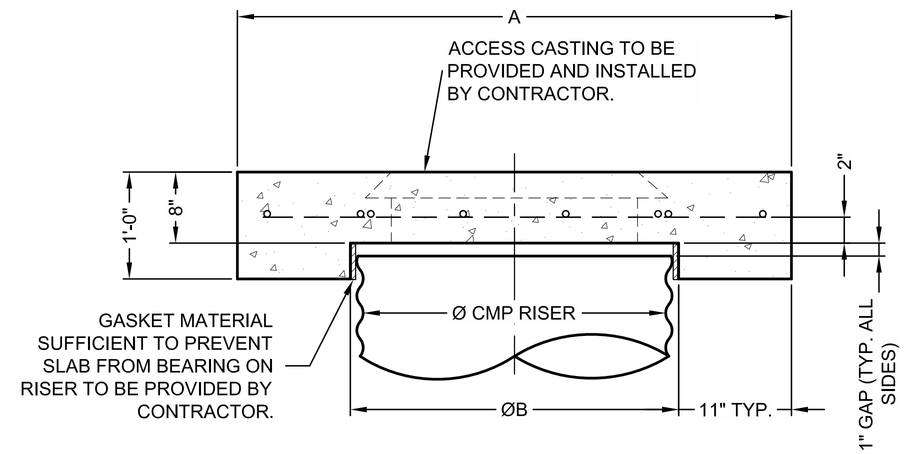
FOR TEMPORARY CONSTRUCTION VEHICLE LOADS, AN EXTRA AMOUNT OF COMPACTED COVER MAY BE REQUIRED OVER THE TOP OF THE PIPE. THE HEIGHT-OF-COVER SHALL MEET THE MINIMUM REQUIREMENTS SHOWN IN THE TABLE BELOW. THE USE OF HEAVY CONSTRUCTION EQUIPMENT NECESSITATES GREATER PROTECTION FOR THE PIPE THAN FINISHED GRADE COVER MINIMUMS FOR NORMAL HIGHWAY TRAFFIC.

PIPE SPAN, INCHES	AXLE LOADS (kips)			
	18-50	50-75	75-110	110-150
	MINIMUM COVER (FT)			
12-42	2.0	2.5	3.0	3.0
48-72	3.0	3.0	3.5	4.0
78-120	3.0	3.5	4.0	4.0
126-144	3.5	4.0	4.5	4.5

*MINIMUM COVER MAY VARY, DEPENDING ON LOCAL CONDITIONS. THE CONTRACTOR MUST PROVIDE THE ADDITIONAL COVER REQUIRED TO AVOID DAMAGE TO THE PIPE. MINIMUM COVER IS MEASURED FROM THE TOP OF THE PIPE TO THE TOP OF THE MAINTAINED CONSTRUCTION ROADWAY SURFACE.

3 / C4 CONSTRUCTION LOADING DIAGRAM

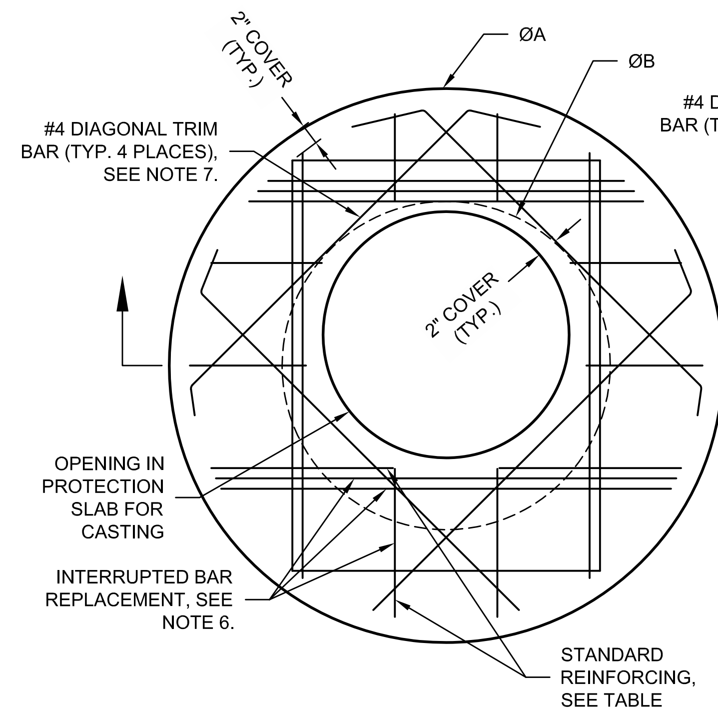
SCALE: N.T.S.



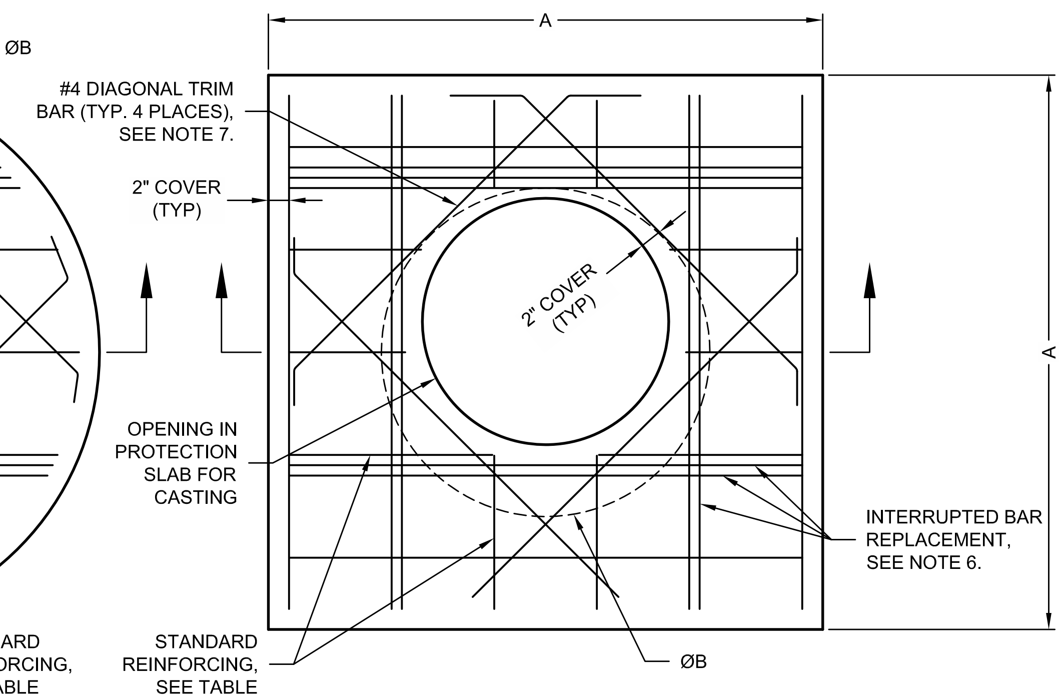
SECTION VIEW

REINFORCING TABLE				
Ø CMP RISER	A	Ø B	REINFORCING	**BEARING PRESSURE (PSF)
24"	Ø 4' 4'X4'	26"	#5 @ 12" OCEW #5 @ 12" OCEW	2,410 1,780
30"	Ø 4'-6" 4'-6" X 4'-6"	32"	#5 @ 12" OCEW #5 @ 12" OCEW	2,120 1,530
36"	Ø 5' 5' X 5'	38"	#5 @ 10" OCEW #5 @ 10" OCEW	1,890 1,350
42"	Ø 5'-6" 5'-6" X 5'-6"	44"	#5 @ 10" OCEW #5 @ 9" OCEW	1,720 1,210
48"	Ø 6' 6' X 6'	50"	#5 @ 9" OCEW #5 @ 8" OCEW	1,600 1,100

** ASSUMED SOIL BEARING CAPACITY



ROUND OPTION PLAN VIEW



SQUARE OPTION PLAN VIEW

NOTES:

1. DESIGN IN ACCORDANCE WITH AASHTO, 17th EDITION.
2. DESIGN LOAD HS25.
3. EARTH COVER = 1' MAX.
4. CONCRETE STRENGTH = 3,500 psi
5. REINFORCING STEEL = ASTM A615, GRADE 60.
6. PROVIDE ADDITIONAL REINFORCING AROUND OPENINGS EQUAL TO THE BARS INTERRUPTED, HALF EACH SIDE. ADDITIONAL BARS TO BE IN THE SAME PLANE.
7. TRIM OPENING WITH DIAGONAL #4 BARS, EXTEND BARS A MINIMUM OF 12" BEYOND OPENING, BEND BARS AS REQUIRED TO MAINTAIN BAR COVER.
8. PROTECTION SLAB AND ALL MATERIALS TO BE PROVIDED AND INSTALLED BY CONTRACTOR.
9. DETAIL DESIGN BY DELTA ENGINEERING, BINGHAMTON, NY.

4 / C4 MANHOLE CAP DETAIL

SCALE: N.T.S.

SPECIFICATION FOR CORRUGATED STEEL PIPE-ALUMINIZED TYPE 2 STEEL

SCOPE

THIS SPECIFICATION COVERS THE MANUFACTURE AND INSTALLATION OF THE CORRUGATED STEEL PIPE (CSP) DETAILED IN THE PROJECT PLANS.

MATERIAL

THE ALUMINIZED TYPE 2 STEEL COILS SHALL CONFORM TO THE APPLICABLE REQUIREMENTS OF AASHTO M274 OR ASTM A929.

PIPE

THE CSP SHALL BE MANUFACTURED IN ACCORDANCE WITH THE APPLICABLE REQUIREMENTS OF AASHTO M36 OR ASTM A760. THE PIPE SIZES, GAGES AND CORRUGATIONS SHALL BE AS SHOWN ON THE PROJECT PLANS.

ALL FABRICATION OF THE PRODUCT SHALL OCCUR WITHIN THE UNITED STATES.

HANDLING AND ASSEMBLY

SHALL BE IN ACCORDANCE WITH RECOMMENDATIONS OF THE NATIONAL CORRUGATED STEEL PIPE ASSOCIATION (NCSPA)

INSTALLATION

SHALL BE IN ACCORDANCE WITH AASHTO STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, SECTION 26, DIVISION II OR ASTM A798 AND IN CONFORMANCE WITH THE PROJECT PLANS AND SPECIFICATIONS. IF THERE ARE ANY INCONSISTENCIES OR CONFLICTS THE CONTRACTOR SHOULD DISCUSS AND RESOLVE WITH THE SITE ENGINEER.

IT IS ALWAYS THE RESPONSIBILITY OF THE CONTRACTOR TO FOLLOW OSHA GUIDELINES FOR SAFE PRACTICES.

MARK	DATE	REVISION DESCRIPTION	BY

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www.ContechES.com
9025 Centre Pointe Dr., Suite 400, West Chester, OH 45069
800-338-1122 513-645-7000 513-645-7993 FAX

CONTECH CMP DETENTION SYSTEMS

CONTECH CONTRACT DRAWING

Ø96" UNDERGROUND DETENTION SYSTEM - 000000-001
SAMPLE PROJECT
ANYTOWN, USA
SITE DESIGNATION: UDS

PROJECT No.: XXXXX	SEQ. No.: 001	DATE: 1/13/2014
DESIGNED: XXX		DRAWN: RTF
CHECKED:		APPROVED:
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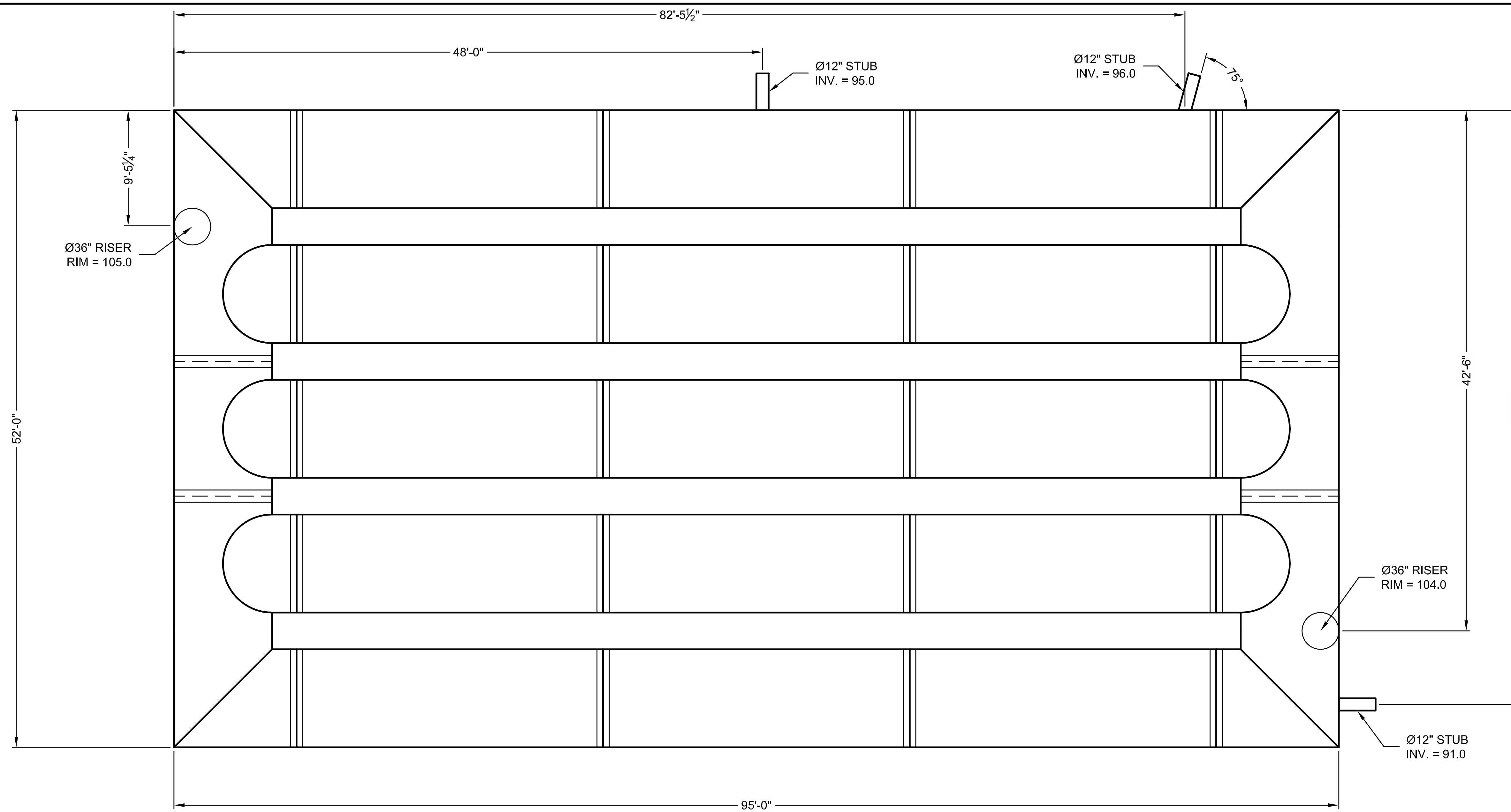

THE UNDERSIGNED HEREBY APPROVES THE ATTACHED (4) PAGES INCLUDING THE FOLLOWING:

- **VOLUME = 25,082 C.F.**
- **MAINLINE PIPE GAUGE = 16**
- **WALL TYPE = SOLID**
- **DIAMETER = 96"**
- **FINISH = ALT2**

CUSTOMER ______________ DATE ______________

ASSEMBLY

SCALE: 1" = 10'
VOLUME: 25,082 C.F.
LOADING: H20/H25
SYSTEM INV = 91.0

NOTES

- ALL RISER AND STUB DIMENSIONS ARE TO CENTERLINE.
- ALL ELEVATIONS, DIMENSIONS, AND LOCATIONS OF RISERS AND INLETS, SHALL BE VERIFIED BY THE ENGINEER OF RECORD PRIOR TO RELEASING FOR FABRICATION.
- ALL FITTINGS AND REINFORCEMENT COMPLY WITH ASTM A998.
- ALL RISERS AND STUBS ARE 2⅔" x ½" CORRUGATION AND 16 GAGE UNLESS OTHERWISE NOTED.
- RISERS TO BE FIELD TRIMMED TO GRADE.
- QUANTITY OF PIPE SHOWN DOES NOT PROVIDE EXTRA PIPE FOR CONNECTING THE SYSTEM TO EXISTING PIPE OR DRAINAGE STRUCTURES. OUR SYSTEM AS DETAILED PROVIDES NOMINAL INLET AND/OR OUTLET PIPE STUB FOR CONNECTION TO EXISTING DRAINAGE FACILITIES. IF ADDITIONAL PIPE IS NEEDED IT IS THE RESPONSIBILITY OF THE CONTRACTOR.

MARK	DATE	REVISION DESCRIPTION	BY

CONTECH ENGINEERED SOLUTIONS LLC
www.ContechES.com
9025 Centre Pointe Dr., Suite 400, West Chester, OH 45069
800-338-1122 513-645-7000 513-645-7993 FAX

CONTECH CMP DETENTION SYSTEMS

CONTECH CONTRACT DRAWING

Ø96" UNDERGROUND DETENTION SYSTEM - 000000-001
SAMPLE PROJECT
ANYTOWN, USA
SITE DESIGNATION: UDS

PROJECT No.:	SEQ. No.:	DATE:
XXXXX	001	1/13/2014
DESIGNED: XXX		DRAWN: RTF
CHECKED:		APPROVED:
SHEET NO.: C1 OF 4		

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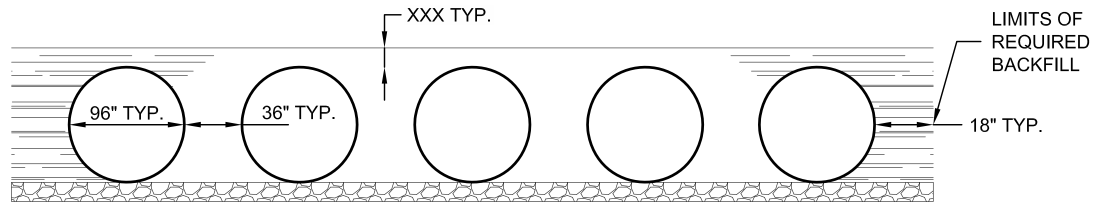
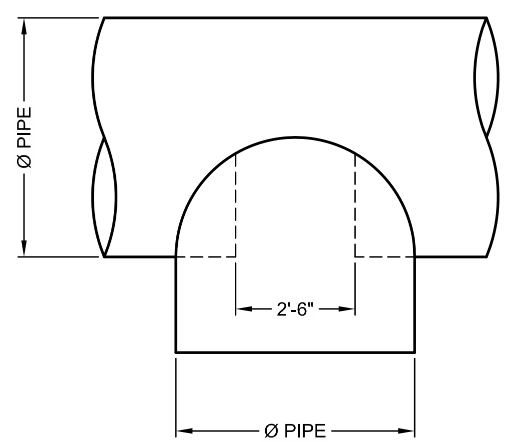
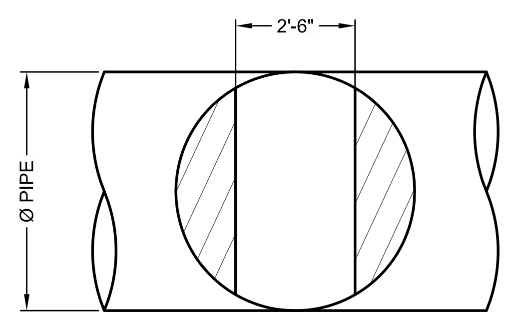


TYPICAL SECTION VIEW
SCALE: N.T.S.



PLAN



FRONT

TYPICAL MANWAY DETAIL
SCALE: N.T.S.

FABRICATION BoM

FITTING	TYPE	QTY	Ø	CORRUGATION	GAGE	FINISH	WALL TYPE	LENGTH	TOTAL
BAND FASTENER	12" HUGGER		96	w/BAR BOLT & STRAP	16	ALT2			
GASKETS	FLAT		96	12" WIDE					

PROJECT SPECIFIC BILL OF MATERIALS
TO BE COMPLETED AT TIME OF CONTRACT
DRAWINGS

C:\USERS\FOWLERT\DESKTOP\SAMPLE PROJECT - SOLIDWALL.DWG 1/13/2014 4:32 PM

MARK	DATE	REVISION DESCRIPTION	BY

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9025 Centre Pointe Dr., Suite 400, West Chester, OH 45069
800-338-1122 513-645-7000 513-645-7993 FAX

CONTECH® CMP DETENTION SYSTEMS

CONTECH CONTRACT DRAWING

Ø96" UNDERGROUND DETENTION SYSTEM - 000000-001
SAMPLE PROJECT
ANYTOWN, USA
SITE DESIGNATION: UDS

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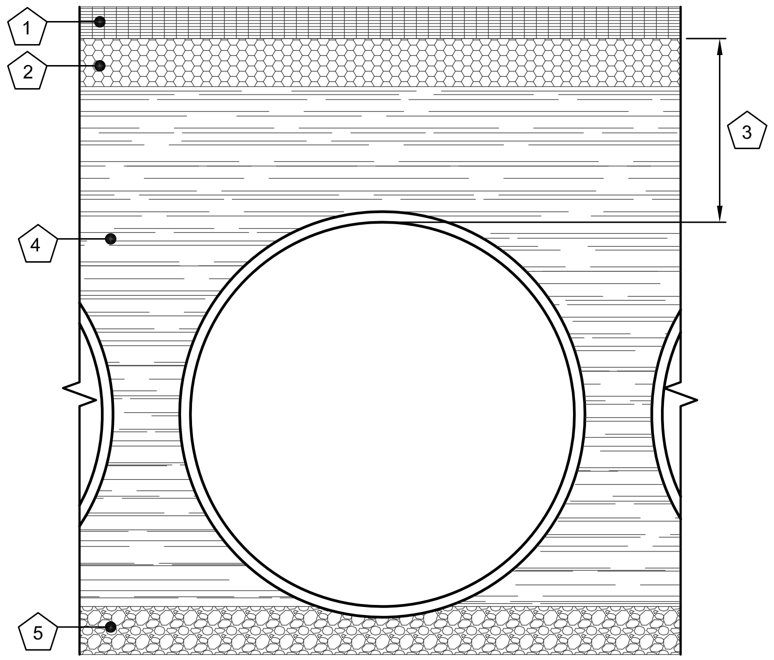


KEY

1. RIGID OR FLEXIBLE PAVEMENT
2. GRANULAR ROAD BASE
3. 12" MIN. FOR DIAMETERS THROUGH 96"
   18" MIN. FOR DIAMETERS FROM 102"
   AND LARGER MEASURED TO TOP OF RIGID
   OR BOTTOM OF FLEXIBLE PAVEMENT.
4. SELECT GRANULAR FILL PER AASHTO M145
   A1, A2 OR A3, OR APPROVED EQUAL.
   PLACED IN 8" LIFTS (COMPACTED TO MIN.
   90% STANDARD DENSITY PER AASHTO T99.)
5. GRANULAR BEDDING, ROUGHLY SHAPED TO
   FIT THE BOTTOM OF PIPE, 4" TO 6" IN DEPTH

FOUNDATION/BEDDING PREPARATION

PRIOR TO PLACING THE BEDDING, THE FOUNDATION MUST BE CONSTRUCTED TO A UNIFORM AND STABLE GRADE. IN THE EVENT THAT UNSUITABLE FOUNDATION MATERIALS ARE ENCOUNTERED DURING EXCAVATION, THEY SHALL BE REMOVED AND BROUGHT BACK TO THE GRADE WITH A FILL MATERIAL AS APPROVED BY THE ENGINEER. ONCE THE FOUNDATION PREPARATION IS COMPLETE, 4" - 6" OF A WELL-GRADED GRANULAR MATERIAL SHALL BE PLACED AS THE BEDDING.

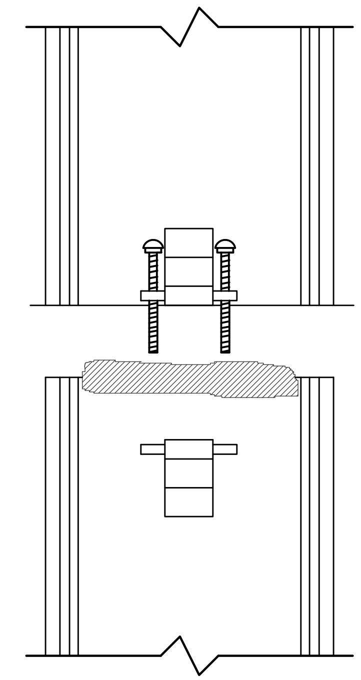
BACKFILL

THE BACKFILL SHALL BE AN A1, A2 OR A3 GRANULAR FILL PER AASHTO M145, OR A WELL-GRADED GRANULAR FILL AS APPROVED BY THE SITE ENGINEER (SEE INSTALLATION GUIDELINES). THE MATERIAL SHALL BE PLACED IN 8" LOOSE LIFTS AND COMPACTED TO 90% AASHTO T99 STANDARD PROCTOR DENSITY. WHEN PLACING THE FIRST LIFTS OF BACKFILL IT IS IMPORTANT TO MAKE SURE THAT THE BACKFILL IS PROPERLY COMPACTED UNDER AND AROUND THE PIPE HAUNCHES. BACKFILL SHALL BE PLACED SUCH THAT THERE IS NO MORE THAN A TWO LIFT (16") DIFFERENTIAL BETWEEN ANY OF THE PIPES AT ANY TIME DURING THE BACKFILL PROCESS. THE BACKFILL SHALL BE ADVANCED ALONG THE LENGTH OF THE DETENTION SYSTEM AT THE SAME RATE TO AVOID DIFFERENTIAL LOADING ON THE PIPE.

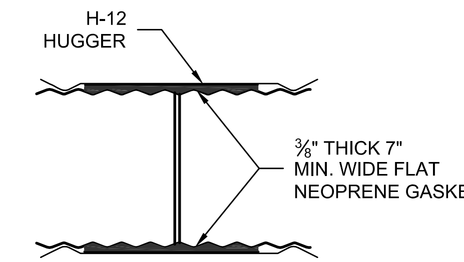
OTHER ALTERNATE BACKFILL MATERIAL MAY BE ALLOWED DEPENDING ON SITE SPECIFIC CONDITIONS, AS APPROVED BY SITE ENGINEER.

1 / C3 BACKFILL DETAIL

SCALE: N.T.S.



CONNECTION DETAIL
SINGLE BOLT, BAR AND STRAP



FLAT GASKET

GENERAL NOTES

1. BANDS ARE NORMALLY FURNISHED AS FOLLOWS:
   12" THRU 48", 1-PIECE
   54" THRU 96", 2-PIECE
   102" THRU 144", 3-PIECES
2. BAND FASTENERS ARE ATTACHED WITH SPOT WELDS, RIVETS OR HAND WELDS
3. REROLLED ANNULAR END CORRUGATIONS ARE NORMALLY $2\frac{2}{3}$" x $\frac{1}{2}$". DIMENSIONS ARE SUBJECT TO MANUFACTURING TOLERANCES

2 / C3 H-12 HUGGER BAND DETAIL

SCALE: N.T.S.

C:\USERS\FOWLERT\DESKTOP\SAMPLE PROJECT - SOLIDWALL.DWG 1/13/2014 4:32 PM

MARK	DATE	REVISION DESCRIPTION	BY

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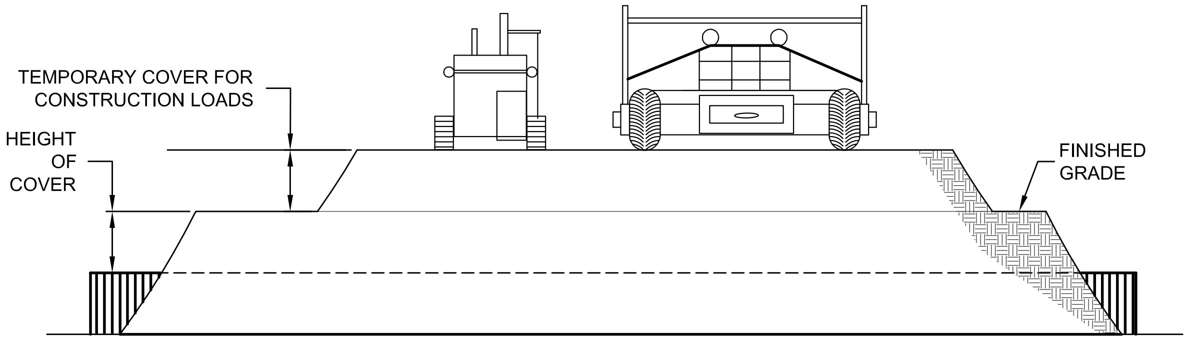
800-338-1122 513-645-7000 513-645-7993 FAX

CONTECH® CMP DETENTION SYSTEMS

CONTECH CONTRACT DRAWING

Ø96" UNDERGROUND DETENTION SYSTEM - 000000-001
SAMPLE PROJECT
ANYTOWN, USA
SITE DESIGNATION: UDS

PROJECT No.:	SEQ. No.:	DATE:
XXXXX	001	1/13/2014
DESIGNED: XXX		DRAWN: RTF
CHECKED:		APPROVED:
SHEET NO.: C3 OF 4		



CONSTRUCTION LOADS

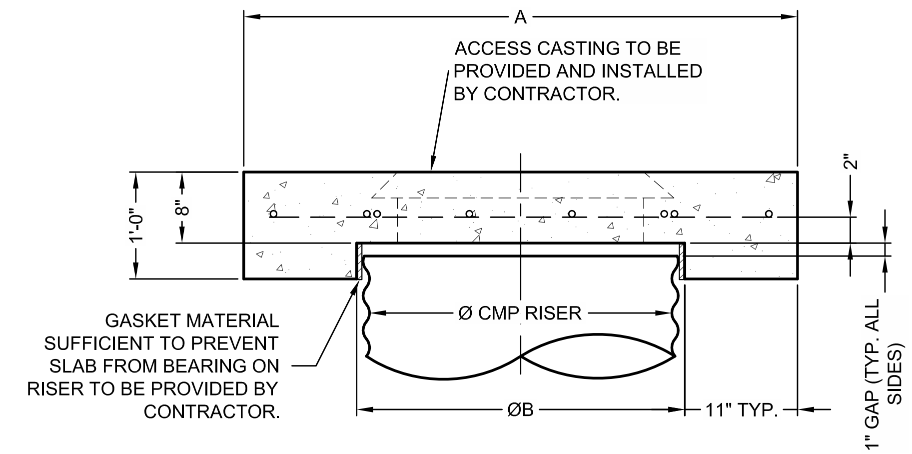
FOR TEMPORARY CONSTRUCTION VEHICLE LOADS, AN EXTRA AMOUNT OF COMPACTED COVER MAY BE REQUIRED OVER THE TOP OF THE PIPE. THE HEIGHT-OF-COVER SHALL MEET THE MINIMUM REQUIREMENTS SHOWN IN THE TABLE BELOW. THE USE OF HEAVY CONSTRUCTION EQUIPMENT NECESSITATES GREATER PROTECTION FOR THE PIPE THAN FINISHED GRADE COVER MINIMUMS FOR NORMAL HIGHWAY TRAFFIC.

PIPE SPAN, INCHES	AXLE LOADS (kips)			
	18-50	50-75	75-110	110-150
	MINIMUM COVER (FT)			
12-42	2.0	2.5	3.0	3.0
48-72	3.0	3.0	3.5	4.0
78-120	3.0	3.5	4.0	4.0
126-144	3.5	4.0	4.5	4.5

*MINIMUM COVER MAY VARY, DEPENDING ON LOCAL CONDITIONS. THE CONTRACTOR MUST PROVIDE THE ADDITIONAL COVER REQUIRED TO AVOID DAMAGE TO THE PIPE. MINIMUM COVER IS MEASURED FROM THE TOP OF THE PIPE TO THE TOP OF THE MAINTAINED CONSTRUCTION ROADWAY SURFACE.

3 / C4 CONSTRUCTION LOADING DIAGRAM

SCALE: N.T.S.

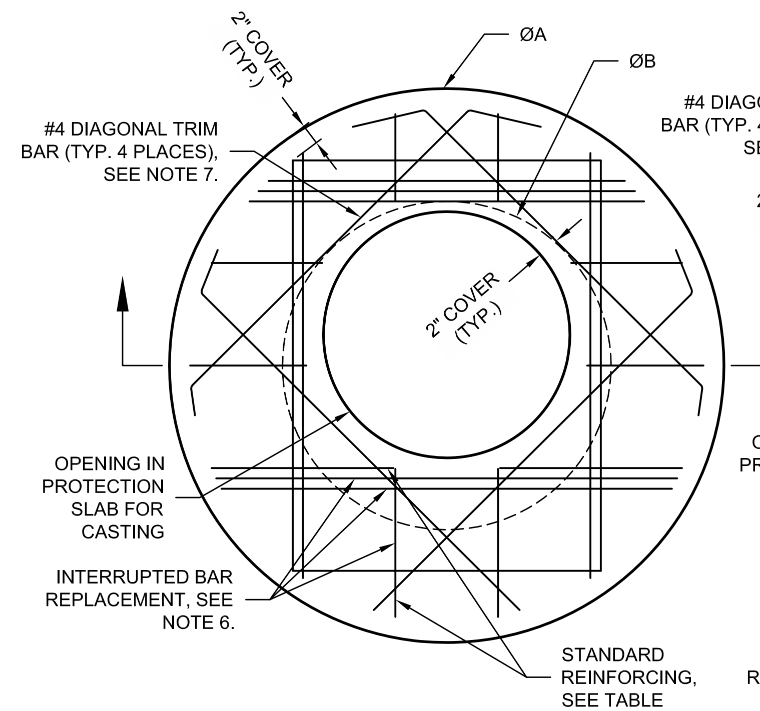


SECTION VIEW

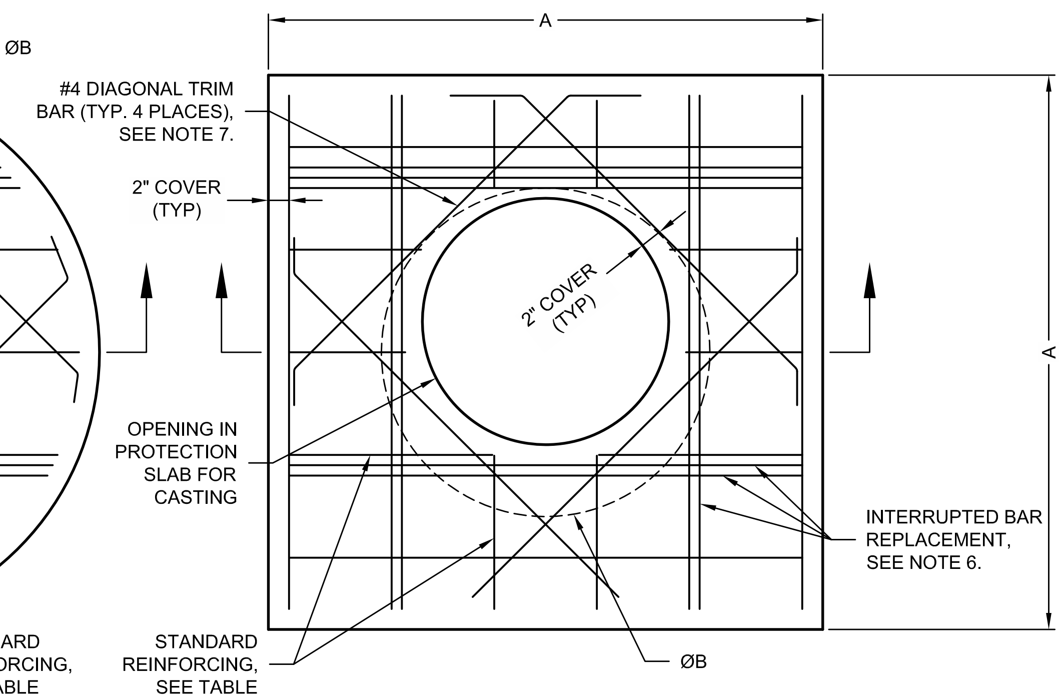
REINFORCING TABLE

Ø CMP RISER	A	Ø B	REINFORCING	**BEARING PRESSURE (PSF)
24"	Ø 4' 4'X4'	26"	#5 @ 12" OCEW #5 @ 12" OCEW	2,410 1,780
30"	Ø 4'-6" 4'-6" X 4'-6"	32"	#5 @ 12" OCEW #5 @ 12" OCEW	2,120 1,530
36"	Ø 5' 5' X 5'	38"	#5 @ 10" OCEW #5 @ 10" OCEW	1,890 1,350
42"	Ø 5'-6" 5'-6" X 5'-6"	44"	#5 @ 10" OCEW #5 @ 9" OCEW	1,720 1,210
48"	Ø 6' 6' X 6'	50"	#5 @ 9" OCEW #5 @ 8" OCEW	1,600 1,100

** ASSUMED SOIL BEARING CAPACITY



ROUND OPTION PLAN VIEW



SQUARE OPTION PLAN VIEW

NOTES:

1. DESIGN IN ACCORDANCE WITH AASHTO, 17th EDITION.
2. DESIGN LOAD HS25.
3. EARTH COVER = 1' MAX.
4. CONCRETE STRENGTH = 3,500 psi
5. REINFORCING STEEL = ASTM A615, GRADE 60.
6. PROVIDE ADDITIONAL REINFORCING AROUND OPENINGS EQUAL TO THE BARS INTERRUPTED, HALF EACH SIDE. ADDITIONAL BARS TO BE IN THE SAME PLANE.
7. TRIM OPENING WITH DIAGONAL #4 BARS, EXTEND BARS A MINIMUM OF 12" BEYOND OPENING, BEND BARS AS REQUIRED TO MAINTAIN BAR COVER.
8. PROTECTION SLAB AND ALL MATERIALS TO BE PROVIDED AND INSTALLED BY CONTRACTOR.
9. DETAIL DESIGN BY DELTA ENGINEERING, BINGHAMTON, NY.

4 / C4 MANHOLE CAP DETAIL

SCALE: N.T.S.

SPECIFICATION FOR CORRUGATED STEEL PIPE-ALUMINIZED TYPE 2 STEEL

SCOPE

THIS SPECIFICATION COVERS THE MANUFACTURE AND INSTALLATION OF THE CORRUGATED STEEL PIPE (CSP) DETAILED IN THE PROJECT PLANS.

MATERIAL

THE ALUMINIZED TYPE 2 STEEL COILS SHALL CONFORM TO THE APPLICABLE REQUIREMENTS OF AASHTO M274 OR ASTM A929.

PIPE

THE CSP SHALL BE MANUFACTURED IN ACCORDANCE WITH THE APPLICABLE REQUIREMENTS OF AASHTO M36 OR ASTM A760. THE PIPE SIZES, GAGES AND CORRUGATIONS SHALL BE AS SHOWN ON THE PROJECT PLANS.

ALL FABRICATION OF THE PRODUCT SHALL OCCUR WITHIN THE UNITED STATES.

HANDLING AND ASSEMBLY

SHALL BE IN ACCORDANCE WITH RECOMMENDATIONS OF THE NATIONAL CORRUGATED STEEL PIPE ASSOCIATION (NCSPA)

INSTALLATION

SHALL BE IN ACCORDANCE WITH AASHTO STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, SECTION 26, DIVISION II OR ASTM A798 AND IN CONFORMANCE WITH THE PROJECT PLANS AND SPECIFICATIONS. IF THERE ARE ANY INCONSISTENCIES OR CONFLICTS THE CONTRACTOR SHOULD DISCUSS AND RESOLVE WITH THE SITE ENGINEER.

IT IS ALWAYS THE RESPONSIBILITY OF THE CONTRACTOR TO FOLLOW OSHA GUIDELINES FOR SAFE PRACTICES.

MARK	DATE	REVISION DESCRIPTION	BY

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CONTECH CMP DETENTION SYSTEMS

CONTECH CONTRACT DRAWING

Ø96" UNDERGROUND DETENTION SYSTEM - 000000-001
SAMPLE PROJECT
ANYTOWN, USA
SITE DESIGNATION: UDS

PROJECT No.:	SEQ. No.:	DATE:
XXXXX	001	1/13/2014

DESIGNED: XXX — DRAWN: RTF

CHECKED: — APPROVED:

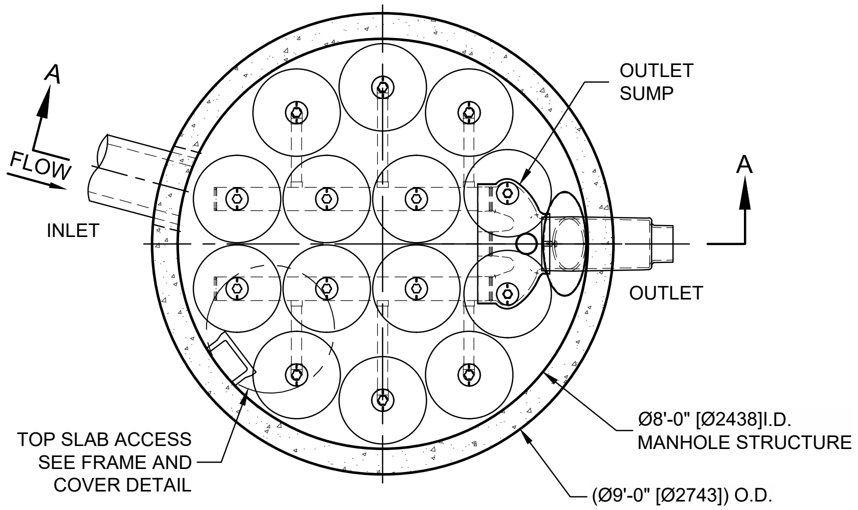
STORMFILTER DESIGN NOTES

STORMFILTER TREATMENT CAPACITY IS A FUNCTION OF THE CARTRIDGE SELECTION AND THE NUMBER OF CARTRIDGES. THE STANDARD MANHOLE STYLE IS SHOWN WITH THE MAXIMUM NUMBER OF CARTRIDGES (14). VOLUME SYSTEM IS ALSO AVAILABLE WITH MAXIMUM 14 CARTRIDGES. Ø8'-0" [2438 mm] MANHOLE STORMFILTER PEAK HYDRAULIC CAPACITY IS 1.8 CFS [51 L/s] . IF THE SITE CONDITIONS EXCEED 1.8 CFS [51 L/s] AN UPSTREAM BYPASS STRUCTURE IS REQUIRED.

CARTRIDGE SELECTION

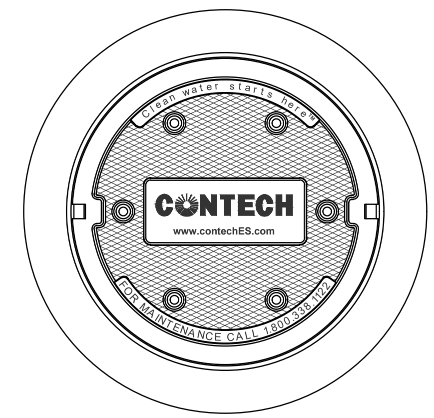
CARTRIDGE HEIGHT	27" [686 mm]			18" [458 mm]			LOW DROP		
RECOMMENDED HYDRAULIC DROP (H)	3.05' [930 mm]			2.3' [700 mm]			1.8' [550 mm]		
SPECIFIC FLOW RATE (gpm/sf) [L/s/m^2]	2 [1.30]	1.67* [1.08]	1 [0.65]	2 [1.30]	1.67* [1.08]	1 [0.65]	2 [1.30]	1.67* [1.08]	1 [0.65]
CARTRIDGE FLOW RATE (gpm) [L/s]	22.5 [1.42]	18.79 [1.19]	11.25 [0.71]	15 [0.95]	12.53 [0.79]	7.5 [0.44]	10 [0.63]	8.35 [0.54]	5 [0.32]

* 1.67 gpm/sf [1.08 L/s/m^2] SPECIFIC FLOW RATE IS APPROVED WITH PHOSPHOSORB® (PSORB) MEDIA ONLY



PLAN VIEW

STANDARD OUTLET RISER
FLOWKIT: 43A



FRAME AND COVER

(DIAMETER VARIES)
N.T.S.

SITE SPECIFIC DATA REQUIREMENTS

STRUCTURE ID	*
WATER QUALITY FLOW RATE (cfs) [L/s]	*
PEAK FLOW RATE (cfs) [L/s]	*
RETURN PERIOD OF PEAK FLOW (yrs)	*
CARTRIDGE HEIGHT (SEE TABLE ABOVE)	*
NUMBER OF CARTRIDGES REQUIRED	*
CARTRIDGE FLOW RATE	*
MEDIA TYPE (PERLITE, ZPG, PSORB)	*

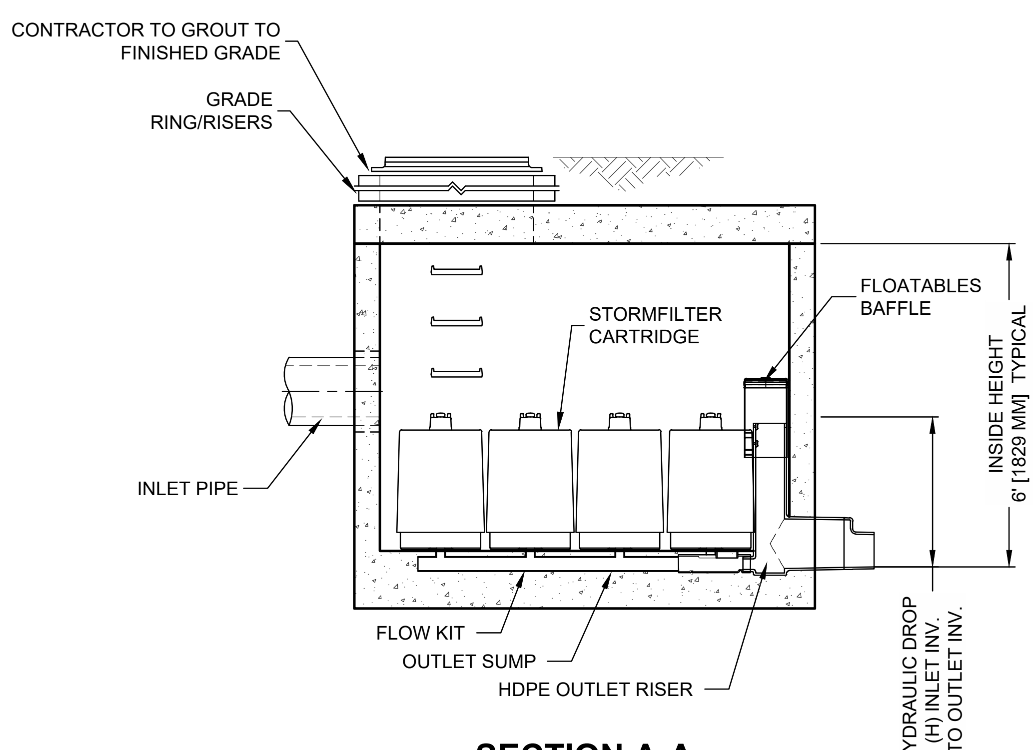
PIPE DATA:	I.E.	MATERIAL	DIAMETER
INLET PIPE #1	*	*	*
INLET PIPE #2	*	*	*
OUTLET PIPE	*	*	*

RIM ELEVATION	*

ANTI-FLOTATION BALLAST	WIDTH	HEIGHT
	*	*

NOTES/SPECIAL REQUIREMENTS:

* PER ENGINEER OF RECORD



SECTION A-A

GENERAL NOTES
1. CONTECH TO PROVIDE ALL MATERIALS UNLESS NOTED OTHERWISE.
2. DIMENSIONS MARKED WITH ( ) ARE REFERENCE DIMENSIONS. ACTUAL DIMENSIONS MAY VARY.
3. FOR SITE SPECIFIC DRAWINGS WITH DETAILED VAULT DIMENSIONS AND WEIGHTS, PLEASE CONTACT YOUR CONTECH ENGINEERED SOLUTIONS LLC REPRESENTATIVE. www.ContechES.com
4. STORMFILTER WATER QUALITY STRUCTURE SHALL BE IN ACCORDANCE WITH ALL DESIGN DATA AND INFORMATION CONTAINED IN THIS DRAWING.
5. STRUCTURE SHALL MEET AASHTO HS-20 LOAD RATING, ASSUMING EARTH COVER OF 0' - 5' [1524 mm] AND GROUNDWATER ELEVATION AT, OR BELOW, THE OUTLET PIPE INVERT ELEVATION. ENGINEER OF RECORD TO CONFIRM ACTUAL GROUNDWATER ELEVATION. CASTINGS SHALL MEET AASHTO M306 AND BE CAST WITH THE CONTECH LOGO.
6. FILTER CARTRIDGES SHALL BE MEDIA-FILLED, PASSIVE, SIPHON ACTUATED, RADIAL FLOW, AND SELF CLEANING. RADIAL MEDIA DEPTH SHALL BE 7-INCHES [178 mm]. FILTER MEDIA CONTACT TIME SHALL BE AT LEAST 38 SECONDS.
7. SPECIFIC FLOW RATE IS EQUAL TO THE FILTER TREATMENT CAPACITY (gpm) [L/s] DIVIDED BY THE FILTER CONTACT SURFACE AREA (sq ft)[m^2].
8. STORMFILTER STRUCTURE SHALL BE PRECAST CONCRETE CONFORMING TO ASTM C-478 AND AASHTO LOAD FACTOR DESIGN METHOD.

INSTALLATION NOTES
A. ANY SUB-BASE, BACKFILL DEPTH, AND/OR ANTI-FLOTATION PROVISIONS ARE SITE-SPECIFIC DESIGN CONSIDERATIONS AND SHALL BE SPECIFIED BY ENGINEER OF RECORD.
B. CONTRACTOR TO PROVIDE EQUIPMENT WITH SUFFICIENT LIFTING AND REACH CAPACITY TO LIFT AND SET THE STORMFILTER STRUCTURE.
C. CONTRACTOR TO INSTALL JOINT SEALANT BETWEEN ALL STRUCTURE SECTIONS AND ASSEMBLE STRUCTURE.
D. CONTRACTOR TO PROVIDE, INSTALL, AND GROUT INLET PIPE(S).
E. CONTRACTOR TO PROVIDE AND INSTALL CONNECTOR TO THE OUTLET RISER STUB. STORMFILTER EQUIPPED WITH A DUAL DIAMETER HDPE OUTLET STUB AND SAND COLLAR. IF OUTLET PIPE IS LARGER THAN 8 INCHES [200 mm], CONTRACTOR TO REMOVE THE 8 INCH [200 mm] OUTLET STUB AT MOLDED-IN CUT LINE. COUPLING BY FERNCO OR EQUAL AND PROVIDED BY CONTRACTOR.
F. CONTRACTOR TO TAKE APPROPRIATE MEASURES TO PROTECT CARTRIDGES FROM CONSTRUCTION-RELATED EROSION RUNOFF.

The Stormwater Management StormFilter®

CONTECH®
ENGINEERED SOLUTIONS LLC
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SFMH96
STORMFILTER
STANDARD DETAIL

Appendix B

Support Calculations

HydroCAD Report

Summary for Subcatchment 1: Pre-developed

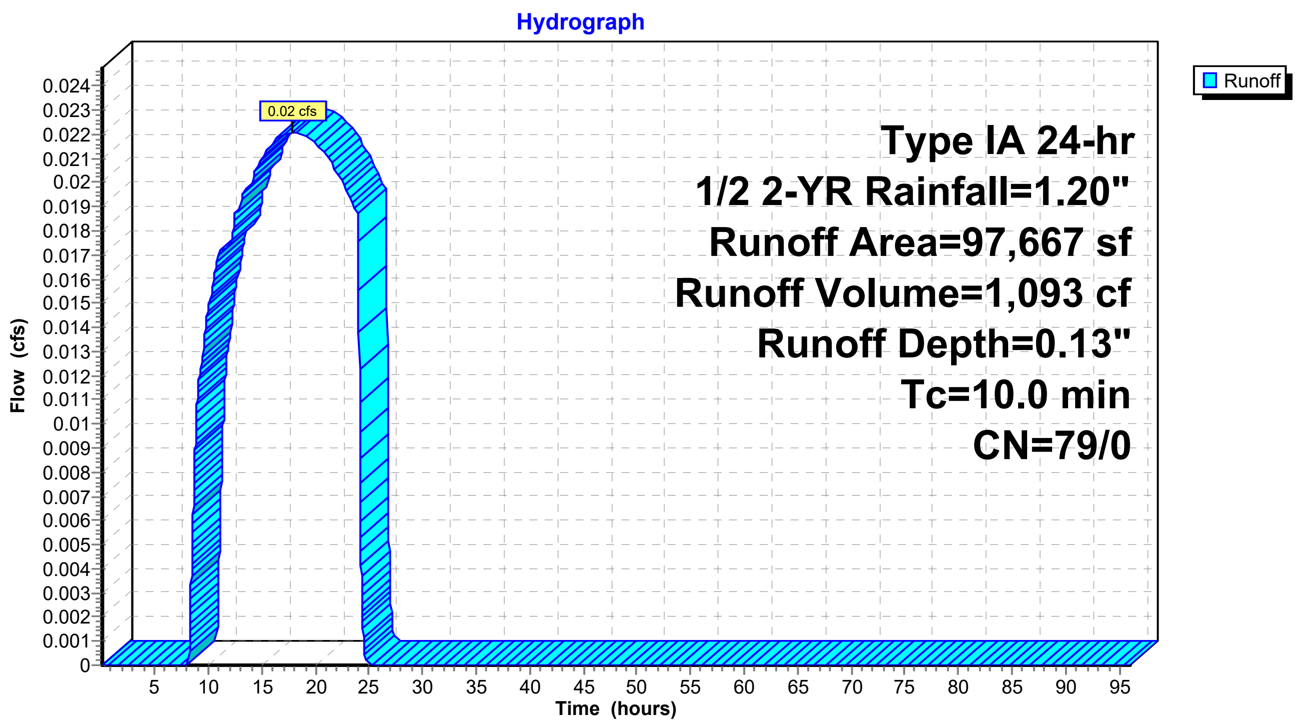
Runoff = 0.02 cfs @ 17.89 hrs, Volume= 1,093 cf, Depth= 0.13"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.10-96.00 hrs, dt= 0.02 hrs
Type IA 24-hr 1/2 2-YR Rainfall=1.20"

	Area (sf)	CN	Description
*	97,667	79	
	97,667	79	100.00% Pervious Area

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

Subcatchment 1: Pre-developed



Summary for Pond 3: Detention

Inflow Area = 97,667 sf,100.00% Impervious, Inflow Depth = 0.99" for 1/2 2-YR event
Inflow = 0.54 cfs @ 7.98 hrs, Volume= 8,022 cf
Outflow = 0.09 cfs @ 13.45 hrs, Volume= 8,022 cf, Atten= 83%, Lag= 328.0 min
Primary = 0.09 cfs @ 13.45 hrs, Volume= 8,022 cf

Routing by Stor-Ind method, Time Span= 0.10-96.00 hrs, dt= 0.02 hrs
Peak Elev= 102.53' @ 13.45 hrs Surf.Area= 1,488 sf Storage= 2,730 cf

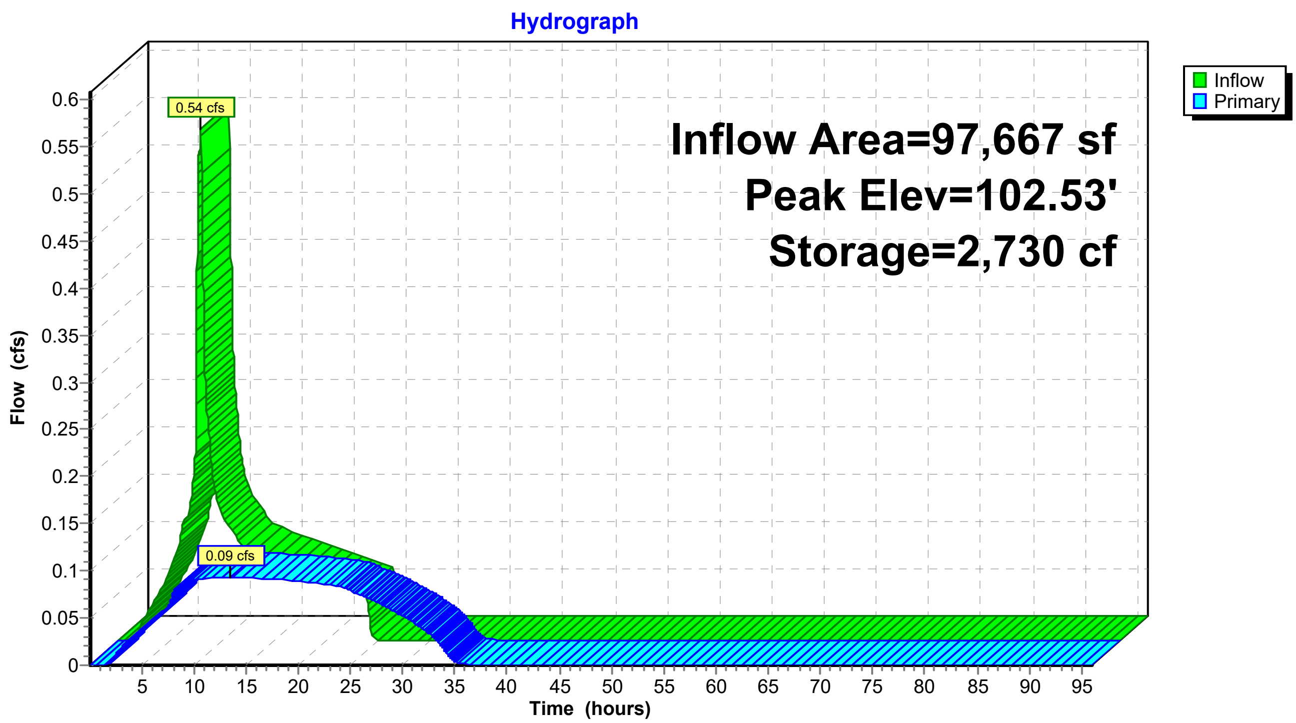
Plug-Flow detention time= 378.7 min calculated for 8,022 cf (100% of inflow)
Center-of-Mass det. time= 378.7 min (1,086.8 - 708.2)

Volume	Invert	Avail.Storage	Storage Description
#1	100.00'	10,053 cf	**96.0" Round Pipe Storage** L= 200.0'

Device	Routing	Invert	Outlet Devices
#1	Primary	100.00'	**1.5" Vert. Orifice/Grate** C= 0.600
#2	Primary	105.65'	**6.0" Vert. Orifice/Grate** C= 0.600
#3	Primary	107.50'	**12.0" Vert. Orifice/Grate** C= 0.600

Primary OutFlow Max=0.09 cfs @ 13.45 hrs HW=102.53' (Free Discharge)
1=Orifice/Grate (Orifice Controls 0.09 cfs @ 7.57 fps)
2=Orifice/Grate (Controls 0.00 cfs)
3=Orifice/Grate (Controls 0.00 cfs)

Pond 3: Detention



Summary for Subcatchment 1: Pre-developed

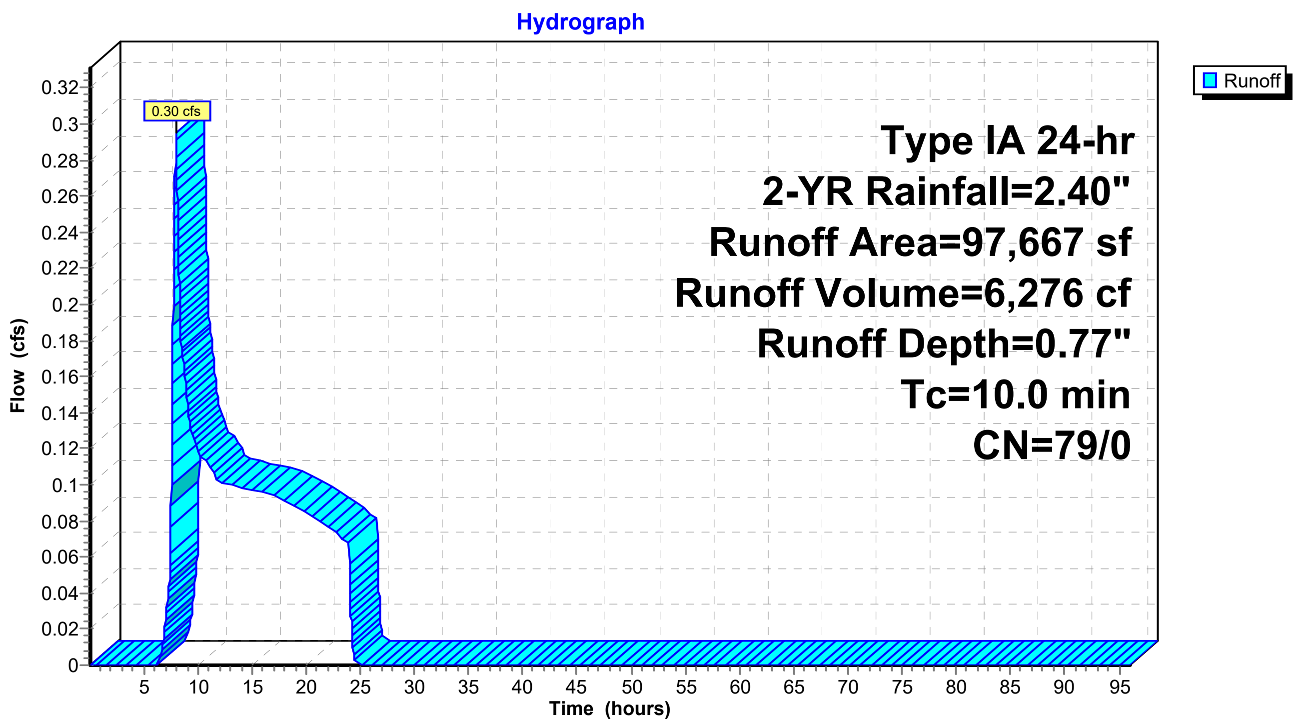
Runoff = 0.30 cfs @ 8.01 hrs, Volume= 6,276 cf, Depth= 0.77"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.10-96.00 hrs, dt= 0.02 hrs
Type IA 24-hr 2-YR Rainfall=2.40"

	Area (sf)	CN	Description
*	97,667	79	
	97,667	79	100.00% Pervious Area

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

Subcatchment 1: Pre-developed



Summary for Pond 3: Detention

Inflow Area = 97,667 sf,100.00% Impervious, Inflow Depth = 2.17" for 2-YR event
Inflow = 1.17 cfs @ 7.97 hrs, Volume= 17,672 cf
Outflow = 0.19 cfs @ 13.48 hrs, Volume= 17,672 cf, Atten= 84%, Lag= 330.8 min
Primary = 0.19 cfs @ 13.48 hrs, Volume= 17,672 cf

Routing by Stor-Ind method, Time Span= 0.10-96.00 hrs, dt= 0.02 hrs
Peak Elev= 105.78' @ 13.48 hrs Surf.Area= 1,433 sf Storage= 7,776 cf

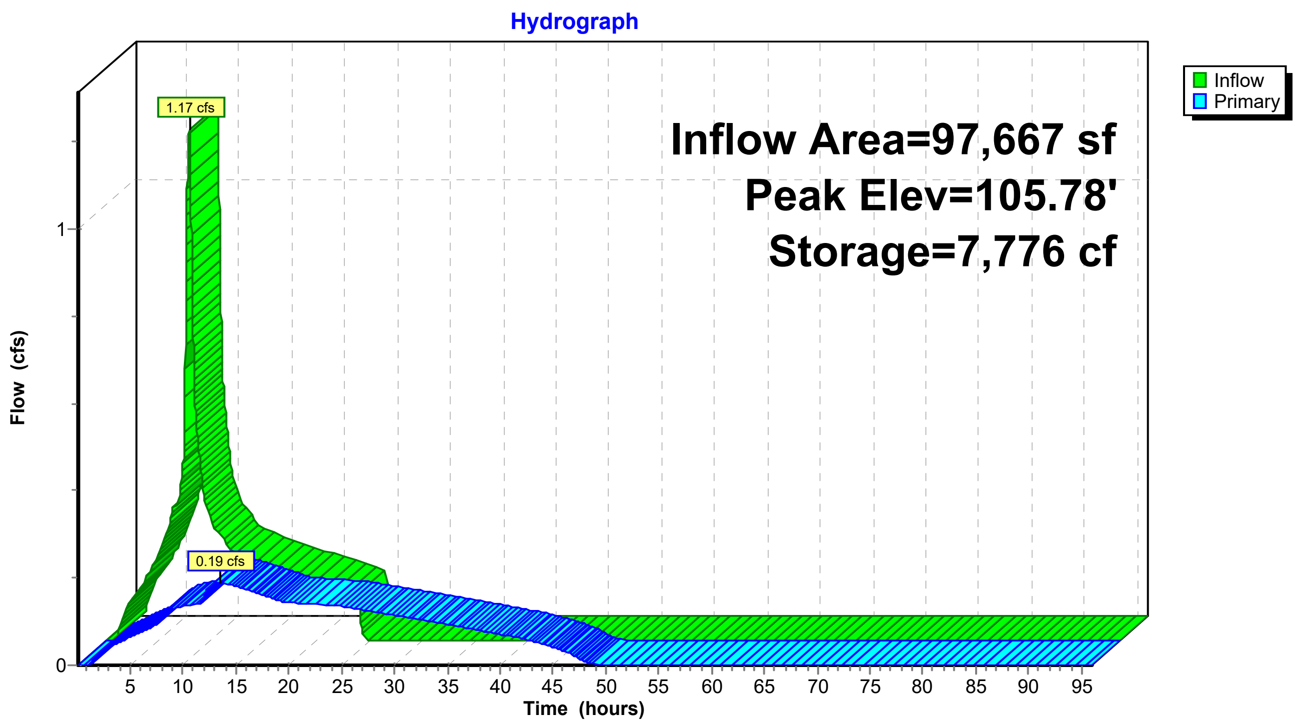
Plug-Flow detention time= 663.9 min calculated for 17,672 cf (100% of inflow)
Center-of-Mass det. time= 663.8 min (1,343.1 - 679.3)

Volume	Invert	Avail.Storage	Storage Description
#1	100.00'	10,053 cf	**96.0" Round Pipe Storage** L= 200.0'

Device	Routing	Invert	Outlet Devices
#1	Primary	100.00'	**1.5" Vert. Orifice/Grate** C= 0.600
#2	Primary	105.65'	**6.0" Vert. Orifice/Grate** C= 0.600
#3	Primary	107.50'	**12.0" Vert. Orifice/Grate** C= 0.600

Primary OutFlow Max=0.19 cfs @ 13.48 hrs HW=105.78' (Free Discharge)
1=Orifice/Grate (Orifice Controls 0.14 cfs @ 11.51 fps)
2=Orifice/Grate (Orifice Controls 0.05 cfs @ 1.22 fps)
3=Orifice/Grate (Controls 0.00 cfs)

Pond 3: Detention



Summary for Subcatchment 1: Pre-developed

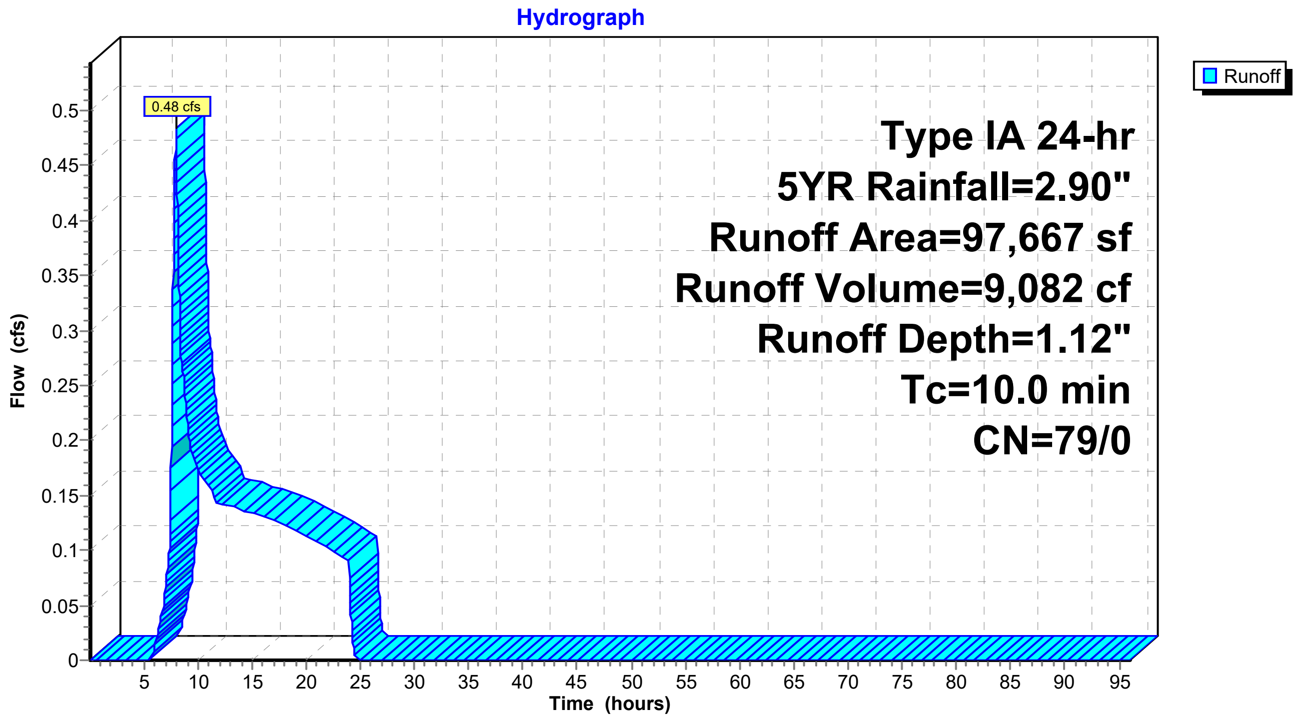
Runoff = 0.48 cfs @ 8.00 hrs, Volume= 9,082 cf, Depth= 1.12"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.10-96.00 hrs, dt= 0.02 hrs
Type IA 24-hr 5YR Rainfall=2.90"

	Area (sf)	CN	Description
*	97,667	79	
	97,667	79	100.00% Pervious Area

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

Subcatchment 1: Pre-developed



Summary for Pond 3: Detention

Inflow Area = 97,667 sf,100.00% Impervious, Inflow Depth = 2.67" for 5YR event
Inflow = 1.43 cfs @ 7.97 hrs, Volume= 21,720 cf
Outflow = 0.40 cfs @ 9.39 hrs, Volume= 21,720 cf, Atten= 72%, Lag= 85.1 min
Primary = 0.40 cfs @ 9.39 hrs, Volume= 21,720 cf

Routing by Stor-Ind method, Time Span= 0.10-96.00 hrs, dt= 0.02 hrs
Peak Elev= 105.97' @ 9.39 hrs Surf.Area= 1,393 sf Storage= 8,042 cf

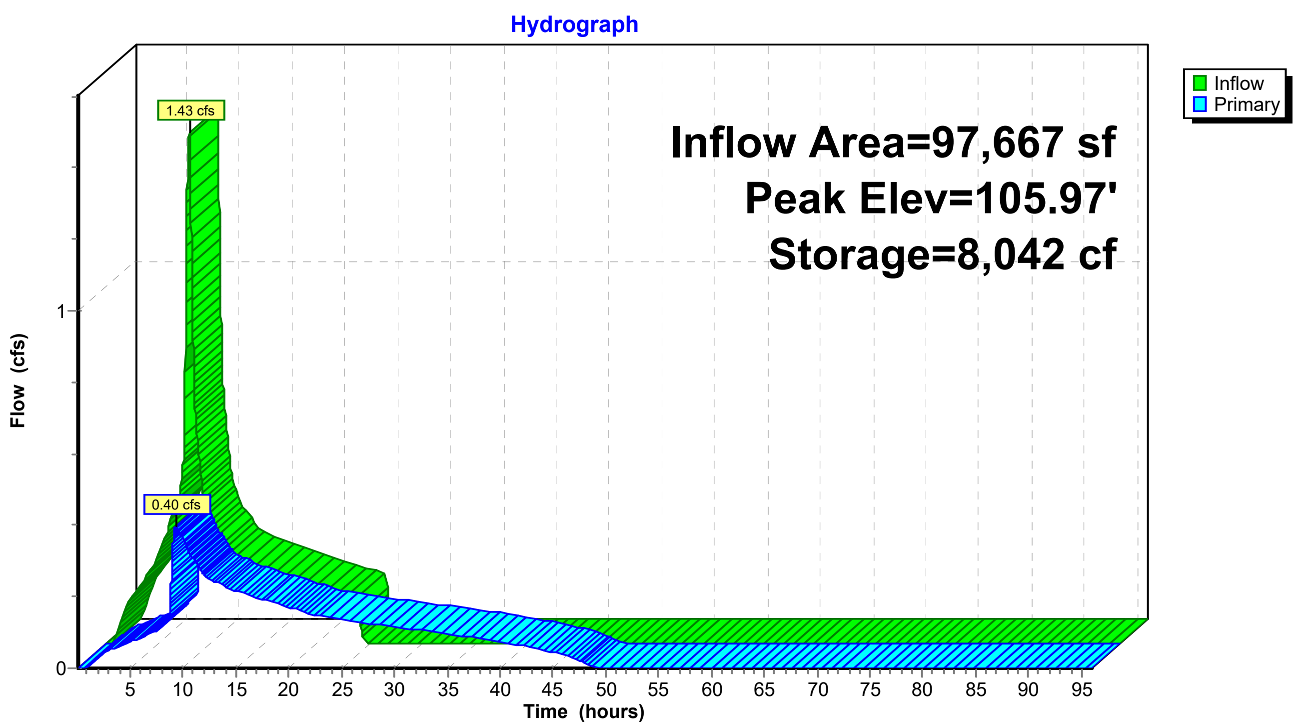
Plug-Flow detention time= 573.7 min calculated for 21,716 cf (100% of inflow)
Center-of-Mass det. time= 573.9 min (1,247.3 - 673.3)

Volume	Invert	Avail.Storage	Storage Description
#1	100.00'	10,053 cf	**96.0" Round Pipe Storage** L= 200.0'

Device	Routing	Invert	Outlet Devices
#1	Primary	100.00'	**1.5" Vert. Orifice/Grate** C= 0.600
#2	Primary	105.65'	**6.0" Vert. Orifice/Grate** C= 0.600
#3	Primary	107.50'	**12.0" Vert. Orifice/Grate** C= 0.600

Primary OutFlow Max=0.40 cfs @ 9.39 hrs HW=105.97' (Free Discharge)
├─**1=Orifice/Grate** (Orifice Controls 0.14 cfs @ 11.70 fps)
├─**2=Orifice/Grate** (Orifice Controls 0.25 cfs @ 1.92 fps)
└─**3=Orifice/Grate** (Controls 0.00 cfs)

Pond 3: Detention



Summary for Subcatchment 1: Pre-developed

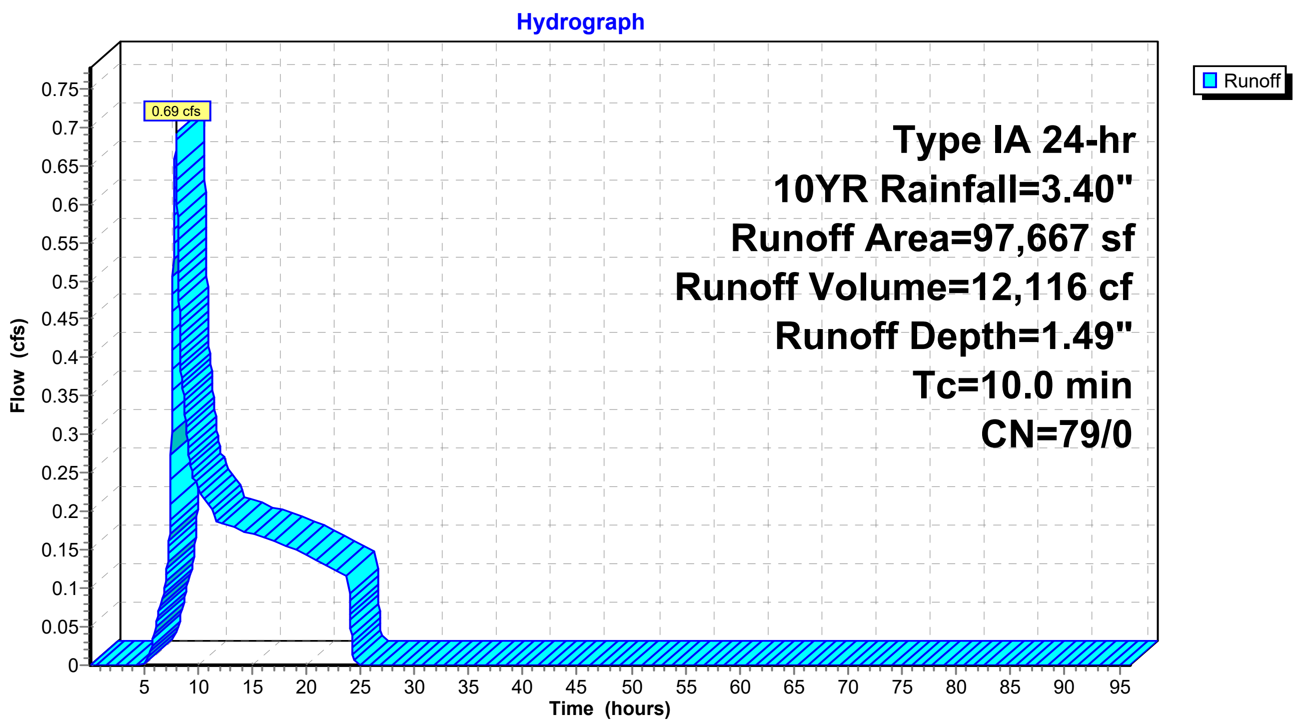
Runoff = 0.69 cfs @ 8.00 hrs, Volume= 12,116 cf, Depth= 1.49"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.10-96.00 hrs, dt= 0.02 hrs
Type IA 24-hr 10YR Rainfall=3.40"

	Area (sf)	CN	Description
*	97,667	79	
	97,667	79	100.00% Pervious Area

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

Subcatchment 1: Pre-developed



Summary for Pond 3: Detention

Inflow Area = 97,667 sf,100.00% Impervious, Inflow Depth = 3.17" for 10YR event
Inflow = 1.69 cfs @ 7.97 hrs, Volume= 25,774 cf
Outflow = 0.70 cfs @ 8.68 hrs, Volume= 25,774 cf, Atten= 59%, Lag= 42.8 min
Primary = 0.70 cfs @ 8.68 hrs, Volume= 25,774 cf

Routing by Stor-Ind method, Time Span= 0.10-96.00 hrs, dt= 0.02 hrs
Peak Elev= 106.24' @ 8.68 hrs Surf.Area= 1,324 sf Storage= 8,419 cf

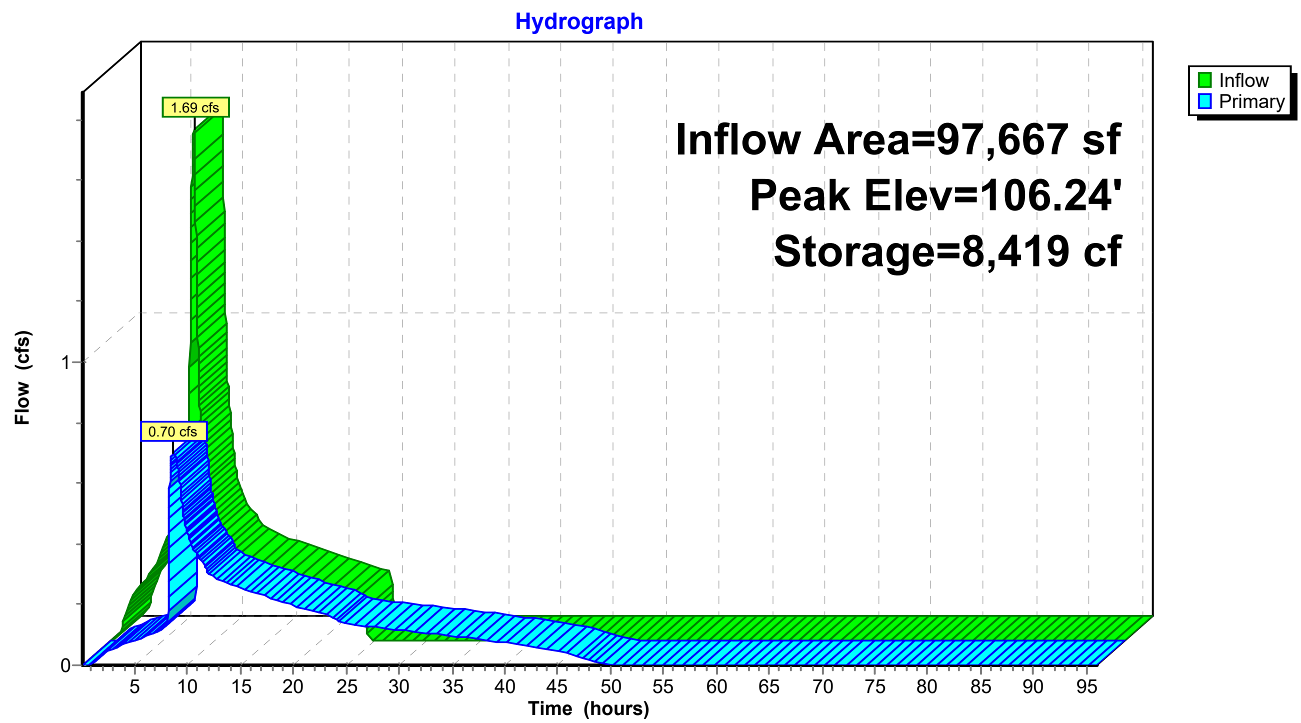
Plug-Flow detention time= 498.8 min calculated for 25,769 cf (100% of inflow)
Center-of-Mass det. time= 499.0 min (1,167.9 - 668.9)

Volume	Invert	Avail.Storage	Storage Description
#1	100.00'	10,053 cf	**96.0" Round Pipe Storage** L= 200.0'

Device	Routing	Invert	Outlet Devices
#1	Primary	100.00'	**1.5" Vert. Orifice/Grate** C= 0.600
#2	Primary	105.65'	**6.0" Vert. Orifice/Grate** C= 0.600
#3	Primary	107.50'	**12.0" Vert. Orifice/Grate** C= 0.600

Primary OutFlow Max=0.70 cfs @ 8.68 hrs HW=106.24' (Free Discharge)
1=Orifice/Grate (Orifice Controls 0.15 cfs @ 11.97 fps)
2=Orifice/Grate (Orifice Controls 0.56 cfs @ 2.83 fps)
3=Orifice/Grate (Controls 0.00 cfs)

Pond 3: Detention



Summary for Subcatchment 1: Pre-developed

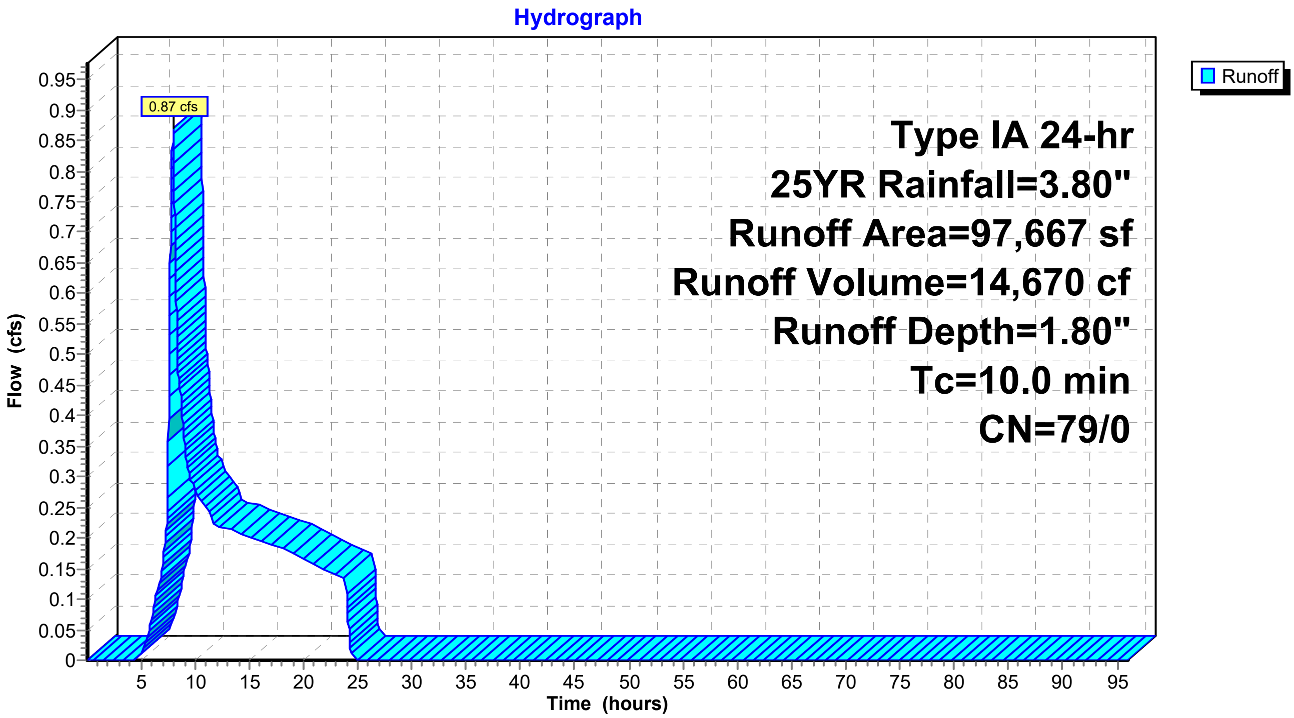
Runoff = 0.87 cfs @ 8.00 hrs, Volume= 14,670 cf, Depth= 1.80"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.10-96.00 hrs, dt= 0.02 hrs
Type IA 24-hr 25YR Rainfall=3.80"

	Area (sf)	CN	Description
*	97,667	79	
	97,667	79	100.00% Pervious Area

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

Subcatchment 1: Pre-developed



Summary for Pond 3: Detention

Inflow Area = 97,667 sf,100.00% Impervious, Inflow Depth = 3.57" for 25YR event
Inflow = 1.90 cfs @ 7.97 hrs, Volume= 29,020 cf
Outflow = 0.96 cfs @ 8.42 hrs, Volume= 29,020 cf, Atten= 49%, Lag= 27.4 min
Primary = 0.96 cfs @ 8.42 hrs, Volume= 29,020 cf

Routing by Stor-Ind method, Time Span= 0.10-96.00 hrs, dt= 0.02 hrs
Peak Elev= 106.64' @ 8.42 hrs Surf.Area= 1,203 sf Storage= 8,916 cf

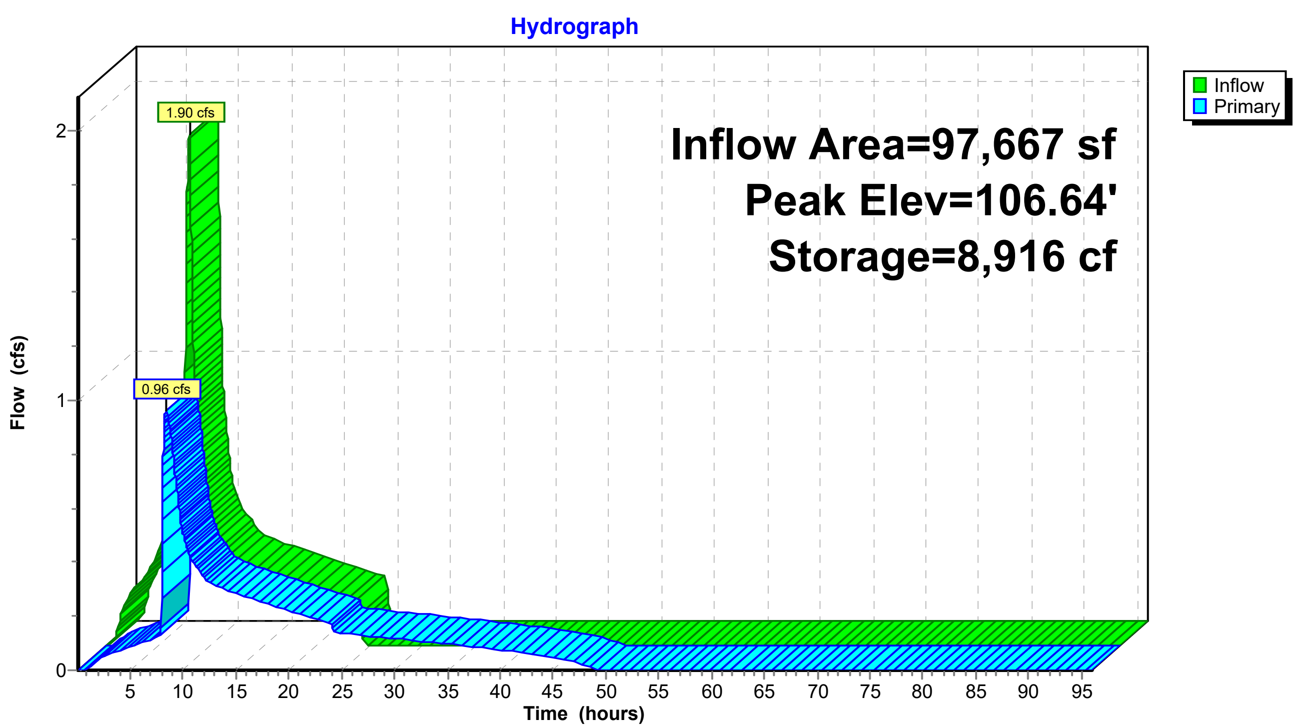
Plug-Flow detention time= 452.7 min calculated for 29,020 cf (100% of inflow)
Center-of-Mass det. time= 452.6 min (1,118.7 - 666.1)

Volume	Invert	Avail.Storage	Storage Description
#1	100.00'	10,053 cf	**96.0" Round Pipe Storage** L= 200.0'

Device	Routing	Invert	Outlet Devices
#1	Primary	100.00'	**1.5" Vert. Orifice/Grate** C= 0.600
#2	Primary	105.65'	**6.0" Vert. Orifice/Grate** C= 0.600
#3	Primary	107.50'	**12.0" Vert. Orifice/Grate** C= 0.600

Primary OutFlow Max=0.96 cfs @ 8.42 hrs HW=106.64' (Free Discharge)
1=Orifice/Grate (Orifice Controls 0.15 cfs @ 12.35 fps)
2=Orifice/Grate (Orifice Controls 0.81 cfs @ 4.13 fps)
3=Orifice/Grate (Controls 0.00 cfs)

Pond 3: Detention



Pali Consulting

June 12, 2023

Lango Hansen Landscape Architects
Attn: Kurt Lango, Brian Martin
1100 NW Glisan St #3A
Portland, OR 97209

Report of Geotechnical Services
Sandy Community Campus Park Project
Sandy, Oregon
Project #163-22-002

1.0 INTRODUCTION

Pali Consulting, Inc. (Pali Consulting) presents this report of geotechnical services for the Sandy Community Campus Park Project (Project), located west of the intersection between SE Meinig Avenue and Scenic Street, in Sandy, Oregon. The site is an approximately 7-acre parcel and developed with two athletic fields, an East Field and a West Field, a running track around the West Field, a Skate Park, and street adjacent parking. The location of the site is shown on Figure 1. The current site layout and pertinent features are shown on Figure 2.

Lango Hansen Landscape Architects (Lango Hansen) are designing improvements to the park, which may include a prefabricated lightweight entrance structure, infiltration facilities, and new pavements. Lango Hansen requested that we provide geotechnical design services for the improvements. Our scope of work included reviewing background information, completing drilled borings at locations identified by Lango Hansen, conducting infiltration testing, and completing laboratory tests on select samples, and preparation of this report. Our work was completed in general accordance with our agreement with Lango Hansen, dated December 9, 2022.

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2.0 BACKGROUND REVIEW

2.1 GEOLOGY

The geology in the area is mapped on the Oregon Department of Geology and Mineral Industries' (DOGAMI) website (https://gis.dogami.oregon.gov/maps/geologicmap/#, accessed May 2023). The website maps the parcel within mixed-lithology Troutdale Formation. This formation consists of Miocene to Pleistocene-aged fluvial mudstone, sandstone, and conglomerate, as well as older fluvial terraces.

2.2 GEOLOGIC HAZARDS

Geologic hazards were reviewed using DOGAMI's Statewide Geohazards Viewer (HAZVU) (https://gis.dogami.oregon.gov/maps/hazvu/, accessed June 2023). Geologic hazards mapped at the site include landslides and shaking from Cascadia and local earthquakes. Mapped landslide hazard is low to moderate at the site, but hazard mapping quicky increases from moderate to very high locally where a mapped landslide is present about 60 feet northwest of the outer northwest corner of the track. The mapped landslide is shown on Figure 3. This mapped landslide is about 30 acres in area and has an arcuate headscarp which extends to the north and west of the park and a body extending away from the park to the northwest. Data from DOGAMI indicates that the landside is deep-seated, with an approximate failure depth of 50 feet, a headscarp height of 55 feet, and a complex movement classification. The landslide is pre-historic in age (>150 years) and is described and mapped with moderate certainty. In addition to landslide hazards, very strong earthquake shaking from Cascadia and local earthquakes is also mapped as a hazard at the site.

2.3 WELL LOGS

We reviewed well logs near the site on the Oregon Water Resources Department website (https://apps.wrd.state.or.us/apps/gw/well_log/, accessed May 2023). Logs reviewed adjacent to the site indicated primarily clay or silty clay soils to depths of 25 to 50 feet below the ground surface (bgs) overlying Troutdale Formation bedrock. Nearby well logs reported zones of perched groundwater as shallow as 6 feet bgs, indicating that multiple zones of groundwater may be present.

2.4 GROUNDWATER MAPPING

We reviewed groundwater mapping of the area completed by the United States Geological Survey (USGS) website (https://or.water.usgs.gov/projs_dir/puz/index.html, accessed May 2023). The mapping shows estimated depths to regional groundwater of about 50 feet bgs.

2.5 SOILS MAPPING

We reviewed soils mapped at the site on the Natural Resource Conservation Service (NRCS) Web Soil Survey website (https://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx, accessed May 2023). The soil mapping shows three soils mapped at the site: Cazadero silty clay loam (0 to 7 percent slopes), Cazadero silty clay loam (7 to 12 percent slopes) and Dystrochrepts (very steep). Cazadero silty clay loam (0 to 7 percent slopes) and Cazadero silty clay loam (7 to 12 percent slopes) together cover the northernmost 85% of the site. These soils have a parent material of old mixed alluvium and are typically found on terraces. Typical profiles consist of silty clay loam from 0 to 21 inches and clay from 21 to 75 inches. Typical depths to both the water table and a restrictive feature are more than 80 inches. Soils are further described as being well drained with a moderately high capacity of the most limiting layer to transmit water (0.20 to 0.57 in/hr). Dystrochrepts (very steep) is mapped in the southernmost 15% of the site. This soil has a parent material of colluvium derived from andesite and basalt and is typically found on terraces. A typical profile consists of gravelly loam from 0 to 8 inches, very gravelly loam from 8 to 44 inches, and unweathered bedrock from 44 to 48 inches. Depths to the water table range from 36 to 72 inches, and depth to a restrictive feature is about 40 to 60 inches to lithic bedrock. This soil is further described as being well drained with a moderately high to high capacity of the most limiting layer to transmit water (0.20 to 1.98 in/hr).

Fill is not mapped at the site, but based on site grades and our geotechnical explorations, described later in this report, grading has occurred which has included fills and modifications to the natural soils.

2.6 AERIAL PHOTOGRAPHS & CONSTRUCTION PLANS

We reviewed historic aerial photographs from the years 1995 through 2023 available on Google Earth Pro©, and from the years 1952, 1956, 1970, and 1986 available through USGS Earth Explorer. We also reviewed as-built plans provided by Lango Hansen.

2.6.1 Development History

Our review of the aerial photographs found that the site was forested at the time of the earliest air photo in 1952. In the 1952 photo, Scenic Street appears to extend westward of its modern terminus and leads to a tear-shaped cleared area within the trees which is likely a landfill, based on anecdotal reports. By the time of the 1956 photo, most of the trees had been cleared from the park area with a few scattered patches of vegetation remaining on the south and east sides. Between the 1956 photo and the next photo in 1970, the park was constructed and consisted of two mowed grass fields separated by a short steep slope, with a running track on the lower field. Vegetation to the northeast of the park is cleared in the 1970 photo and gradually fills in over the next air photo years to the current condition. Between the 1995 and 2000 air photos, the Skate Park located in the southeast corner of the park was built. In air photos taken from 1970 to present, grading and development at the site appears consistent with what is present today.

2.6.2 Landforms

Because of the nearby mapped landslide, we also reviewed the aerial photographs for signs of slope instability and related landforms. The 1952 air photo shows two irregularly shaped cleared areas in the vicinity of the park area. The first, located west of the park, is likely the landfill noted in the section above. The second cleared area is smaller and located at the terminus of modern-day Scenic Street, to the north of the park. This could be a second landfill, or a cleared and graded area intended for development or other use. These two areas remain visible in the 1956 air photo, and much of the land to the south and east of them (future Sandy Park) is cleared of vegetation. At the time of the next air photo, in 1970, the west (landfill) cleared area is no longer visible, as it has apparently revegetated. The north cleared area, however, appears to be incorporated into a broader cleared area extending down to Scenic Street. An arcuate landform is visible in the 1970 photo at approximately the same location as the mapped scarp of the landslide discussed in *Section 2.2*. This landform is mostly bare, with some scattered vegetation. Downslope (northwest) of the scarp, vegetation consists mostly of forested land with some small bare areas which may indicate ground disturbance. Vegetation appears younger on the east side of the mapped landslide body, but it is not clear whether this is due to die-off caused by ground movement or harvest which occurred between air photo years. There is a triangular patch of bare ground extending northwest from about the middle of the visible scarp which may indicate an area of greater localized instability. The 1986 air photo shows revegetation of the mapped scarp and body areas, with only a small bare area visible at the location of the triangular bare ground in the 1970 photo. Air photos dating from between 1995 and 2023 do not show further evidence of disturbance to these features.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The site consists of a 7-acre parcel bound to the northeast by Scenic Street, to the east by SE Meinig Avenue, to the south by a short private road leading to an adjacent commercial development (the SandyNet facility), and to the west and northwest by forested land. The Sandy Skate Park is located in the southeast corner of the property. The bulk of the site is developed with two grass-covered fields, the East Field and West Field, which are separated by a short steep slope. The West Field contains a running track and the ground within the track varies in elevation from the track, raised up to a few feet in some locations and lower than the track in others. A drainage ditch parallels the inside edge of the track and inlet grates are visible within the ditch. Parking for the park consists of off-street parking abutting Meinig Avenue near the Skate Park. Access to the park is via a short paved ramp from parking area. There is also a narrow paved access road which runs down to the West Field from the SandyNet facility.

West of the West Field track, flat ground continues to an area which is heavily wooded. This area is believed to be the former landfill area.

Elevations at the site range from 940 feet MSL in the northwest corner of the site adjacent the skate park to about 900 feet MSL at its westernmost point.

3.2 SUBSURFACE CONDITIONS

We completed three machine-drilled borings, designated B-1 through B-3, to depths ranging between approximately 21.5 feet to 26.5 feet bgs. Infiltration testing was completed adjacent to two of the borings, Borings B-1 and B-2, with test designations IT-1 and IT-2, respectively. IT-1 was completed at a depth of 5 feet bgs and IT-2 was completed at a depth of 15 feet bgs. The approximate locations of our explorations and infiltration tests are shown on Figure 2.

Our site explorations and testing were completed on May 20th, 2023. Descriptions and logs of our subsurface explorations are included in Appendix A. Infiltration testing is described in Appendix A and the results are discussed in *Section 4.0.*

Our site explorations encountered a thin layer of topsoil in all borings, overlying about 5 feet of fill in Borings B-1 and B-3. Beneath the topsoil or fill, we encountered native silt and clay soils to 26.5 feet bgs, the maximum depth of explorations. These units are described in more detail below.

3.2.1 Topsoil

Our explorations encountered moist brown silty topsoil up to 6 inches deep across the site. The topsoil contained a variable root zone/organics which extended to about 4 inches depth. No topsoil samples were collected, and it is not noted on the logs in Appendix A, except the thickness of a root mass where encountered.

3.2.2 Silt Fill

Underlying the topsoil, our explorations encountered up to 5 feet of silt soil we interpret as fill in two of the borings, Boring B-1 and Boring B-3. The fill in Boring B-1 appears to be from raising the field within the track to allow for drainage to a drainage ditch paralleling the inside edge of the track. The fill in Boring B-3 appears to be from general grading for the field. The fill was generally brown with black, red, and grey mottling, and was characterized by a blocky appearance, which was used to distinguish it from similar native soils. The fill was found to be medium stiff based on SPT blow counts (N-values) of 4 to 7 in the borings completed, with an average of 6.

Laboratory testing on samples from the fill found moisture contents ranging from 33 to 37 percent. The plasticity of the fill was interpreted as low in B-1 to high in B-3, based on Atterberg limits testing, which measured plasticity indices (PI's) of 13 to 28, resulting in a USCS classification of ML to MH.

3.2.3 Native Silt

In the West Field we encountered native silt below the topsoil or fill that extended to 26.5 feet bgs, the maximum depth of exploration. This native silt was varicolored, contained small amounts of sand and gravel, and was moist to wet. Mottling of the soils was generally noted at all depths. The silt varied from medium stiff to very stiff, based on N-values that ranged from 5 to 20 in the borings completed, with an average of 11.

Laboratory testing found moisture contents ranging from 31 to 62 percent. The plasticity of the silt was interpreted as low to moderate, based on Atterberg limits testing, which measured a PI of 21 in one sample tested, resulting in a USCS classification of MH. A second sample tested was found to be non-plastic. The silt contained varying amounts of sand and gravel ranging from 7 to 11 percent in the samples tested.

3.2.3 Native Clay

In the east field we encountered native clay below the fill that extended to 21.5 feet bgs, the maximum depth of exploration. This native clay was brown-red to grey, contained small amounts of sand, gravel, wood, and other organic material, and was moist at all depths. Slight mottling of the clay was noted beginning at about 15 feet bgs. The clay varied from soft to stiff, based on N-values that ranged from 4 to 14, with an average of 9.

Laboratory testing found moisture contents ranging from 34 to 57 percent. The plasticity of the clay was interpreted as moderate, based on Atterberg limits testing, which measured a PI of 22 in one sample tested, resulting in a USCS classification of CL. It was noted in the field that plasticity of the clay generally increased with depth. The clay contained about 12 percent sand and gravel, based on one sample tested.

3.2.4 Groundwater

Groundwater was encountered in Borings B-1 and B-2 at depths of 20.3 and 22.8 ft bgs, respectively. These were likely perched zones of groundwater, based on USGS regional groundwater mapping and local water well logs. These perched zones are likely variable and higher during the wet season. We estimate that seasonal high groundwater and/or intermittent saturation occurs within about 15 feet or less of the ground surface during the rainy season. This is based on NRCS soil descriptions, soil mottling we observed, moisture content determined in our laboratory tests, and standing water observed at the site during our site explorations.

We note that groundwater elevations can vary from those encountered and interpreted due to the time of year, precipitation, and other factors.

4.0 INFILTRATION TESTING

We completed infiltration tests at two locations within the West Field. IT-1 was completed at a depth of 5 feet bgs and IT-2 was completed at a depth of 15 feet bgs. The tests were completed on May 20th, 2023, at the approximate locations shown on Figure 2. The tests were completed as described in Appendix A of this report. We measured the results documented in Table 1 below during our field infiltration tests.

Table 1. Field-Measured Infiltration Rates

Location	Unfactored Rate	Soil Type	Notes
B-1	1.1 in/hr	ML (fill)	Measured over a 2-hour period following a 1-hour soaking period.
B-2	0.2 in/hr	ML (native)	Measured over a 2-hour period following a 1-hour soaking period.

As indicated in Table 1, the measured field infiltration rate is moderate to low at Boring B-1 (IT-1) and negligible at Boring B-2 (IT-2). Conclusions regarding the application of the field infiltration rates are provided in *Section 5.0*.

5.0 CONCLUSIONS

Based on our explorations, testing, and analyses, it is our opinion that the proposed improvements are feasible from a geotechnical perspective, provided the recommendations in this report are included in design and construction. We offer the following general summary of our conclusions:

- The site is adjacent a mapped deep-seated landslide which is considered pre-historic, but exhibits possible indications within the photo record. The stability of the landslide was not determined so development of the park should consider the risk of future movement of this landform. Such considerations should, at a minimum, include precluding or minimizing fills on the West Field and directing stormwater away from the mapped landslide.
- The site is underlain by fill locally and native soils throughout that are predominately high to low plasticity silt in the west field and clay in the east field. These soils continue to depths of at least 26.5 feet bgs.
- Perched groundwater is expected to be present at variable depths throughout much of the year and within the upper 15 feet bgs during wetter periods of the year. Regional groundwater is expected to be at about 50 feet bgs, as mapped.
- Soils have very low permeability across the site and to the depths explored. The low permeability of site soils make on-site stormwater infiltration unlikely.
- Excavation and handling of site soils should be readily accomplished with conventional earthwork equipment in good working condition. However, the fine-grained soils are moisture-sensitive and will be easily disturbed (e.g., rutted, pumped, etc.) by construction activities during wet weather if special measures are not taken to reduce disturbance.
- Soils at the site are generally medium stiff or better and, based on the measured N-values, exhibited a relatively uniform stiffness across the site, including in areas of fill. Such soils should be capable of supporting anticipated structures and infrastructure, although areas of fill have the potential to include areas of soft or unsuitable soils which are difficult to predict. Construction records confirming compaction of the fill were not located, but based on the uniform material type, soil consistency, and lack of deleterious materials, the fill appears to have been placed as structural fill in areas of our explorations. The on-site fill is expected to be able to support the improvements suitably but should be further evaluated during construction.
- The use of shallow foundations are suitable for lightly loaded structures.
- Pavements should follow the recommendations in this report.

6.0 EARTHWORK RECOMMENDATIONS

We understand that grading for the site will be limited to cuts and fills of less than about 4 feet. All earthwork activities should be conducted in general accordance with Appendix J of the Oregon Structural Specialty Code (OSSC), City of Sandy (City) Municipal Code, and the Oregon Department of Transportation (ODOT) Standard Specifications for Construction (SSC), and the recommendations that follow.

Due to the presence of the mapped deep-seated landslide, additional fill should not be placed within a distance of at least 110 feet of the mapped landslide headscarp (2 times the mapped headscarp height) without more detailed analysis. The approximate location of this line is shown on Figure 3.

Due to the presence of moisture-sensitive soils, subgrade preparation should be limited to the dry season, typically June through September, and follow the recommendations in *Section 6.2* related to wet weather conditions.

6.1 Site Preparation

Initial site preparation will include demolition of existing facilities where present, followed by clearing, stripping and excavating to grade in areas of improvements. Demolition should include removal of existing structures, improvements, and uncontrolled fill to the full extent they occur. Where piping is present, it should be fully removed, or grouted full if abandoned in place. Excavations and areas below grade resulting from demolition should be backfilled with structural fill as described later in this report.

In unimproved aeras, clearing and stripping should extend approximately 5 feet laterally beyond areas of improvements, as needed for equipment access. Pathways should be stripped at least 2 foot wider than the pathway or the minimum necessary to prepare the subgrade per *Section 6.3*, whichever is greater. Based on our explorations, the average depth of stripping will be approximately 6 inches, although greater stripping depths may be required to remove localized zones of loose or organic soil or in areas of the site which were not explored. Actual stripping depths should be evaluated based on observations during the stripping operation. Stripped materials should be hauled off-site or stockpiled for later use as landscaping material.

6.2 Soft Soil/Wet Soil/Wet Weather Construction

The existing surface soils are fine-grained and will be susceptible to disturbance (e.g., pumping and rutting) during periods of wet weather or when the moisture content of the material is more than a few percentage points above optimum. This may be the case during much of the year, but especially in late fall through spring. When wet, the on-site soils are susceptible to disturbance and generally will provide inadequate support for construction equipment. As such, we recommend that site earthwork operations be scheduled for the dry months. If site grading and fill placement occur during wet weather conditions, however, it will be necessary to use wet weather construction techniques. Such measures may include, but are not limited to the following:

- The use of track-mounted equipment and staging to limit subgrade disturbance.
- The use of haul roads or working pads where the subgrade may be subjected to repeated heavy construction traffic. Haul roads and working pads will likely require 18 inches of imported granular material, while twelve inches of imported granular material may be sufficient for light staging areas. The imported granular material should consist of crushed rock that is well-graded between coarse and fine particle sizes, contains no unsuitable materials or particles larger than 4 inches, and has less than 5 percent by weight passing the U.S. Standard No. 200 sieve. The imported granular material should be placed in one lift over the prepared, undisturbed subgrade

and be compacted using a smooth-drum, nonvibratory roller. A geotextile separator will reduce the required rock section as well as subgrade disturbance.

- The use of smooth edge buckets.
- Other methods to limit subgrade disturbance, as determined by the contractor.
- The use of cement-amended soils may be considered as well.

Because subgrade disturbance can vary greatly depending on the Contractor's means, methods, and schedule, we recommend that the Contractor be responsible to protect the subgrade as needed to complete earthworks and grading necessary for this project.

6.3 Subgrade Evaluation and Preparation

Following demolition and stripping, the existing subgrade within areas to be improved should be proofrolled with a fully-loaded dump truck or similar heavy rubber-tired construction equipment to identify remaining soft, loose, or unsuitable areas, where accessible. The proofrolling should be observed by Pali Consulting, who should evaluate the suitability of the subgrade and identify any areas of yielding that are indicative of soft soil. If soft zones are identified during proofrolling, these areas should be excavated to the extent indicated by Pali Consulting and replaced with structural fill. Because of the presence of undocumented fill encountered in the site explorations, greater than typical overexcavation should be anticipated in areas of undocumented fill.

6.4 Excavation

Site soils within expected excavation depths of up to 4 feet bgs will generally consist of clay and silt soils at variable moisture content but which are typically above optimum. It is our opinion that conventional earthmoving equipment in proper working condition should be capable of making necessary general excavations for the project, although low impact tracked equipment may be required to minimize site disturbance per *Section 6.2*. The earthwork contractor should be responsible to provide the equipment and procedures to excavate the site soils described in the exploration logs and text of this report. Softened material or pumping subgrades at the base of excavations should be moisture-conditioned and compacted as structural fill or replaced with granular structural fill prior to placing additional fill or placing concrete.

6.5 Excavation Dewatering

Perched groundwater may occur within the depths of planned excavations during most of the year. During the wet season, perched groundwater is expected to be more shallow and likely. Excavations that extend into saturated soils may need to be dewatered. If groundwater is encountered, sump pumps placed in the excavations should be sufficient for dewatering in most situations, however, other methods may be necessary if groundwater inflow becomes significant.

In addition to groundwater seepage, surface water inflow to the excavations during the wet season could be problematic.

Provisions for temporary ground and surface water control should be included in the project plans and should be installed prior to commencing work.

6.6 Excavation Stability

Excavation sidewalls should stand near-vertical to a depth of approximately 4 feet or more, provided perched or near-surface groundwater seepage does not affect the sidewalls. Excavations made to construct footings or other structural elements should be laid back at the surface as necessary to prevent soil from falling into excavations. All trench excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations. On-site soils anticipated within excavation depths are generally OSHA Type B soils.

While this report describes certain approaches to excavation, the contractor is responsible for selecting and designing the specific methods, monitoring the excavations for safety, and providing shoring required to protect personnel and adjacent structural elements.

6.7 Structural Fill and Backfill

Structural areas include all areas beneath fields, foundations, pavements, and any other areas intended to support structures or within the influence zones of structures.

Structural fill for the project can consist of the following soils per *Sections 6.7.1* through *6.7.4*. All structural fill should be free of debris, roots, organic matter, frozen soil, man-made contaminants, particles with greatest dimension exceeding 4 inches, and other deleterious materials. The suitability of soil for use as structural fill will depend on the gradation and moisture content of the soil. As the fines content of the soil increases, the soil becomes increasingly more sensitive to small changes in moisture content and achieving the required degree of compaction becomes more difficult or impossible.

6.7.1 On-Site Soils

The on-site soils may be used as structural fill, where they meet the general criteria above and have a PI of less than 20. Of the four PI's measured in site soils, only one had a PI below 20 (13) while two had PI's just over 20 (21 and 22) and one had a PI of 28. Based on the PI testing, shallow soil in the West Field may be suitable for use for fill, but in the East Field may not. Consideration could be given to the use of soils with marginally high PI's if special measures are taken. This general distribution of material can be used for planning purposes, but testing during construction should confirm the suitability of on-site soil used as structural fill.

The on-site soils will be sensitive to moisture content and may require moisture conditioning. If used as structural fill, the material should be placed and compacted in lifts with maximum uncompacted thicknesses and relative densities as recommended in the tables that follow. If proper moisture conditions cannot be attained, we recommend using imported structural fill per the following sections.

6.7.2 Imported Select Structural Fill

Imported granular material used as structural fill should be pit or quarry run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in SSC 00330.14 – Selected Granular Backfill or SSC 00330.15 – Selected Stone Backfill. The imported granular material should also be angular, fairly-well graded between coarse and fine material, have less than 10 percent by dry weight passing the U.S. Standard No. 200 Sieve, and have at least two mechanically fractured faces. The material should be placed and compacted in lifts with maximum uncompacted thicknesses and relative densities as recommended in the tables that follow. During dry weather, the fines content may be increased to a maximum of 20 percent.

6.7.3 Aggregate Base

Imported granular material used as aggregate base (base rock) beneath structures should be clean, crushed rock or crushed gravel and sand that is well graded between coarse and fine. The base aggregate should meet the specifications of SSC 00641 – Aggregate Subbase, Base, and Shoulder Base Aggregate, depending upon application, with the exception that the aggregate have less than 5 percent by dry weight passing a U.S. Standard No. 200 Sieve based on the minus 3/4-inch fraction and have at least two mechanically fractured faces. The aggregate base should have a maximum particle size of 1 inch.

The aggregate base material should be placed and compacted in lifts with maximum uncompacted thicknesses and relative densities as recommended in the tables that follow.

6.7.4 Trench Backfill

Utility trench backfill for pipe bedding and in the pipe zone should consist of well-graded granular material with a maximum particle size of 3/4-inch and less than 10 percent fines. The material should meet the structural fill recommendations provided above. Further, the pipe bedding and fill in the pipe zone should meet the pipe manufacturer’s recommendations. Above the pipe zone imported select granular fill or on-site soils may be used as described above, consistent with the overlying use of the area.

The pipe bedding and backfill should be placed and compacted in lifts with maximum uncompacted thicknesses and relative densities as recommended in Table 4.

6.8 Fill Placement and Compaction

Structural fill should be placed and compacted in accordance with the following guidelines.

- Place fill and backfill on an approved subgrade prepared as recommended in *Sections 6.1 through 6.3*. Place fill or backfill in uniform horizontal lifts with a thickness appropriate for the material type and compaction equipment. Table 2 provides general guidance for lift thicknesses.
- Use appropriate operating procedures to attain uniform coverage of the area being compacted.

Table 2. Guidelines for Uncompacted Lift Thickness

Compaction Equipment	Guidelines for Uncompacted Lift Thickness (inches)		
	On-Site Soil	Granular and Crushed Rock (Maximum Particle Size < 1½”)	Crushed Rock (Maximum Particle Size > 1½“)
Plate Compactors and Jumping Jacks	4 – 8	4 – 8	Not Recommended
Rubber-Tire Equipment	6 – 8	10 – 12	6 – 8
Light Roller	8 – 10	10 – 12	8 – 10
Heavy Roller	10 – 12	12 – 18	12 – 16
Hoe Pack Equipment	12 – 16	18 – 24	12 – 16

Note: The above table is based on our experience and is intended to serve as a guideline. The information provided in this table should not be included in the project specifications.

- Place fill at a moisture content within about 3 percent of optimum as determined in accordance with ASTM Test Method D 1557. Moisture condition fill to achieve uniform moisture content within the specified range before compacting. Compact fill to the percent of maximum dry densities as noted in Table 3.
- Do not place, spread, or compact fill soils during freezing or unfavorable weather conditions. Frozen or disturbed lifts should be removed or properly recompacted prior to placement of subsequent lifts of fill soil.

Table 3. Fill Compaction Criteria

Fill Type	Percent of Maximum Dry Density Determined in Accordance with ASTM D 1557		
	0 – 2 Feet Below Subgrade	>2 Feet Below Subgrade	Pipe Bedding and Pipe Zone
Mass Fill (on-site)[1]	92	90	----
Mass Fill (imported)[1]	95	92	----
Aggregate Base[1]	95	95	----
Trench Backfill	95	92	90
Nonstructural Trench Backfill	88	88	----
Nonstructural Zones	88	88	90

Notes:

1. Structural fill with more than 30 percent retained on the ¾-inch sieve should be compacted to a well-keyed dense state within 3 percent of optimum moisture content.

During structural fill placement and compaction, a sufficient number of in-place density tests should be completed by Pali Consulting to verify that the specified degree of compaction is being achieved.

6.9 CUT AND FILL SLOPES

The following sections provide recommendations for cut and fill slopes up to 4 feet high. If cut or fill slopes greater than 4 feet in height are planned, Pali Consulting should be contacted for additional geotechnical evaluation. Cut and fill slopes should be planted with appropriate vegetation as soon as possible after grading to provide protection against erosion.

6.9.1 Cut Slopes

Permanent cut slopes should be limited to an inclination of 2 horizontal to 1 vertical (2H:1V) or flatter for slopes up to 4 feet in height unless supported by retaining structures. Slopes to be mowed or otherwise maintained should be limited to an inclination of 3H:1V. If seepage occurs within any slope, flatter slopes or structural measures may be needed for stability. A qualified engineer should design such measures.

6.9.2 Fill Slopes

Permanent fill slopes should not exceed 2H:1V gradients, or 3H:1V if mowed or maintained as noted above. Keyways will be necessary for support of all fill slopes where the subgrade slopes at greater than 5H:1V. Additionally, when placed on ground sloping steeper than 5H:1V, the ground should be benched. Keyways should have a minimum embedment of 2 feet into firm, undisturbed native soils. Keyway depths should be evaluated in the field on a case-by-case basis by the geotechnical engineer.

6.10 Drainage and Erosion Control

Surface runoff can be controlled during construction by careful grading practices. Such practices typically include the construction of shallow, perimeter ditches or low earthen berms, and the use of temporary sumps to collect runoff and prevent water from ponding and damaging exposed subgrades.

Storm drainage should be carefully planned so surface gradients direct stormwater away from building foundations, slopes, paved areas, and sidewalks. Water from roof downspouts should similarly be conveyed away from such areas. All storm drainage should be conveyed away from the mapped deep-seated landslide and to the drainage west of the West Field, rather than north of the West Field.

Erosion control measures during and after construction should comply with City standards.

7.0 PAVEMENT DESIGN

New pavements may consist of conventional asphaltic concrete (AC) or Portland cement concrete (PCC) for roadways, parking areas and paths. Our recommendations for these roadways are provided in the sections below.

7.1 ROADWAY AND PARKING DESIGN

Roadway and parking pavement will consist of conventional AC or PCC pavements. We understand that traffic counts are not available but are expected to be very light. Traffic is expected to be almost exclusively consist of passenger vehicles with an occasional firetruck in emergencies and an occasional maintenance vehicle. Thus, we assumed a traffic loading of 10,000 equivalent single-axle loads (ESALs).

For AC pavement design, this is consistent with the Asphalt Paving Association of Oregon (APAO) Traffic Level I, which is described as follows:

- Traffic Level I – Very light traffic for parking lots and residential driveways (up to one truck per day and 10,000 equivalent axle loads [EAL's] in a 20-year period).

In calculating the AC pavement, we used a reliability level of 75 percent. A reliability level of 75 percent is recommended for facilities that are moderately important but can allow some disruption in use during the lifetime of the pavements, which is appropriate for this facility.

For PCC pavement design, we used the guidelines developed by the American Association of State Highway and Transportation Officials (AASHTO) Guide for Design of Pavement Structures (AASHTO 1993). We assumed a reliability and standard deviation of 95 percent and 0.35, respectively, a PCC compressive strength of 4,000 psi, and a modulus of rupture of 500 psi.

For all pavements, we assumed that site development occurs during a period of dry weather, and that site and subgrade preparation are completed in accordance with the recommendations of this report.

If the above assumptions are inaccurate, please contact us to develop updated recommendations.

7.1.1 AC and PCC Pavement Sections

Based on the above and provided the soil subgrade will be prepared as described in *Sections 6.1* through *6.3*, the conventional AC pavement section shown in Table 4 may be utilized, with an approximate service life of 20 years. If preferable to the City, the more conservative standard pavement section for a Local Street Section, per Standard Drawing No. 201, can be used in lieu of the minimum section.

Table 4. Minimum Pavement Section with Compacted Subgrade

Pavement Designation	AC (inches)	Aggregate Base (inches)
Conventional AC	3.0	6.0
City Local Street Section	3.5	10.0

For PCC pavements, the recommended section is shown in Table 5 below.

Table 5. Minimum Pavement Sections with Compacted Subgrade

Pavement Type	Pavement (inches)	Aggregate (inches)
PCC	5.0	6.0

The pavement sections in Tables 4 and 5 are minimum recommended material thicknesses and assume the subgrade has been prepared as recommended in this report.

We note that the "design aggregate base" thickness for pavement areas is intended to support post-construction design traffic and should not be used to support construction traffic or when the subgrade soils are wet. Accordingly, if staging areas or haul roads are proposed in pavement areas, the "design thickness" of the base rock should not be relied upon and additional thicknesses of base rock should be placed.

7.1.2 Pavement Materials

7.1.2.1 AC Pavements

The AC should be Level 2, 12.5-mm, dense hot mixed asphalt concrete according to the Oregon Department of Transportation (ODOT) Standard Specifications for Construction (SSC) 00744 – Minor Hot Mixed Asphalt Concrete Pavement. The asphalt cement binder should be PG 64-22 Performance Grade Asphalt Cement. The minimum AC lift thickness should be 1.5 inches. The AC should be compacted to 91 percent of Rice Density of the mix, as determined in accordance with ASTM D 2041.

7.1.2.2 PCC Pavements

The PCC should conform to the specifications provided in OSS Section 00756 - Plain Concrete Pavement. The PCC should have a minimum compressive strength of 4,000 psi and nominal maximum aggregate size of 1.5 inches. The PCC should be constructed with a maximum joint spacing of 15 feet. The slabs shall be interlocked at contraction joints (e.g., continuous slab with no dowels). However, dowels should be used at construction and expansion joints.

7.1.2.3 Aggregate Base

Imported granular material used as base aggregate (base rock) for conventional pavements should meet the criteria specified in *Sections 6.7.3 and 6.8.*

7.1.3 Pavement Construction Considerations

Construction should be completed in general accordance with the SSC and applicable recommendations in *Section* 6.0 of this report. Construction traffic should not be allowed on new pavements. If construction traffic is to be allowed on newly constructed pavements, an allowance for additional traffic will need to be made in the design pavement section.

7.2 Pathway Pavements

Pathways for pedestrian use will consist of conventional AC or PCC surfacing. Minimum sections for AC pathways are provided in the Trail Design Guidelines (Portland Parks & Recreation, 2009). For both single and multiple users, including maintenance vehicles, an AC section of 3 inches is recommended over a crushed rock base. For pedestrian use only, however, a thinner AC section is appropriate. For PCC sidewalks, we recommend the requirements of the City of Sandy Standard Drawing No. 205 be met, except with an increased rock section to improve drainage and support on the seasonally wet soils. The recommended sections for pedestrian only walkways are provided in Table 6, below. If occasional vehicle traffic will use the pathways, for example, maintenance or emergency vehicles, we recommend the sections in Tables 4 and 5, as applicable, be utilized in lieu of those below.

Table 6. Minimum Pathway Pavement Sections with Compacted Subgrade

Pavement Type	Pavement (inches)	Aggregate Base (inches)
AC	2.5	6.0
PCC	4.0	6.0

8.0 STRUCTURAL DESIGN RECOMMENDATIONS

8.1 SHALLOW FOUNDATIONS

Based on our understanding of the site improvements, shallow foundations are suitable for support of proposed lightly loaded structures. The foundations may be continuous wall or individual spread footings bearing on medium stiff or better native soils or structural fill placed over these soils. We recommend that continuous wall footings have a minimum width of 18 inches and individual spread footings have a minimum width of 24 inches.

The bottom of exterior footings should be founded at least 18 inches below adjacent grade. Interior column footings should be founded at least 12 inches below grade.

8.1.1 Foundation Overexcavation and Subgrade Preparation

If unsuitable fill or deleterious material is encountered in footing excavations, we recommend the unsuitable material be overexcavated the depth it occurs and replaced with structural fill. The overexcavation should be wider than the footing by a distance equal to the overexcavation depth, and the footing should be centered on the backfilled subgrade. Before overexcavating, the subgrade should be evaluated by Pali Consulting, to confirm soft, loose, disturbed, or deleterious soils are present that should be removed and the required depth of removal.

Structural fill placement and compaction should be performed as described in *Sections 6.7* and *6.8*. The structural fill should meet the specifications of *Section 6.7.2* or *6.7.3*. Foundation bearing surfaces should not be exposed to standing water. If water infiltrates and pools in the excavation, the water, along with any disturbed soil should be removed before placing foundation forms or reinforcing steel.

We recommend that Pali Consulting observe final foundation subgrades before placing concrete forms and reinforcing steel to determine that bearing surfaces have been adequately prepared and that the soil conditions are consistent with those observed during our explorations.

8.1.2 Bearing Capacity

We recommend that conventional wall and column foundations be proportioned using a maximum allowable bearing pressure of 2,000 pounds per square foot (psf). This bearing pressure applies to the total dead and long-term live loads and may be increased by one-third when considering earthquake or wind loads. This is a net bearing pressure. The weight of the footing and overlying backfill can be ignored in calculating footing sizes.

8.1.3 Foundation Settlement

Shallow foundations designed and constructed as recommended are expected to experience movement (settlement or expansion) of less than 1 inch. Differential settlement up to ½-inch can be expected between adjacent footings supporting comparable loads.

8.1.4 Lateral Resistance

Lateral loads on footings can be resisted by passive earth pressure on the sides of footings and by friction on the bearing surface. We recommend that passive earth pressures be calculated using an equivalent fluid weight of 300 pounds per cubic foot (pcf) for foundations confined by native soils or structural fill. We recommend using a friction coefficient of 0.35 for foundations placed on native soil subgrade or on-site fill and 0.50 for foundations placed on crushed rock. The passive earth pressure and friction components may be combined provided that the passive component does not exceed two-thirds of the total.

The passive earth pressure value is based on the assumptions that the adjacent grade is level and that static groundwater remains below the base of the footing throughout the year. The top 12 inches of soil should be neglected when calculating passive lateral earth pressures unless the foundation area is covered with pavement or is inside a building. The lateral resistance values do not include safety factors.

8.1.5 Foundation and Slab Drains

We recommend that a foundation drain be included at the base of exterior footings if moisture sensitive floorings will be used inside of any structures, high interior moisture is not acceptable, or if the design passive pressures are required to resist lateral forces against the structures. The foundation drain should consist of a perforated drainpipe embedded in free-draining material per the OSSC (2022). The drainpipe should be tightlined to the storm drain system or other suitable discharge point and in accordance with *Section 6.10*.

8.2 SEISMIC DESIGN

We recommend that seismic design be performed using the 2022 Oregon Structural Specialty Code (OSSC) and ASCE 7-22 (or latest edition). We obtained the seismic hazard from the ASCE Hazard Tool Website for Latitude 45.399956 degrees and Longitude -122.260304 degrees for the 2,475-year return period. Risk Category II was assumed appropriate for site structures. The code-based seismic design parameters are included below in Table 7 and are only appropriate for code-level seismic design.

Table 7. Seismic Design Parameters.

Parameter	Value
Site Class	D
Spectral Response Acceleration, S_s	0.71g
Spectral Response Acceleration, S_1	0.27g
Maximum Spectral Response Acceleration (Short Period), S_{MS}	0.92
Maximum Spectral Response Acceleration (1-Seond Period), S_{M1}	0.6
Design Spectral Response Acceleration (Short Period), S_{DS}	061
Design Spectral Response Acceleration (1-Seond Period), S_{D1}	0.4
Maximum Considered Earthquake Geometric Mean PGA, PGA_M	0.39

9.0 LIMITATIONS

We have prepared this geotechnical evaluation for use by Lango Hansen Landscape Architects and their affiliates for the proposed Sandy Community Campus Park improvements, as described in this report. Our work was completed in general accordance with our services agreement for the project. Our report is intended to provide geotechnical recommendations for design of the project in accordance with our scope of work. However, geotechnical conditions can vary between exploration locations and our report should not be construed as a warranty of subsurface conditions. Favorable site performance in the near term does not imply a certainty of long-term performance, especially under conditions of adverse weather or other factors.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. No warranty, express or implied, should be understood.

Any electronic form, facsimile, or hard copy of the original document (email, text, table, and/or figure), if provided, and any attachments are only a copy of the original document. The original document is stored by Pali Consulting and will serve as the official document of record.

10.0 REFERENCES

American Association of State Highway and Transportation Officials (AASHTO), 1993, Guide for Design of Pavement Structures.

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Oregon Department of Geology and Mineral Industries, Statewide Landslide Information Layer for Oregon (SLIDO). Accessed at https://gis.dogami.oregon.gov/maps/slido/, June 2023.

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Oregon Water Resources Department, Well Report Query. Accessed at https://apps.wrd.state.or.us/apps/gw/well_log/Default.aspx, May 2023.

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Soil Survey Staff, Natural Resources Conservation Service, United States Department of Agriculture (USDA). Web Soil Survey. https://websoilsurvey.sc.egov.usda.gov/. Accessed May 2023.

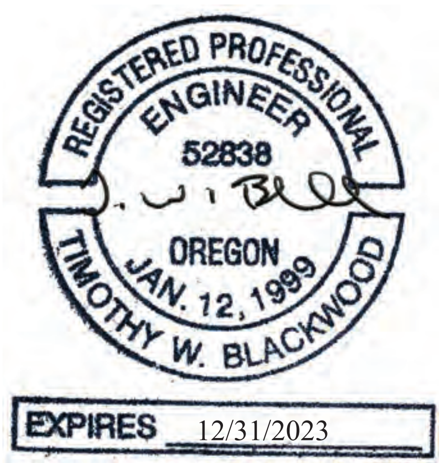
United States Geologic Survey, Estimated Depth to Groundwater in the Portland, Oregon Area. Accessed at https://or.water.usgs.gov/projs_dir/puz/index.html, May 2023.

United States Geologic Survey, Earth Explorer Website, accessed at https://earthexplorer.usgs.gov/, May 2023.

10.0 CLOSING

We appreciate the opportunity to submit this report for your project. Please contact us if you have any questions or need additional information.

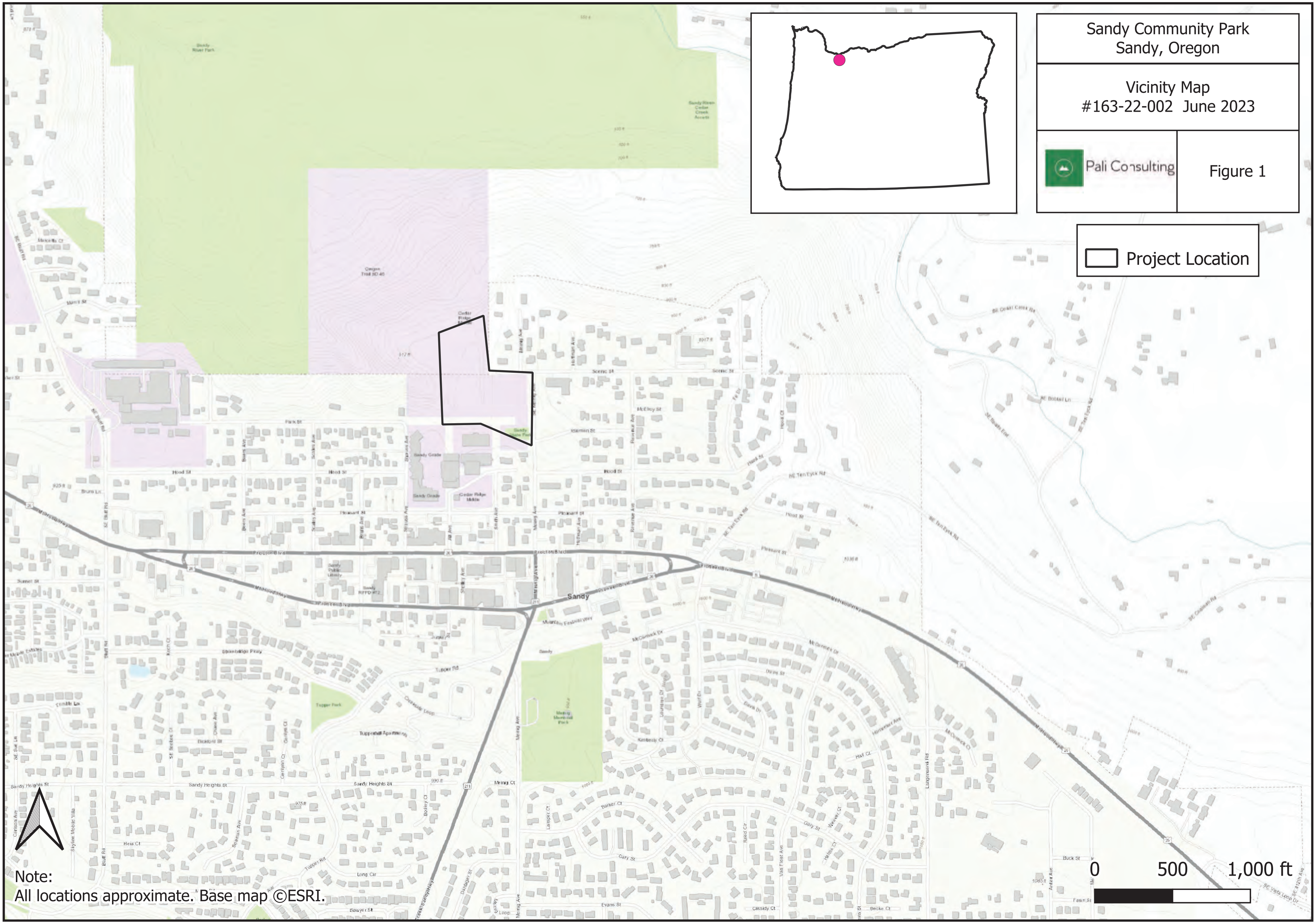
Sincerely,

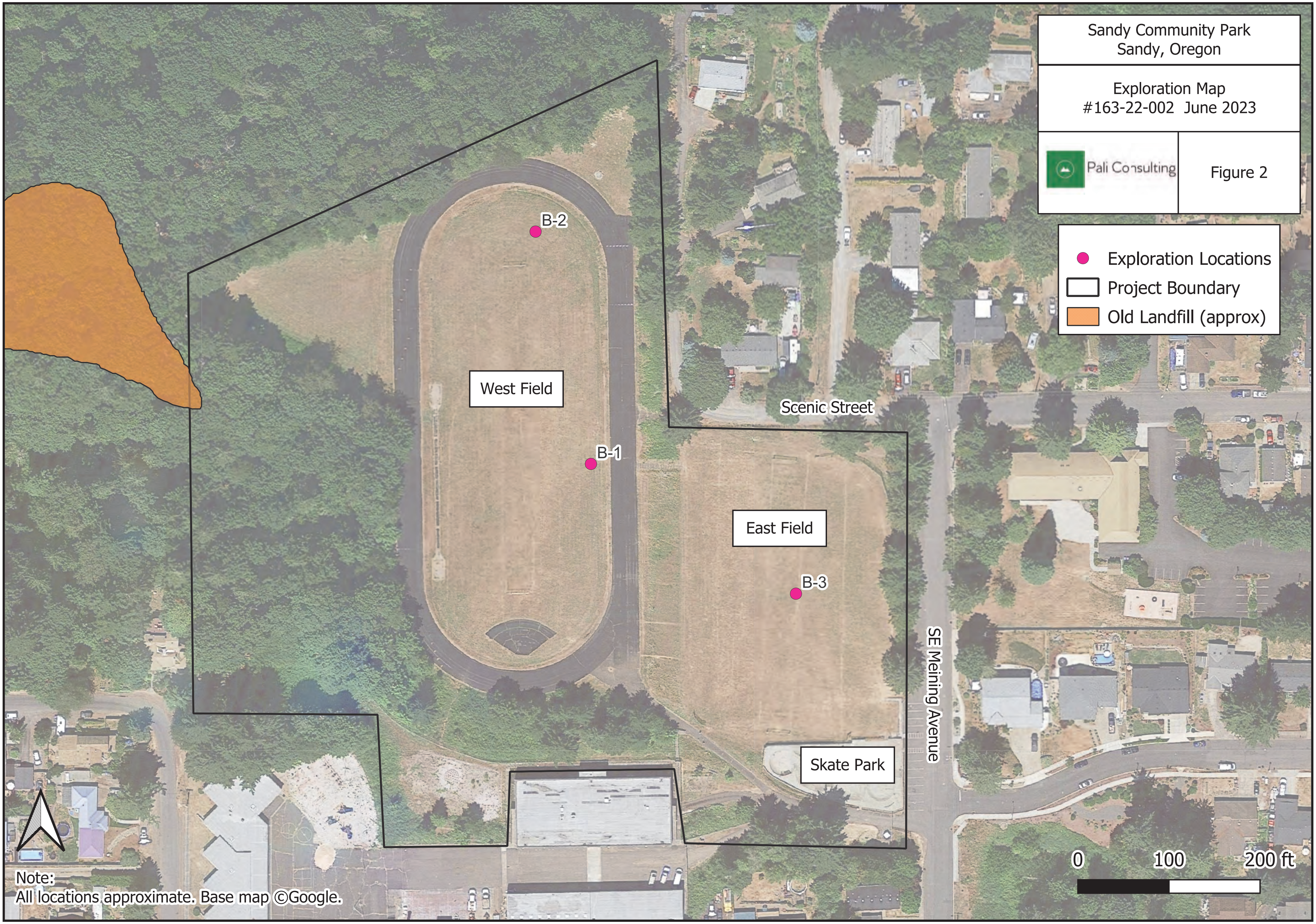


Timothy W. Blackwood, PE, GE, CEG
President/Principal Engineer

Attachments
- Figures 1 - 2
- Appendix A – Field Explorations, Infiltration and Laboratory Testing

Document ID: 163-22-002SandyGeotechnicalReport


Sandy Community Park
Sandy, Oregon
Vicinity Map
#163-22-002 June 2023
Pali Consulting
Figure 1
Project Location
Sandy
0
500
1,000 ft
Note:
All locations approximate. Base map ©ESRI.


Sandy Community Park
Sandy, Oregon
Exploration Map
#163-22-002 June 2023
Pali Consulting
Figure 2
Exploration Locations
Project Boundary
Old Landfill (approx)
B-2
West Field
B-1
East Field
B-3
Scenic Street
SE Meining Avenue
Skate Park
0
100
200 ft
Note:
All locations approximate. Base map ©Google.


Sandy Community Park
Sandy, Oregon
Landform Map
#163-22-002 June 2023
Pali Consulting
Figure 3
Project Boundary
Potential Landfill Sites
Mapped Landslides
Approximate Fill Setback
0
250
500 ft
Note:
All locations approximate. Base map provided by DOGAMI.

APPENDIX A -
FIELD EXPLORATIONS, LABORATORY AND INFILTRATION TESTING

FIELD EXPLORATIONS

GENERAL

We evaluated subsurface conditions at the site by completing three machine-drilled borings on May 20th, 2023. The machine-drilled borings were completed with a trailer mounted solid stem auger rig operated by Dan J. Fisher Excavations, Inc. The locations of the explorations are shown on Figure 2 of the report and were estimated based on field measurements.

The field explorations were coordinated by a geologist on our staff, who classified the various soil units encountered, obtained representative soil samples for geotechnical testing, and maintained a detailed log of each boring. Exploration logs are included in this Appendix.

SAMPLING AND LOGGING

The exploration logs within this Appendix show our interpretation of the drilling, sampling, and testing data. They indicate the depth where the soils change. Note that the change may be gradual. In the field, we classified the samples taken from the explorations according to the methods presented on the *Key to Exploration Logs* in this Appendix. The key also provides a legend explaining the symbols and abbreviations used in the logs.

Materials encountered in the explorations were classified in the field in general accordance with American Society for Testing and Materials (ASTM) International Standard Practice D 2488 "Standard Practice for the Classification of Soils (Visual-Manual Procedure)." Soil classifications and sampling intervals are shown in the exploration logs in this Appendix.

Soil samples were obtained from the borings using a SPT sampler completed in general conformance with ASTM Test Method D 1586 "Standard Method for Penetration Test and Split-Barrel Sampling of Soils." The sampler was driven with a 140-pound cathead operated hammer falling 30 inches. The N-value, or number of blows required to drive the sampler 1 foot or as otherwise indicated into the soils, is shown adjacent to the sample symbols on the boring logs. Disturbed samples were obtained from the sampler for subsequent classification and testing.

INFILTRATION TESTING

We conducted two infiltration tests at the locations shown on Figure 2. The tests consisted of encased falling head tests in general accordance with the Clackamas County Service District #1, Stormwater Design Standards, Appendix E, E.2.2.b, but modified for duration due to the limited drilling schedule. Our specific procedures are briefly described below.

- Borings were advanced to the test depths of 5 feet and 15 feet bgs, respectively. Pipes were seated approximately 6 inches into the bottoms of the holes to create plugs of soil at the bases of the pipes. A 6-inch diameter pipe was used for IT-1 (5 feet bgs) and a 3-inch pipe diameter pipe was used for IT-2 (15 feet bgs).

- The pipes were filled with greater than 12 inches of water to saturate the subgrade. The pipes were allowed to saturate for at least one hour. Infiltration test measurements were taken over the subsequent hours.

- To conduct the infiltration tests after the saturation period, the pipes were refilled approximately 5 feet above the test depth and the infiltration rate monitored. Water levels in the pipe were recorded every 10 minutes for a two-hour period.

The results of the testing are provided in our report.

LABORATORY TESTING

GENERAL

Soil samples obtained from the explorations were evaluated to confirm or modify field classifications, as well as to evaluate their engineering properties. Representative samples were selected for laboratory testing. The tests were performed in general accordance with the test methods of the ASTM or other applicable procedures. Test results are indicated on the boring logs and as described below.

SOIL CLASSIFICATIONS

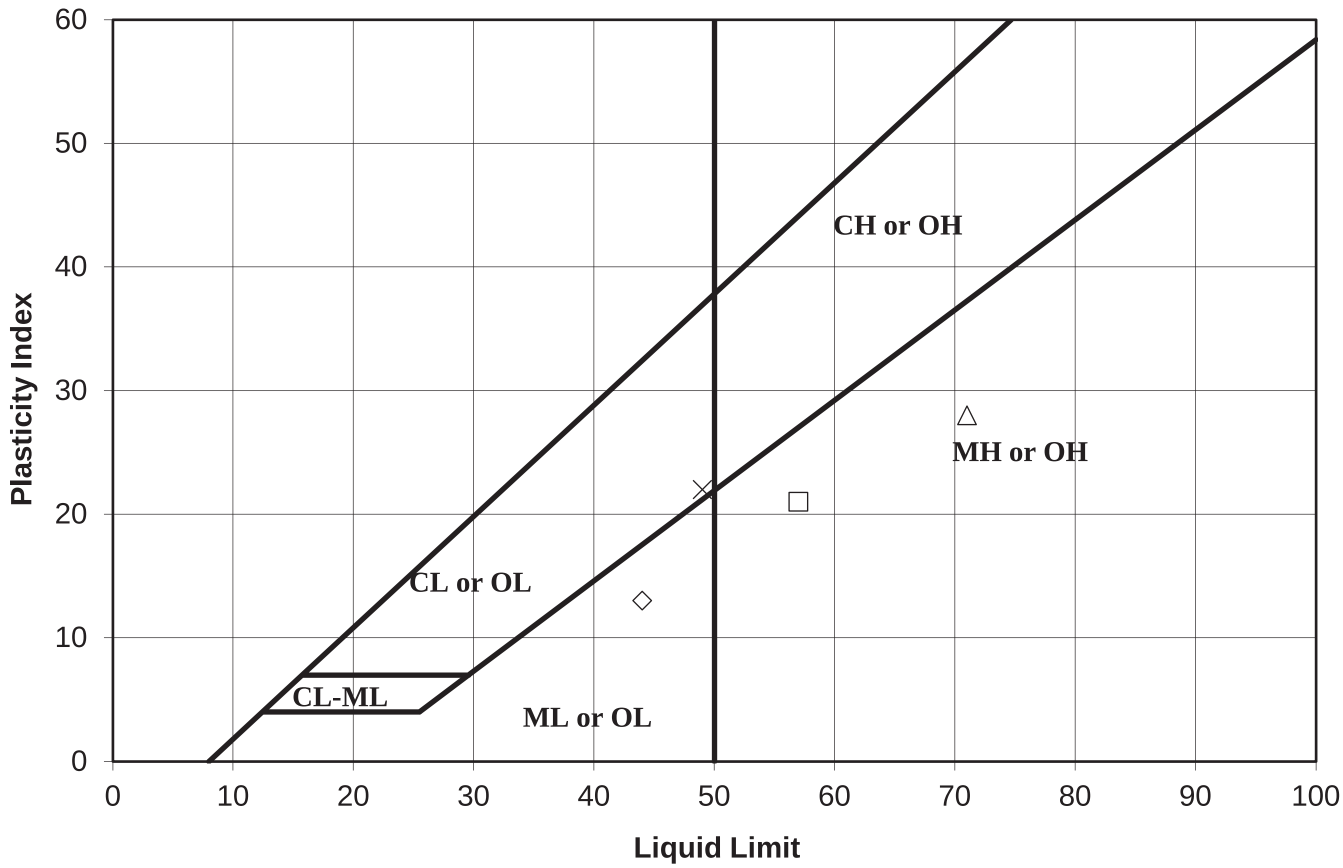
Soil samples obtained from the explorations were visually classified in the field and in our geotechnical laboratory based on the USCS and ASTM classification methods. ASTM Test Method D2488 was used to classify soils using visual and manual methods. ASTM Test Method D2487 was used to classify soils based on laboratory test results.

LABORATORY TESTING

Moisture Content
Moisture contents of samples were obtained in general accordance with ASTM Test Method D 2216. The results of the moisture content tests completed on samples from the explorations are presented on the exploration logs included in this Appendix.

Fines Content Analyses
Fines content analyses were performed to determine the percent of soils finer than the U.S. No. 200 Sieve, the boundary between coarse- and fine-grained soils. The tests were performed in general accordance with ASTM Test Method D 1140. The test results are indicated on the exploration logs included in this Appendix.

Atterberg Limits
Atterberg limits (liquid limit, plastic limit, and plasticity index) of fine-grained soil samples were obtained in general accordance with ASTM Test Method D4318-02. The results of the Atterberg limits tests completed on samples from the explorations are presented in the boring logs and on pages A-15 and A-16 in this Appendix.



Atterberg Limits Determination							
Symbol	Boring	Sample	Depth (ft)	Liquid Limit	Plastic Limit	PI	Classification
◇	B-1	S-1	2.5	44	31	13	ML
□	B-1	S-3	7.5	57	36	21	MH
△	B-3	S-1	2.5	71	43	28	MH
╳	B-3	S-2	5	49	27	22	CL

NOTE: This report may not be reproduced, except in full, without written approval of Pali Consulting. Test results are applicable only to the specific sample on which the test was performed, and should not be interpreted as representative of samples obtained at other times or locations, or generated by other operations or processes.

ATTERBERG LIMITS - ASTM D4318	
Sandy Community Park Project No.: 163-22-002	
Pali Consulting	**Figure A-1**

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KEY TO EXPLORATION LOGS

Pali Consulting

4891 Willamette Falls Drive, Suite 1
West Linn, Oregon 97068
www.pali-consulting.com

SOILS CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)	GW	WELL-GRADED GRAVELS, GRAVEL - SAND - MIXTURES
			GP	POORLY GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	GM	SILTY GRAVELS, GRAVELS - SAND - SILT MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL- SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	CLEAN SAND (LITTLE OR NO FINES)	SW	WELL-GRADED SANDS, GRAVELLY SANDS
			SP	POORLY-GRADED SANDS, GRAVELLY SANDS
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)	SM	SITLY SANDS, SAND - SILT MIXTURES
			SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% PASSING NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50	ML	INORGANIC SILTS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY
			OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS			PT	PEAT-HUMUS, SWAMP SOILS WITH HIGH-ORGANIC CONTENTS

Note: Multiple symbols are used to indicate borderline or dual soil classifications

SYMBOLS LETTER	DESCRIPTIONS
CC	CEMENT CONCRETE
AC	ASPHALT CONCRETE
TS	TOPSOIL/SOD FORREST DUFF

Stratigraphic Contact

———	Distinct contact between soil strata or geologic units
-----	Gradual or approximate change between soil strata or geological units

Moisture Modifiers

Dry - Absence of moisture, dusty, dry to the touch

Moist - Damp, but no visible water

Wet - Visible free water or saturated, usually soil is obtained from below the water table

Seepage Modifiers

None

Slow - < 1 gpm

Moderate - 1- 3 gpm

Heavy - > 3 gpm

Caving Modifiers

None

Minor - isolated

Moderate - frequent

Severe - general

Minor Constituents

Trace: < 5% (silt/clay)

Occasional: < 15% (sand/gravel)

With: 5-15% (silt/clay) in sand or gravel

15-30% (sand/gravel) in silt or clay

Sampler Symbol Descriptions

- Core
- Standard Penetration Test (SPT)
- Shelby tube
- Piston
- Bulk or grab

Laboratory / Field Tests

- %F Percent fines
- AL Atterberg Limits
- CP Laboratory compaction test
- CS Consolidation test
- DS Direct shear
- HA Hydrometer analysis

Laboratory / Field Tests

- DD Dry density
- OC Organic content
- PP Pocket penetrometer
- SA Sieve analysis
- TV Torvane shear
- MC Moisture Content

Blowcount (N) is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted) per ASTM D-1586. See exploration log for hammer weight and drop.

A "P" indicates sampler pushed using the weight of the drill rig.

(2.4-inch) sampler N approximately corrected to equivalent SPT N by 50% reduction in N - modified California.

Note: Refer to the report text and exploration logs for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the exploration locations at the time the explorations were made. The logs are not warranted to be representative of the subsurface conditions at other locations or times.

Figure A-2

Pali Consulting

Sandy Community Park

Sandy, OR

B-1

Project: Sandy Community Park Project	Driller: Dan Fisher, Inc
Proj No. 163-22-002	Date: 5/20/23
Drilling Method: Solid Stem Auger	Elevation: 913'
Diameter: 4" / Water Table: 20.3'	Logged by: JLE

Sample No.	Sample Type	Recovery (%)	RQD (%)	Blow Count per 6 inches	Blows/Foot (N)	Water Table	Depth (ft BGS)	Graphic Log	Materials Description	Moisture (%)	Remarks
							0	**ML**	4" topsoil / root mass		
S1		75		2-2-2	4				Soft to medium stiff, moist, brown SILT with minor black and red mottles, occasional charcoal (FILL)	33	AL
S2		75		2-4-5	9		5	**MH**	Stiff moist, grey to rusty red, mottled ELASTIC SILT (NATIVE)	34	
S3		100		5-8-10	18				Grades to very stiff, orange to grey	31	AL
S4		100		5-10-10	20		10		Grades to varicolored (orange/red/yellow/blue/black), with charcoal	31	
S5		100		5-7-6	13		15		Grades to stiff, varicolored (yellow/grey/brown/red), with a 6" zone of weathered grey siltstone		Drillers report 1' zone of hard drilling at 17' BGS
S6		100		3-5-5	10	▼	20		Grades to wet, with minor sand	57	%F=89
S7		100		1-2-3	5		25		Grades to grey, brown, red		
								END	Boring completed at 26.5' BGS		
							30				

Figure A-3

Pali Consulting

Sandy Community Park

Sandy, OR

B-2

Project: Sandy Community Park Project	Driller: Dan Fisher, Inc
Proj No. 163-22-002	Date: 5/20/23
Drilling Method: Solid Stem Auger	Elevation: 910'
Diameter: 4" / Water Table: 22.8	Logged by: JLE

Sample No.	Sample Type	Recovery (%)	RQD (%)	Blow Count per 6 inches	Blows/Foot (N)	Water Table	Depth (ft BGS)	Graphic Log	Materials Description	Moisture (%)	Remarks
							0	ML	4" topsoil / root mass		
S1		100		4-4-5	9				Stiff, moist, varicolored (red/orange/yellow/white/black/green) SILT with rock fragments and minor sand (NATIVE)	51	
S2		100		4-3-3	6		5		Grades to medium stiff, with few rounded gravels	54	
S3		100		2-4-4	8				Grades to medium stiff to stiff, highly variable, with distict color zones and relict rock structures	58	
S4		100		3-5-5	10		10		Grades to stiff, moist to wet, no gravels	47	AL
S5		100		4-6-5	11		15		Varicolored (grey/black/yellow/white), with sand and charcoal, grading to grey and brown mottled silt with few rounded gravels at bottom of sampler	54	%F=91
S6		100		2-2-8	10		20		Grades to wet, varicolored (grey/yellow/white/black/red/pink/purple) silt with sand and rounded gravel	62	Drillers report water at 20' bgs
						▼					
S7		100		4-3-7	10		25			58	%F=93
								END	Boring completed at 26.5' BGS		
							30				

Figure A-4

Pali Consulting

Sandy Community Park

Sandy, OR

B-3

Project: Sandy Community Park Project	Driller: Dan Fisher, Inc
Proj No. 163-22-002	Date: 5/20/23
Drilling Method: Solid Stem Auger	Elevation: 927'
Diameter: 4" / Water Table: Not encountered	Logged by: JLE

Sample No.	Sample Type	Recovery (%)	RQD (%)	Blow Count per 6 inches	Blows/Foot (N)	Water Table	Depth (ft BGS)	Graphic Log	Materials Description		Moisture (%)	Remarks
						GWT not encountered	0		**ML**	4" topsoil / root mass		
S1		75		3-3-4	7					Medium stiff, moist, red-brown to grey mottled ELASTIC SILT with charcoal (FILL)	37	AL
S2		75		2-2-2	4		5		**CL**	Soft to medium stiff, moist, brown to grey to red-brown CLAY with few small gravels / coarse sand, and wood/organic material (NATIVE)	36	AL, %F=88
S3		75		2-2-3	5					Grades to medium stiff, brown to grey	34	
S4		75		2-4-4	8		10			Grades to medium stiff to stiff, with occasional charcoal, no wood / organic material	36	
S5		100		4-6-8	14		15		**CL-CH**	Grades to stiff, grey to orange-brown with slight orange mottling, no charcoal, increasing plasticity		
S6		100		4-7-7	14		20				57	
									END	Boring completed at 21.5' BGS		
							25					
							30					

File: C:\Users\Jane\Pali Consulting Dropbox\1-Projects\Active-Projects\163-LangoHansen\163-22-002SandyPark\Analysis\SandyParkLogs.log   Date: 6/12/2023

Figure A-5