STANWOOD SEPTAGE FACILITY STORMWATER IMPROVEMENTS

DESIGN REPORT

Prepared by:
Wilson Engineering
805 Dupont Street, Suite 7
Bellingham, WA. 98225
(360) 733-6100

Prepared for:
McDay Holdings

July 2016

Wilson
SURVEY/ENGINEERING

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JENIFER ANN RAMSEY
STATE OF WASHINGTON
43086
REGISTERED
PROFESSIONAL ENGINEER
8-2-2016

Prepared by:
Wilson Engineering
805 Dupont Street, Suite 7
Bellingham, WA. 98225
(360) 733-6100

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1.0 INTRODUCTION

General

The Stanwood Septage Facility will be a septage receiving and treatment facility, which will receive sewage from septic tank pumper trucks. The sewage will be processed, treated and disposed into the local sanitary sewer. The facility will consist of a storage tank, two process tanks, a lime feed silo, and operations building, and intake headworks structure and a driveway with employee parking.

The entire site is approximately 3.4 acres in size, and is bordered by 270th Street NW to the north, 84th Avenue NW to the east, an undeveloped lot and State Route 532 to the south, and an auto body shop and residences to the west. All of the site lies within the 100-year FEMA defined floodplain. Approximately 2.05 acres of the entire site will be used for the development of the septage facility.

The development is required to meet Washington State's minimum stormwater management requirements 1 through 10 per the 2005 Western Washington Stormwater Management Manual issued by the Washington State Department of Ecology (2005 Ecology Manual), per City of Stanwood code. The minimum requirements dictate that stormwater from pollution generating surfaces, such as pavement for vehicles and landscaping areas be treated. Stormwater from buildings or undisturbed areas of the site do not need treatment. Due to the proximity of the project to fish bearing waters, the level of treatment must be enhanced, which is the removal of sediment and metals from stormwater.

In addition to treatment of pollution generating surfaces, the all disturbed areas of site are subject to flow control to reduce the peak stormwater event flows to pre-developed (before settlement) conditions. The total disturbed area is 2.05 acres.

Stormwater from the paved areas will be directed to one side of the roadway, and travel over an amended filter strip (BMP 9.40) for treatment. The amended filter strip is a basic filter strip with 6 inches of compost amended soil tilled into the existing soil to support enhanced treatment. The filter strip must be a minimum of 12 feet in width. Once the stormwater travels over the filter strip, it will enter an open channel and be directed towards a catch basin, which will direct the stormwater under the pavement into a detention pond (Section 3.2.1, Vol V of the 2005 Ecology Manual).

The detention pond will have a bottom area of approximately 18,750 square feet. Requirements for the pond are 6 inches of sediment storage at the bottom of the pond (dead storage) and one foot of freeboard. The proposed detention pond has been designed to have one foot of active storage, making the total required pond depth 3 feet. Outflow from the pond is controlled by three orifice outlets to ensure the project meets pre-development discharge rates.

2.0 SITE DESCRIPTION

The site is vegetated with grass and some trees and brush. It is relatively flat (less than 5 percent slope) and currently has no buildings present onsite. There are existing water and sewer service lines that terminate towards the center of the site. The site was previously used as a lumber storage yard, but has been cleared, vegetated and vacant since at least 2005.

Stormwater currently leaves the site via drainage ditch and 30-inch stormwater pipe on the eastern edge of the property (see Drainage Plan). A catch basin located along Marine Drive likely drains into the 30-

inch stormwater pipe and was a discharge point of the original stormwater system for the property. The 30-inch pipe discharges south to a drainage ditch that drains into Irvine Slough. There will be no increase in effective impervious surfaces within 300 feet and no development whatsoever within 150 feet of the slough.

The proposed development will include pavement, buildings and tanks, and a detention pond.

Table 1. Land Use Summary for Site

Conditions	Pavement (acres)	Roof (Buildings and Tanks) (acres)	Detention Pond (acres)	Pasture (Cleared Land) (acres)	Total (acres)
Existing	0.0	0.0	0.0	2.05	2.05
Proposed	0.2021	0.0994	0.4304	1.3181	2.05

3.0 SOILS INFORMATION AND GEOTECHNICAL INVESTIGATION

Per the geotechnical report prepared by GeoEngineers (April 2016 – see Appendix A), the subsurface soils at the site are characterized as fill at the surface, overlying peat deposits and fine grained alluvium and sand. The groundwater levels were monitored during the month of March 2016, and were found to be approximately 3 feet below the surface (at Elevation 5 feet).

When the site was visited by Talasaea during a heavy precipitation event in November 2015, no water ponding was seen as to indicate wetlands were present. After installing data loggers, they determined the site had groundwater levels greater than 12 inches below the ground surface, which means the site does not have wetland hydrology (Critical Areas Report and Habitat Management Plan, April 2016).

Due to the silty, peat nature of the soils, it is expected that the site will be difficult to work when it is wet. Therefore, construction should only occur when the site is dry during the summer months (see Geotechnical Report, Appendix A).

4.0 EXISTING DRAINAGE CONDITIONS

Existing Drainage Controls

Ditches, culverts and some storm piping make up the existing stormwater controls surrounding the site. There are no engineered flow control or treatment systems currently used for stormwater leaving the site. Current flowrates off the upper portion of the site (where the development will take place), assuming pasture/cleared land, range from 0.06 cfs to 0.24 cfs.

Table 2. Existing Flow rate Summary for Site

Basin	2-Yr (cfs)	10-Yr (cfs)	50-Yr (cfs)	100-Yr (cfs)
Existing Pasture Land	0.0585	0.1219	0.2012	0.2427

5.0 MINIMUM REQUIREMENTS

The project is a redevelopment project site, as the site has previously been used as lumber yard. Based on Figure 2.4.1 in the 2005 Ecology Manual, Vol I, Minimum Requirements #1 - #10 apply only to disturbed areas of the site (see Appendix B for flow chart). Below is a summary of how each of the Minimum Requirements are satisfied.

#1: Preparation of Stormwater Site Plan
This design report has been prepared to respond to and explain the stormwater project required to meet 2005 Washington State Department of Ecology's Stormwater Manual for Western Washington minimum requirements and thus will serve as the Stormwater Site Plan.

#2: Construction Stormwater Pollution Prevention Plan
A preliminary Temporary Erosion and Sediment Control (TESC) Plan has been prepared to address construction related runoff (see Appendix C – Plan Set). The contractor will develop the final TESC Plan in compliance with the General NPDES Construction Permit.

#3: Source Control of Pollution
This project is designed to remove pollutants near their source and attenuate these pollutants. The proposed project does not generate additional pollution sources. Potential pollution from facility operations are covered in the facility's Spill Prevention and Countermeasure's Plan.

#4: Preservation of Natural Drainage Systems and Outfalls
This project will treat runoff before directing it to the existing closed-conveyance systems. The existing conveyance system routes runoff to Irvine Slough. Flow control is being used on this project to reduce flowrates to pre-developed conditions (see #7: Flow Control, below). No adverse effects to the downstream drainage system are expected because of this project.

#5: On-Site Stormwater Management
The project is required to use On-site Stormwater Management BMPs that infiltrate, disperse, and retain stormwater runoff onsite to the maximum extend feasible without causing flooding or erosion impacts. The site will use amended soils in treatment areas to encourage stormwater runoff treatment and retention. Treatment will use a low impact development method, an amended filter strip, as described in Minimum Requirement #6 below.

#6: Runoff Treatment
The project proposes to add 8,800 square feet of asphalt pavement, which is a pollution generating impervious surface and requires treatment. Since the project discharges ultimately to fish bearing waters, any treatment must be at an enhanced level of treatment. An enhanced level of treatment is described by the 2005 Ecology Manual as providing basic treatment (sediment removal) and removal of metals, particularly copper and zinc. A compost amended filter strip (BMP T9.40) (amended filter strip) is a facility that meets this enhanced treatment requirement.

An amended filter strip is a basic filter strip where the grass filter strip area is compost-amended to a minimum of 10% organic content in accordance with BMP T5.13 (Post-Construction Soil Quality and Depth – adding compost or topsoil to disturbed soils) and hydroseeded grass maintained at 4-inches high and

95% density (with mowing and periodic reseeding). Herbaceous shrubs for landscaping are allowed within the filter strip area.

#7: Flow Control
Flow control is required on the site because the impervious surfaces cause a greater than 0.1 cubic feet per second (cfs) increase in the 100-year storm event flow frequency, has approximately 13,100 square feet of impervious surface (pavement, buildings and tanks), and discharges indirectly into a non-exempt water body (Irvine Slough). Stormwater detention has been applied to only the areas that will be disturbed.

The site will use a detention pond with an outlet control structure to reduce peak stormwater flow rates discharging from the site to predevelopment levels. See Section 6.0 for more details on the stormwater detention pond.

#8: Wetlands Protection
The project does not discharge directly or indirectly into a wetland.

#9: Basin/Watershed Planning
The site is located within the FEMA 100-year floodplain and therefore all construction must be evaluated under the Floodplain Management regulations, such as the Floodplain Habitat Assessment and Mitigation plan. A plan has been prepared for this site and is submitted under a separate cover.

#10: Operation and Maintenance
The operation and maintenance procedures for the proposed stormwater facilities appears in Section 7.0.

6.0 PROPOSED DESIGN

The proposed stormwater facilities consist of compost amended soil filter strips to provide stormwater treatment. Stormwater from the paved areas will be directed to one side of the roadway, and travel over an amended filter strip.

Once the stormwater travels over the filter strip, it will enter an open channel and be directed towards a catch basin, which will direct the stormwater under the pavement into a detention pond. Outflow from the pond is controlled by three orifice outlets to ensure the pond meets pre-development discharge rates.

The stormwater flows for the entire site are summarized in Table 3. Only flow from the pavement area will be directed to the filter strip, therefore the filter strip is designed to treat only the pavement area. The flows are 15-minute flows as modeled by the WWHM2012 stormwater model (see Appendix B for model results).

Table 3. Proposed Flow Rate Summary for Site

Basin Area	Water Quality (cfs)	2-Yr (cfs)	10-Yr (cfs)	50-Yr (cfs)	100-Yr (cfs)
Project Site – Not Detained	0.04	0.1489	0.2563	0.3795	0.4412
Project Site – After Detention Pond	N/A	0.0213	0.0455	0.0810	0.1016
Treatment (Pavement) Area Only	0.0320	0.0858	0.1383	0.1933	0.2195

Filter Strip Design

The only area requiring treatment will be pavement area. To provide stormwater treatment, stormwater from the paved areas will be directed to one side of the roadway, and travel over an amended filter strip (BMP T9.40). The amended filter strip is a basic filter strip with 6 inches of compost-amended soil tilled into the existing soil to support enhanced treatment. The filter strip must be a minimum of 12 feet in width, but can be wider. Once the stormwater travels over the filter strip, it will enter an open channel and be directed towards a catch basin, which will direct the stormwater under the pavement into a detention pond (Section 3.2.1, Vol V of the stormwater manual).

Conveyance Swale and Piping capacity

The swale that will convey the water from the filter strip to a catch basin for conveyance under the pavement to the detention pond has been sized to be trapezoidal in shape, with a 2 foot wide bottom, 4 feet wide at the top, and 2:1 side slopes. The swale will be 6 inches in depth with a 0.005 ft/ft slope. The swale will be lined with grass for erosion protection. See swale calculations in Appendix B.

The pipe used to convey the swale stormwater to the detention pond will be 8 inches in diameter, with a 0.004 ft/ft slope. See pipe calculation in Appendix B.

Detention Pond

The detention pond will have a bottom area of approximately 18,750 square feet. Requirements for the pond are 6 inches of sediment storage at the bottom of the pond (dead storage) and one foot of freeboard. The proposed detention pond has been designed to have one foot of active storage, making the total required pond depth 3 feet. Outflow from the pond is controlled by three orifices to ensure the pond meets pre-development discharge rates. Note the model found the orifice sizes determined by the model are smaller than the minimum orifice diameter allowed by the Department of Ecology (Ecology requires a minimum of 0.5 inches, page 3-50 of Vol III of the 2005 stormwater manual). Since the pond will be within the 100-year floodplain, it is unlikely the pond will be able to fully drain during the 100-year storm event regardless of the orifice sizes. The orifice sizes used in the model ensure the pond has adequate storage capacity to protect the floodplain.

7.0 OPERATION AND MAINTENANCE

Maintenance Criteria for Amended Soil Filter Strip

- Inspect filter strip areas at least once every 6 months, preferably during storm events, and also after storm events of > 0.5 inch rainfall/ 24 hours or flooding events. Maintain adequate grass

growth and eliminate bare spots. Grass shall be at least at 95% density. Till in compost into the soil in bare areas (assuming bare area is not caused by sediment build up) and reseed. Use the seed mix shown below in Table 9.3.

- Mow grasses, if needed for good growth (typically maintain at 4 – 9 inches of grass height).
- Remove sediment as needed from the gravel portion of filter strip if grass growth is inhibited in greater than 10 percent of the filter strip length, or if the sediment is blocking the distribution and entry of the water onto the filter strip.
- Remove leaves, litter, and oily materials, and re-seed (see Table 9.3 below for seed mix) or resod, and regrade, as needed. Clean curb cuts and level spreaders as needed. Prevent scouring and soil erosion in the filter strip. If flow channeling occurs, regrade and reseed the filter strip, as necessary.
- Maintain drainage swale at base of filter strip to ensure positive drainage during storm events. Remove any accumulated sediment, leaves, debris, and maintain a grass height that does not impede stormwater flow. Maintain access to drainage swale outlet and ensure positive drainage during storm events. Ensure inlet catch basin is clean and free of sediment buildup (no more than 6 inches of sediment buildup in sump).

Table 9.3 – Grass Seed Mixes Suitable for Biofiltration Swale Treatment Areas

Mix 1		Mix 2	
75-80 percent	tall or meadow fescue	60-70 percent	tall fescue
10-15 percent	seaside/colonial bentgrass	10-15 percent	seaside/colonial bentgrass
5-10 percent	Redtop	10-15 percent	meadow foxtail
		6-10 percent	alsike clover
		1-5 percent	marshfield big trefoil
		1-6 percent	Redtop

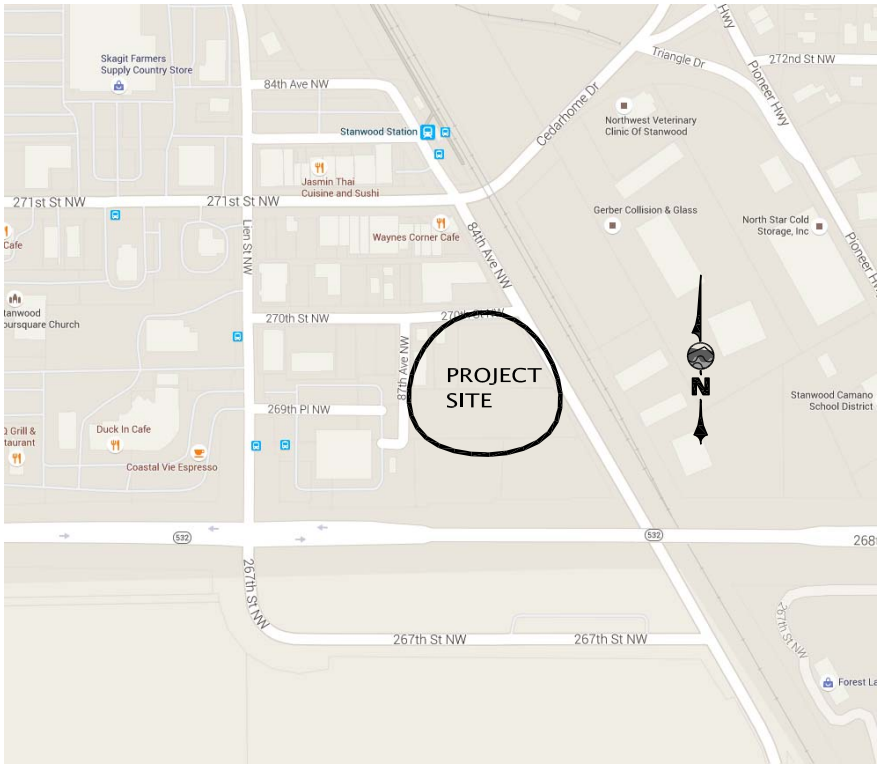
Note: *all percentages are by weight. * based on Briargreen, Inc.*

Maintenance Requirements for Detention Ponds

Maintenance Component	Defect Conditions	Conditions When Maintenance Is Needed	Results Expected When Maintenance Is Performed
General	Trash & Debris	Any trash and debris which exceed 5 cubic feet per 1,000 square feet (this is about equal to the amount of trash it would take to fill up one standard size garbage can). In general, there should be no visual evidence of dumping. If less than threshold all trash and debris will be remove	Trash and debris cleared from site.
General	Poisonous Vegetation and noxious weeds	Any poisonous or nuisance vegetation which may constitute a hazard to maintenance personnel or the public. Any evidence of noxious weeds as defined by State or local regulations. (Apply requirements of adopted Integrated Pest Management (IPM) policies for the use of herbicides).	No danger of poisonous vegetation where maintenance personnel or the public might normally be. (Coordinate with local health department) Complete eradication of noxious weeds may not be possible. Compliance with State or local eradication policies required
General	Contaminants and Pollution	Any evidence of oil, gasoline, contaminants or other pollutants (Coordinate removal/cleanup with local water quality response agency).	No contaminants or pollutants present.
General	Rodent Holes/Beaver Dams	Any evidence of rodent holes/beaver dams or any evidence of water piping through soil via rodent holes.	Rodents destroyed and any holes or obstructions repaired. (Coordinate trapping of beavers and removal of dams with appropriate permitting agencies)
General	Insects	When insects such as wasps and hornets interfere with maintenance activities.	Insects destroyed or removed from site. Apply insecticides in compliance with adopted IPM policies
General	After Flooding Event	Sedimentation or siltation of pond outlet control structure. Debris clogging orifice structure. See “Storage Area” below for sedimentation in pond.	Clean outlet control structure to ensure it functions as intended.
Side Slopes of Pond	Erosion	Eroded damage over 2 inches deep where cause of damage is still present or where there is potential for continued erosion. Any erosion observed on a compacted berm embankment.	Slopes should be stabilized using appropriate erosion control measure(s); e.g., rock reinforcement, planting of grass, compaction. If erosion is occurring on compacted berms a licensed civil engineer should be consulted to resolve source of erosion.
Storage Area	Sediment	Accumulated sediment that exceeds 10% of the designed pond depth unless otherwise specified or affects inletting or outletting condition of the facility.	Sediment cleaned out to designed pond shape and depth; pond reseeded if necessary to control erosion.

AREA MAP- NOT TO SCALE

VICINITY MAP- NOT TO SCALE



Wilson
SURVEY/ENGINEERING

WILSON ENGINEERING, LLC
805 DUPONT STREET
BELLINGHAM, WA 98225
(360) 733-6100 • FAX (360) 647-9061
www.wilsonengineering.com

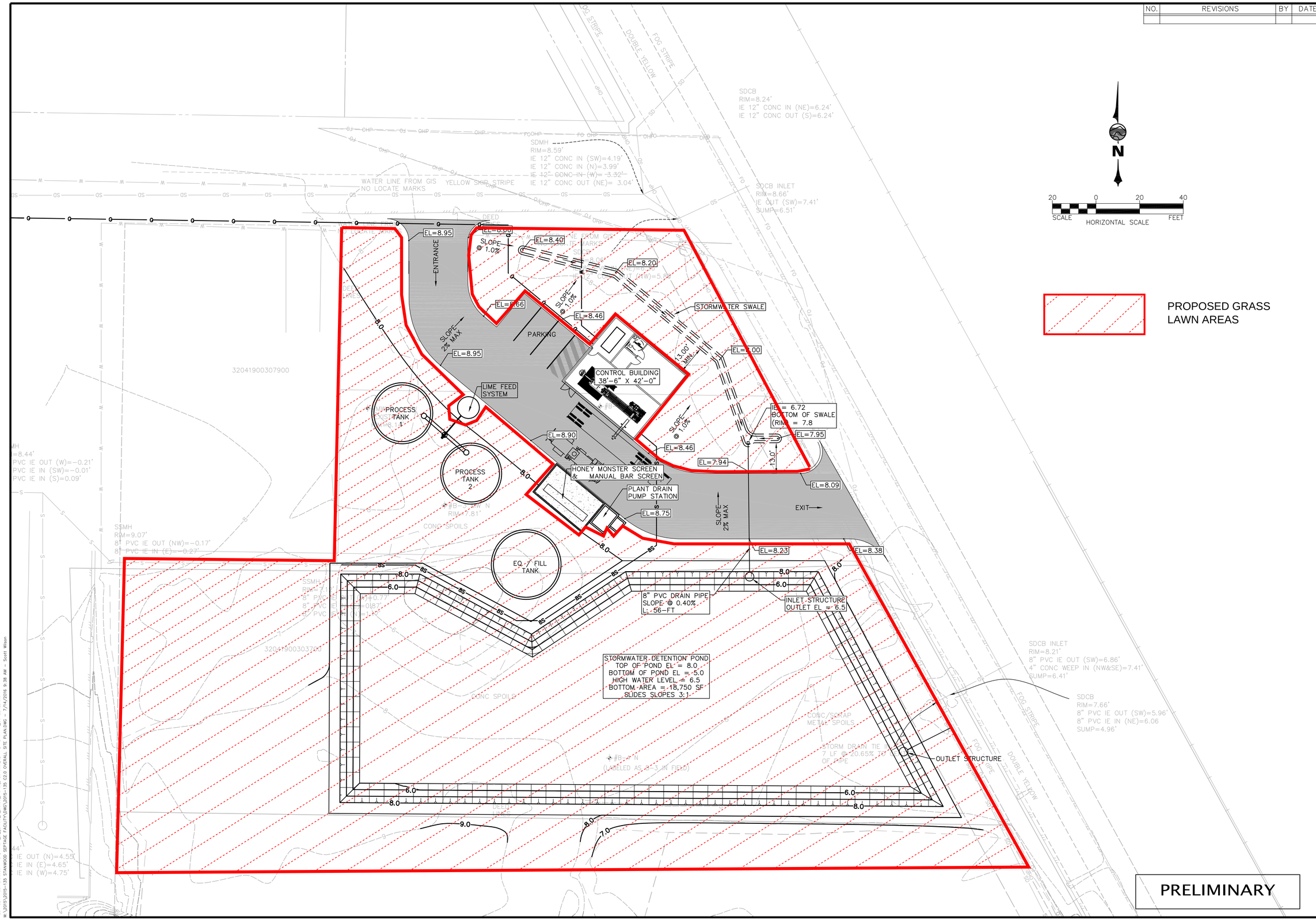
MCDAY HOLDINGS
SEPTAGE RECEIVING FACILITY

STANWOOD WASHINGTON

VICINITY MAP

DATE	SHEET
7-14-16	1
SCALE XXX	OF
JOB NO. 2015-135	1

NO.	REVISIONS	BY	DATE



WILSON ENGINEERING, LLC
805 DUPONT STREET
BELLINGHAM, WA 98225
(360) 733-6100 • FAX (360) 647-9061
www.wilsonengineering.com
Wilson SURVEY/ENGINEERING

JEFFREY G. CHRISTNER
STATE OF WASHINGTON
38888
REGISTERED PROFESSIONAL ENGINEER

DESIGNED BY	DRAWN BY	CHECKED BY
JGC	SJW	JGC

MCDAY HOLDINGS

STANWOOD WASHINGTON

SEPTAGE RECEIVING PLANT

PAVING PLAN / GRADING & DRAINAGE PLAN AND LANDSCAPING PLAN

DATE	SCALE	JOB NUMBER
07-14-2016	AS SHOWN	2015-135

SHEET C2.4

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PLOT SETTINGS: Bluebeam PDF, ARCH_D_(24.00_x_36.00_inches), Landscape, 1:1, WE APPA_SCREENED.ctb
W:\2015\2015-135 STANWOOD SEPTAGE FACILITY\DWG\2015-135 C2.0 OVERALL SITE PLAN.DWG - 7/14/2016 9:38 AM - Scott Wilson

APPENDIX A

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Geotechnical Engineering Services

Proposed Septage Facility
Stanwood, Washington

for
McDay Holdings, LLC

April 18, 2016

GeoEngineers

600 Dupont Street
Bellingham, Washington 98225
360.647.1510

Geotechnical Engineering Services

Proposed Septage Facility
Stanwood, Washington

File No. 22160-001-00

April 18, 2016

Prepared for:

McDay Holdings, LLC
c/o Wilson Engineering
805 Dupont Street, Suite 7
Bellingham, Washington, 98225

Attention: Jeff Christner, PE

Prepared by:

GeoEngineers, Inc.
600 Dupont Street
Bellingham, Washington 98225
360.647.1510

Aaron Hartvigsen, PE
Geotechnical Engineer

J. Robert Gordon, PE
Principal

MWR:AJH:JRG:tln

Two copies submitted

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Appendix B. Report Limitations and Guidelines for Use

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INTRODUCTION

This report presents the results of our geotechnical engineering services for the proposed Stanwood Septage Facility project located at 8622 270th Street in Stanwood, Washington. The general location of the site is shown in the Vicinity Map, Figure 1.

We understand the proposed facility will receive sewage pumped from septic tank trucks for processing, treatment and disposal into the sanitary sewer. The proposed improvements and site conditions are shown in the Site and Exploration Plan, Figure 2.

The facility will include:

- One fill/storage tank 30 feet in diameter and 20 feet tall (100,000 gallons)
- Two process tanks 26 feet in diameter and 20 feet tall (75,000 gallons)
- One 10-foot-diameter by 20-foot-tall lime feed silo
- Office/operations building (40 feet by 60 feet [2,400 square feet]) on a raised concrete deck above flood level (10 feet above grade)
- Intake headworks structure (20 feet by 40 feet) and associated equipment
- A stormwater pond across the southern third of the site as part of the flood storage balance required by Federal Emergency Management Agency (FEMA); the pond will be elevated so that the base is above local groundwater
- A pile supported structural slab for supporting trucks that are off-loading septage to the facilities
- Asphalt paved driveway

The site consists of four lots, and totals approximately 3½ acres. The lots are vegetated with grass, brush and a few medium to large trees. We understand that a wetland survey has been completed. The site is located in the lowland areas of Stanwood and is underlain by significant thickness of alluvial soils. The critical geotechnical considerations for the proposed site development include potential liquefaction, settlement and bearing support for the tanks, and dewatering/hydraulic uplift for the project elements.

The purpose of our geotechnical engineering services is to explore subsurface conditions at the site as a basis for developing geotechnical recommendations for the proposed development. The scope included drilling four geotechnical borings, installing a monitoring well at the location of one of the boring locations, completing laboratory testing on samples obtained from the explorations, performing engineering analyses, and preparing this report. The scope of work is described in our proposal for the project dated November 13, 2015 and authorized on November 16, 2015.

SITE CONDITIONS

Surface Conditions

The site is relatively level and currently undeveloped. The site is vegetated with grass and a few medium to large trees and brush. Various sized piles of brush, concrete, and trash are sporadically located across the

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site. The site is bordered by 270th Street NW to the north, 84th Avenue NW to the east, an undeveloped lot and State Route 532 to the south, and an auto body shop and residences to the west.

Geology

We reviewed a United State Geological Survey map for the project area, "Surficial Geologic Map of the Port Townsend 30- by 60- Minute Quadrangle, Puget Sound Region, Washington" by Pessl et al. (1989). Soil deposits in the site area are mapped as Holocene era alluvium deposits. Glacial consolidated deposits of the Vashon age, including till and recessional continental deposits are located to the east and upland of the site. Alluvium deposits in the area consist of clay, silt, and fine sand with minor amounts sand and gravel. Organic material is also common in the local alluvium.

Subsurface Explorations

Subsurface soil and groundwater conditions were evaluated by advancing four geotechnical borings (B-1 through B-4) at the site using a track-mounted drill rig subcontracted to GeoEngineers on February 25 and 26, 2016. The borings were completed to depths ranging from 11½ to 66½ feet below the existing ground surface (bgs). The approximate locations of the borings are shown in Figure 2. Details of the field exploration program, laboratory testing, and the boring logs are presented in Appendix A.

Subsurface Conditions

Subsurface soil conditions generally consisted of fill at the surface of overlying peat deposits overlying fine grained alluvium and then sand.

Fill was encountered in all borings. The fill consisted of very soft to soft sandy silt and very loose to loose silty sand. Variable amounts of gravel and organic matter were observed in the fill. Localized areas of fill may also contain concrete, wood debris, and other garbage as observed on the surface. Fill typically extended to a depth of approximately 4 feet bgs. The fill has low to moderate strength and compressibility characteristics.

Native peat deposits were encountered below the fill. Peat deposits consisted of very soft fibrous peat with minor amounts of silt, clay and sand. Peat deposits typically extended to a depth of approximately 6 to 8 feet bgs. Peat has very low strength and high compressibility characteristics.

Alluvial deposits were encountered below the peat deposits. The alluvium consisted of very soft to soft silt with minor amounts of sand, organic matter and shell fragments. The soft silt extended to 47 to 50 feet bgs in B-1 and B-2. The alluvium has low strength and high compressibility characteristics. Below the soft silt we encountered medium dense fine to coarse sand with silt to trace silt to the full depth explored of 51½ to 66½ feet bgs. The sand has moderate to high strength and low compressibility characteristics.

Groundwater Conditions

A groundwater monitoring piezometer was installed to a depth of 20 feet bgs at the location of boring B-3. Groundwater level of the monitoring well (B-3 MW on Figure 2) was measured on March 25, 2016; the measured depth of the groundwater from the surface was measured at 3.1 feet bgs (Elevation 4.7 feet). A pre-existing monitoring well (MW-0 on Figure 2) was encountered at the site during drilling activities and the groundwater level was measured on March 25, 2016 at 3.1 feet bgs (Elevation 5 feet). Groundwater

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was encountered in all of the borings near the top of the peat layer. Groundwater conditions should be expected to vary as a function of season, precipitation, nearby surface water level and other factors.

CONCLUSIONS AND RECOMMENDATIONS

General

We conclude that the proposed tanks and support structures should be supported by deep pile foundations to limit total and differential settlement associated with the soft and compressible fill, peat and alluvial soils.

The primary and secondary consolidation settlement of the soil profile across the site and could lead to potentially large differential settlement under the loads of the tanks and other infrastructure. In addition, the organic fibrous peat material encountered at the site has the potential for large magnitude long-term secondary consolidation settlement and decomposition of organic matter. GeoEngineers met with the owner and design team to discuss the site conditions and development options. A summary of the primary site preparation, design and construction considerations for the proposed project is provided below, based on our analyses and conversations. The summary is presented for introductory purposes only and should be used in conjunction with the complete recommendations presented in this report.

- The peat and very soft silt soils encountered at the site are highly compressible.
- A Site Class E in accordance with the 2012 International Building Code (IBC) is appropriate for design due to the depth of soft silt encountered underlying the site.
- The site soils have a low potential for liquefaction during the design earthquake.
- Deep pile foundations are recommended to support the proposed tanks and associated structural infrastructure at the site.
- The pile supported foundation elements should be underlain by a minimum 6-inch thickness of permeable granular capillary break material.
- Peat should be excavated from beneath pavement areas for better long term performance and reduced maintenance.
- The surficial soils encountered in our borings consist of silty fill. Wet weather trafficability will be very poor. We recommend that earthwork occur during dry summer months to reduce earthwork costs.

Seismic Considerations

Seismicity

The site is located within the Puget Sound region, which is seismically active. Seismicity in this region is attributed primarily to the interaction between the Pacific, Juan de Fuca and North American plates. The Juan de Fuca plate is subducting beneath the North American plate. It is thought that the resulting deformation and breakup of the Juan de Fuca plate might account for the deep focus earthquakes in the region. Hundreds of earthquakes have been recorded in the Puget Sound area. In recent history, four of these earthquakes were large events: (1) in 1946, a Richter magnitude 7.2 earthquake occurred in the Vancouver Island, British Columbia area; (2) in 1949, a Richter magnitude 7.1 earthquake occurred in the Olympia area; (3) in 1965, a Richter magnitude 6.5 earthquake occurred between Seattle and Tacoma; and (4) in 2001, a Richter magnitude 6.8 earthquake occurred near Olympia.

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Research has concluded that historical large magnitude subduction-related earthquake activity has occurred along the Washington and Oregon coasts. Evidence suggests several large magnitude earthquakes (Richter magnitude 8 to 9) have occurred in the last 1,500 years, the most recent of which occurred about 300 years ago. No earthquakes of this magnitude have been documented during the recorded history of the Pacific Northwest. Design practice in Puget Sound and building codes consider the local seismic conditions including local known faults in the design of structures.

Seismic Hazards

We evaluated the site for seismic hazards including liquefaction, lateral spreading and fault rupture. Our evaluation indicates the saturated alluvial deposits at the site have a low susceptibility to liquefaction as discussed in detail below. The site has a low risk of lateral spreading since it is relatively level. Based on USGS maps of active faults in the Puget Sound region, there are mapped faults within 2 miles of the Project area; however, because there are no mapped faults in the immediate vicinity of the Project area, it is our opinion that there is a low risk of fault displacement resulting in ground rupture at the surface.

2012 IBC Seismic Design Information

We understand the project will be designed utilizing 2012 IBC. We recommend the project site be classified as Site Class E – Soft Soil Profile as defined in the IBC. The design parameters for the 2012 IBC are summarized in Table 1 below. These values are based on an earthquake event that has a 2 percent chance of exceedance in a 50-year period (2,475 year event).

TABLE 1. SPECTRAL RESPONSE ACCELERATIONS (SRAs)

(SRA) and Site Coefficients	Short Period	1 Second Period
Mapped SRA	S_S = 1.138	S_1 = 0.444
Site Coefficients	F_a = 0.900	F_v = 2.400
Maximum Considered Earthquake SRA	S_{MS} = 1.024	S_{M1} = 1.066
Design SRA	S_{DS} = 0.683	S_{D1} = 0.711

Note: Soil Profile Type E Description: Soft Soil Profile (N<15)

Liquefaction Potential

Liquefaction is a phenomenon where soils experience a rapid loss of internal strength as a consequence of strong ground shaking. Ground settlement, lateral spreading and/or sand boils may result from liquefaction. Structures supported on liquefied soils could suffer foundation settlement or lateral movement that could be severely damaging to the structures. Conditions favorable to liquefaction occur in loose to medium dense, clean to moderately silty sand that is below the groundwater level. Dense soils/bedrock or soils that exhibit cohesion are generally considered not to be susceptible to liquefaction.

The evaluation of liquefaction potential is complex and is dependent on numerous site parameters, including soil grain size, soil density, site geometry, static stresses, and the magnitude and ground acceleration of the design-level earthquake. The alluvium encountered consisted of very soft silt. As previously stated, groundwater was encountered at a shallow depth within the soil profile. Based on the liquid limit and estimated clay content of the alluvial soils encountered, it is our opinion that the soils have a low potential for liquefaction based on criteria presented in (Andrews and Martin, 2000). Although the

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site soils are not anticipated to liquefy, the liquefaction risk will be mitigated by pile supporting the proposed structures.

Driven Steel Pipe Pin Piles

We recommend using 6- or 8-inch-diameter driven steel pipe "pin piles" for support of the proposed tanks, headworks, elevated building and reinforced concrete slab. The pile spacing should be determined by the project structural engineer. The pipe piles should be connected to grade beams and a structural slab/mat.

General

The pipe piles should be driven into the medium dense sand layer, which was encountered at approximately 50 feet bgs (Elevation -45 feet). We recommend that the piles extend a minimum of 10 feet into this layer. Piles typically come in 21 foot lengths. Therefore, we recommend planning for pile lengths of at least 62 feet (minimum tip elevation of -55 feet), with 1 foot of stickup. The actual length may vary based on driving resistance encountered during construction. Design recommendations for the driven piles are given in the following sections.

Axial Loading

For the proposed 6- or 8-inch steel pipe piles, we recommend that the piles be driven to refusal as recommended into the medium dense sand layer. Due to the soft soils encountered and a medium dense sand layer we recommend that the piles be driven close ended. This is typically accomplished by having the contractor weld a plate on the bottom of the pile. Piles driven to refusal in the medium dense sand layer can be designed with an allowable pile capacity of 30 kips for the 6-inch pile and 45 kips for the 8-inch piles, based on meeting appropriate refusal criteria, discussed below.

The allowable capacity presented above is based on refusal criteria being met and ignores the weight of the pile. The capacities apply to single piles. If piles groups are spaced at least 3-pile diameters on center, no reduction for pile group action is required. The capacity values may be increased by one-third when considering design loads of short duration such as wind or seismic forces.

Lateral Capacity

Lateral resistance and deflections of pile foundations are governed primarily by the lateral capacity of near-surface soil and the strength of the pile itself. The design lateral capacity of the piles will depend to a large extent on the allowable lateral deflections of the piles. We understand from discussions with the structural engineer that the grade beams/structural slab or mat will result in a fixed-head condition.

As requested by the structural engineer, we performed lateral pile capacity analysis. Table 2 below provides the input soil parameters used in our LPILE™ analysis of lateral pile behavior. The values are provided in units typically used in the LPILE™ program. The values in the table are ultimate values and no factor of safety has been applied. The top of the pile was assumed to be located 2 feet below existing grade.

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TABLE 2. LATERAL PILE ANALYSIS SOIL INPUT PARAMETERS

Soil Parameter	Layer 1	Layer 2	Layer 3	Layer 4
Depth[1] (ft bgs)	2 - 3	3 - 8	8 - 45	45 - 75
LPILE™ Soil Type	Soft Clay	Soft Clay	Soft Clay	Sand (Reese)
Effective Soil Unit Weight (pcf)	100	40	52	52
Soil Internal Angle of Friction (degrees)	N/A	N/A	N/A	32
Soil Cohesion (lb/ft^2)	250	50	250	N/A
E_{50}	0.02	0.025	0.02	N/A
Soil Modulus, k (pci)	N/A	7	N/A	20

Notes:

[1] Depth is referenced to anticipated pile top

in = inch; pci = pounds per cubic inch; psf = pounds per square foot

We have provided the lateral load which results in ½, 1 and 2 inches of deflection. We have provided the results in Table 3 below.

TABLE 3. LATERAL PILE ANALYSIS RESULTS

Pile Size	Applied Load and Lateral Deflection		
	0.5 inches	1.0 inches	2.0 inches
6 inch 0.280 inch wall thickness	3.6 kips	5.2 kips	6.8 kips
8 inch 0.322 inch wall thickness	5.2 kips	7.5 kips	10.2 kips

Notes:

The loads provided in the table above are unfactored and do not include a factor of safety.

Uplift Capacity

Lengths of steel pipe are typically connected using couplers. For piles that have uplift requirements, piles should be welded together in accordance with structural engineering requirements. Welded piles driven to refusal criteria below may be designed with design uplift capacities of 20 kips for 6 inch piles and 30 kips for 8 inch piles, which includes a factor of safety of about 1.5.

Alternatively, batter piles may be utilized to achieve lateral and uplift loads by using the compressive capacity of the piles in the direction of loading.

Pile Settlement

We estimate that settlement of pile foundations, designed and installed as recommended, will be less than ¾ inch if driven to refusal criteria. Most of this settlement will occur rapidly as loads are applied. Postconstruction differential settlements should be minor.

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Pile Corrosion Protection

Organic soils are corrosive to steel piles, therefore we recommend that the piles be hot dip galvanized or otherwise designed for aggressive soil corrosion.

Pile Testing Recommendations

We recommend a load test be completed on a minimum of one pipe pile per structure to verify load bearing capacity. However, the contractor should include time in the schedule to perform an additional load test should irregularities be observed. The load test should be to at least two times the design capacity in accordance with ASTM quick test standards.

Pile Installation Considerations

The 6- and 8-inch steel pipe pin piles are generally driven using a hydraulic hammer attached to a small excavator until a prescribed penetration rate is achieved. We recommend using a minimum 3,000-pound rated hammer. We recommend that the pipe piles be driven to practical refusal as defined in Table 4 below. Because of the variability that will occur at this site and footage payment schedule that will be appropriate, we recommend that GeoEngineers observe the pile installation on a full-time basis.

TABLE 4. PIN PILE REFUSAL CRITERIA- 3000 LB HAMMER

Pile Size	Seconds per Inch of Penetration
6 inch	6 seconds
8 inch	10 seconds

Due to the presence of uncontrolled fill at the site, potential wood in the alluvium, there is a risk that obstructions may be encountered during pile installation that may prevent reaching the minimum pile penetration. If an obstruction is encountered during driving that cannot be readily excavated/removed. GeoEngineers and the structural engineer should determine if the pile is acceptable or determine if another location should be identified. The contractor should be prepared to relocate the pile or install additional piles should an obstruction be encountered that prevents pile installation to design depth and refusal. We recommend that the project be bid on a lump basis, based on total estimated pile length; however, because the length of pipe and number of piles (because of obstructions) is unknown, we recommend that payment be based on actual footage installed.

Several Puget Sound contractors have the capability to install the recommended small diameter pipe pin piles. Contractors with small diameter pipe pile experience include McDowell NW Pile King of Kent, Davis Construction Services of Arlington, and Versatile Pipe and Pile Driving of Seattle.

Pavement Design Considerations

We do not have specific design parameters (traffic data) to develop pavement sections. The owner can choose the pavement section based on the available capital budget for the project and desired pavement performance. We recommend the site preparation be completed in accordance with the “Site Preparation” section of this report. To limit long term maintenance of the pavements, we recommend that the peat layer be removed below pavement areas. All organic soils should be removed prior to placing structural fill. It may be necessary to place a layer of quarry spalls to stabilize the base prior to placement of geotextile fabric and structural fill. The structural fill may consist of sand to lower the unit weight and should include

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a layer of bixaxial geogrid 2 feet below the pavement layer. We recommend the pavement section consist of a layer of gravel base to provide strength and drainage (and resistance to frost heave), a layer of crushed rock, and hot mix asphalt pavement. The pavement materials should be in conformance with the most recent version of the Washington State Department of Transportation (WSDOT) Standard Specifications for Bridge and Municipal Construction (herein after referred to as the WSDOT Standard Specifications). Fill should be placed in accordance with our subsequent earthwork recommendations.

Automobile Parking Areas

- 2½ to 3 inches of hot mix asphalt (HMA), Class ½-inch, PR 64-22 per the WSDOT Standard Specifications 5-04 and 9-03.8, or Class B asphalt from previous specifications
- 3 to 4 inches of crushed surfacing base course WSDOT Standard Specifications 9-03.9(3)
- 12 inches of gravel base consistent with the material described subsequently as "Select Import Fill"
- A woven geotextile placed directly over the silt subgrade with a minimum grab tensile strength (ASTM 4632) of 200 pounds (such as Mirafi 500X)
- Quarry spalls if needed

Access Roads, Truck Routes, and Loading Areas

- 3 to 4 inches of HMA
- 4 to 6 inches of crushed surfacing base course
- 12 to 18 inches of gravel base
- Biaxial grid 2 feet below pavement layer
- Structural fill, consisting of sand to reduce unit weight
- A woven geotextile placed directly over the silt subgrade
- Quarry spalls if needed

The pavement sections recommended above are based on our experience for conventional commercial site use. The minimum thickness of gravel base recommended will not protect the subgrade from degradation during wet conditions with heavy construction traffic.

Portland Cement Concrete Pavement

Portland cement concrete (PCC) sections will be used for the loading/unloading areas where trucks will be parked for long periods of time. We understand that the loading/unloading apron will be pile supported and reinforced per structural design. We recommend that the concrete pavement be supported on at least 6 inches of crushed surfacing base course in accordance with WSDOT Standard Specifications 9-03.9(3).

Pipe Design Considerations

Piping between all of the proposed site features will be required for this project. We recommend that bedding be completed in accordance with pipe manufacturer's recommendations. Based on the results of our exploration program, the piping will be supported predominately within very soft silt fill soils. Care should

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be taken to limit disturbance of subgrade soils prior to placement of the bedding material or the pipe on the excavated subgrade.

We recommend that the pertinent utilities such as gas, water, sewer lines and drains be attached to the building via flexible fittings to account for the potential differential settlement between the pile supported structures and the surrounding ground. Flexible couplings could be used at critical joints where differential settlement might occur. Also, we recommend that gas and water lines be equipped with automatic shutoff valves that are activated by ground shaking to reduce the potential for serious flooding and/or gas leaks during an earthquake.

Corrosivity

The organic peat soils are corrosive to ductile iron pipe. Therefore, we understand that alternative pipe materials are planned for this project.

Pipe Settlement

The likely mechanisms for pipe settlement include consolidation of underlying compressible soils under new loads. In areas where organic soils are present below the pipe or new loads applied, fill induced settlement will occur over time. For these areas, we recommend using clean uniform sand, with maximum dry density (MDD) on the order of 100 pounds per cubic foot (pcf), or other lighter weight material as trench backfill in these areas to reduce settlement potential.

Drainage Considerations

We recommend all downspouts be tightlined away from the building foundation area. It is conventional construction practice to include a perimeter drainage system for building foundations; however, based on the elevated nature of the building and pile supported tanks, it is our opinion that footing drains are not necessary. Site features that extend below the groundwater table should be watertight or include a sump.

Earthwork

Site Preparation

The existing surface and near surface soils (fill and peat, respectively) are moisture sensitive and susceptible to disturbance by construction equipment during wet weather. Therefore, we recommend that site preparation occur during periods of dry weather to minimize earthwork costs. We recommend that any existing vegetation/grass sod, and topsoil be stripped from the site prior to fill placement.

Structural Fill

General. All new fill placed under the new infrastructure and pavement areas should be placed and compacted as structural fill after the site preparation work is completed. The fill should be placed in horizontal lifts not exceeding 10 inches in loose thickness or that necessary to obtain the specified compaction with the equipment used. Each lift must be thoroughly and uniformly compacted. All structural fill material should be free of organic matter, debris, and other deleterious material. The maximum particle size diameter should be the lesser of either 5 inches or one half of the loose lift thickness.

- In non-settlement sensitive areas, such as outside of the roadway support prism, we recommend that the backfill be compacted to at least 85 percent of its MDD in accordance with ASTM D 1557.

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- In settlement sensitive areas, such as roadways and driveways, backfill placed at depths greater than 2 feet bgs should be compacted to at least 90 percent of the MDD. The upper 2 feet of backfill should be compacted to at least 95 percent of the MDD in existing/future pavement areas.
- Gravel base and crushed surfacing materials should be compacted to at least 95 percent of the MDD.
- In settlement sensitive areas (due to new loading), we recommend that imported fine to medium sand be used that has a MDD of about 100 pcf.

As the amount of fines (material passing the U.S. No. 200 sieve) increases in a soil, it becomes more sensitive to small changes in moisture content and during wet conditions, adequate compaction becomes more difficult to achieve. Generally, soils containing more than about 5 percent fines by weight cannot be properly compacted when the moisture content is more than a few percent from optimum.

Sufficient earthwork monitoring and a sufficient number of in-place density tests should be performed to evaluate fill placement and compaction operations and to confirm that the required compaction is being achieved.

Suitability of On-site Soil. Excavations at the site will be completed for foundations, utilities, peat removal below pavement and for the flood capacity on the southern portion of the site. We anticipate these excavations will extend into the upper fill soils consisting predominantly of fine-grained material with relatively high moisture content, and peat. Based on the fine-grained nature and results of laboratory moisture content tests, and the organic nature of the peat, these materials are not adequate for use as structural fill, and select import fill should be used where structural fill is required.

Select Import Fill. Imported soil should conform to the recommendations provided in the "General" section above. In the building, tank and pavement areas, and during wet weather or on wet subgrades in all areas, we recommend using a select import fill consisting of the following: a sand and gravel with a fines content of less than 5 percent based on that portion passing the ¾-inch sieve. We generally recommend at least 30 percent gravel (retained on the U.S. No. 4 sieve). However, many of the local borrow soils have 20 to 30 percent gravel and are generally acceptable. As previously mentioned, using a sand borrow source would reduce post construction settlement because it will have a similar unit weight to that of the fill being removed. Other import soils may be submitted and approved by the geotechnical engineer.

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Excavations

All excavations and other construction activities must be completed in accordance with applicable city, county, state and federal safety standards. The on-site soils can be excavated using conventional earthmoving equipment. The subgrade will be susceptible to disturbance and softening which could be reduced by use of smaller or low ground pressure equipment.

Regardless of the soil type encountered in the excavation either shoring, trench boxes or sloped sidewalls will be required for excavations deeper than 4 feet under Washington State Administrative Code (WAC) 296-155, Part N. We expect that most of the trench excavations will be made as open cuts in conjunction with the use of a trench box and/or sloped sidewalls for shielding workers. For planning purposes only, the soil found on site is classified as "Type C" soil. The regulations allow temporary slopes for this condition up to 1.5H:1V (horizontal:vertical). However, the peat has very low shear strength and will likely be in a

saturated condition. This situation may result in very unstable slopes that may need to be managed with some limited shoring or very shallow side slopes, or using steel plates.

The above regulations assume that surface loads such as construction equipment and storage loads will be kept a sufficient distance away from the top of the cut so the stability of the excavation is not affected. Flatter slopes and/or shoring will be necessary for those portions of the excavations which are subjected to significant seepage in order to maintain the stability of the cut. Temporary slopes in wet/saturated sand will be susceptible to sloughing, raveling and "running" conditions. It should be expected that unsupported cut slopes will experience some sloughing and raveling if exposed to surface water. Berms, hay bales or other provisions should be installed along the top of the excavation to intercept surface runoff to reduce the potential for sloughing and erosion of cut slopes during wet weather.

Construction Dewatering

We encountered groundwater at approximately 3.1 feet below existing grades when taking measurements in the new and old monitoring wells at the site. It is likely that sumps and pumps with limited shoring (trench boxes) will be sufficient to dewater the trenches if the water level needs to be lowered only 1 to 2 feet within the fill soils. Any excavations extending into the peat layer may require more aggressive dewatering. *Dewatering of peat can induce settlement for a significant distance laterally from the area being dewatered.* More aggressive pumping would be required for deeper excavations and in the winter/spring months. It may be more appropriate to use some kind of temporary shoring to keep the water out of the excavations that extend into the peat.

If the excavations will be significantly deeper than the groundwater, some pumped wells or wellpoints could be required. The contractor should have the responsibility to determine the dewatering measures needed for the project if more than just localized sumps and pumps. The Dewatering Plan should be prepared by or under the direct supervision of a Professional Engineer and Licensed Hydrogeologist registered in the State of Washington. All dewatering should be accomplished in accordance with local, state and federal regulations.

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LIMITATIONS

We have prepared this report for use by McDay Holdings, LLC, Wilson Engineering and other members of the design team for use in design of the proposed Stanwood Septage Facility project in Stanwood, Washington.

Within the limitation of scope, schedule and budget, our services have been executed in accordance with generally accepted geotechnical practices in the area at the time the report was prepared. No warranty or other conditions, 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 GeoEngineers, Inc. and will serve as the official document of record.

Please refer to the Appendix B, "Report Limitations and Guidelines for Use," for additional information pertaining to use of this report.

REFERENCES

International Code Council. 2012. "International Building Code."

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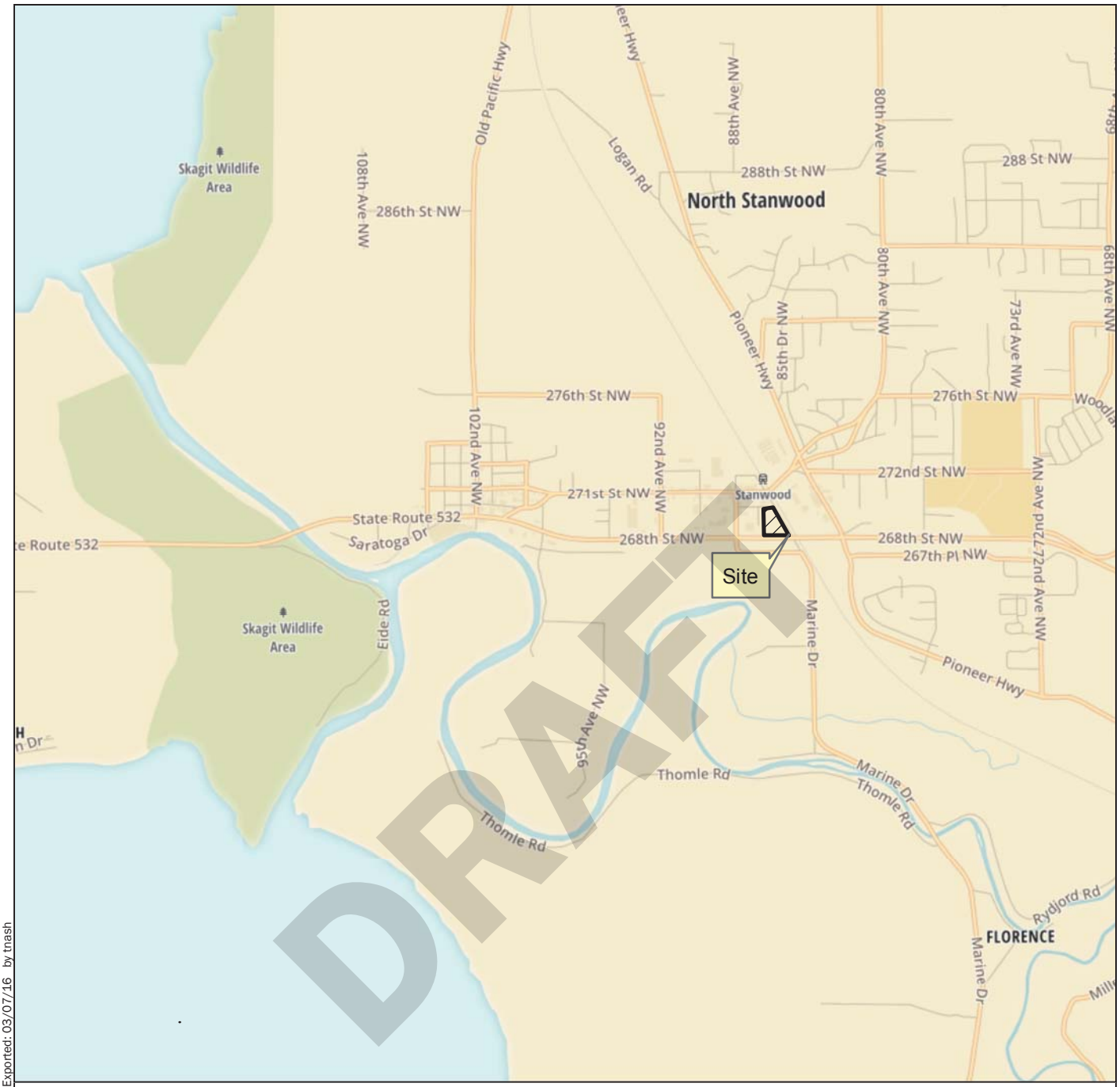

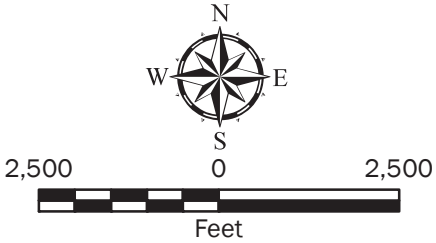

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2015

Projection: NAD 1983 UTM Zone 10N

Vicinity Map

McDay Holdings, Septage Facility
Stanwood, Washington

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Figure 1

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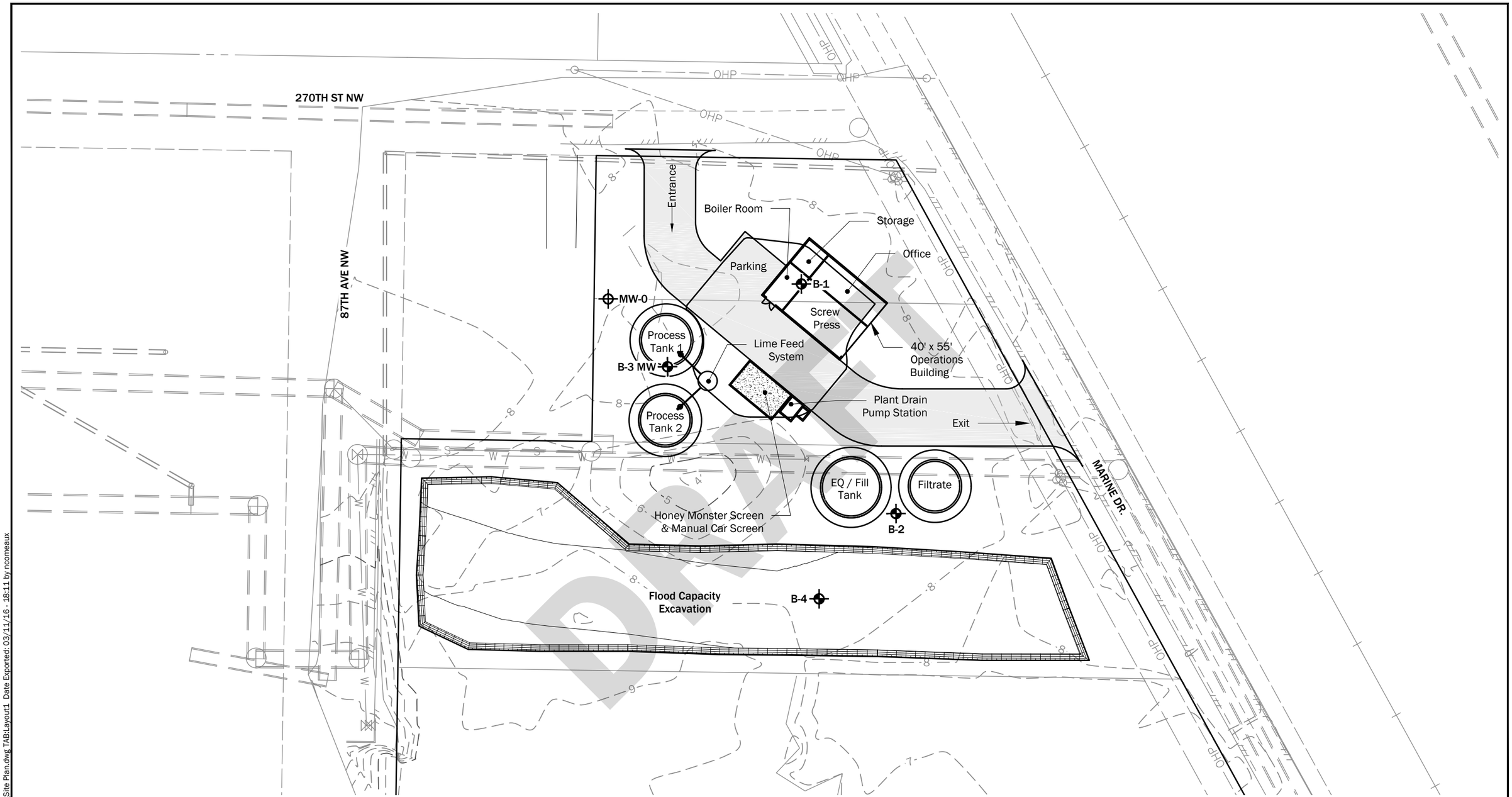

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1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Survey data provided by Wilson Engineering, LLC, dated 10/10/2015.
Site plan provided by Wilson Engineering, LLC, dated 02/02/2016.

Vertical Datum: NAVD88
Horizontal Datum: NAD83, Washington State Plane, North Zone, US Foot

N
W E
S

50 0 50
FEET

Legend

B-1 Approximate Boring Location
MW-0 Approximate Location of Pre-Exisiting Monitoring Well
MW Monitoring Well

Site and Exploration Plan

McDay Holdings, Septage Facility
Stanwood, Washington

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Figure 2

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APPENDIX A
Field Exploration and Laboratory Testing

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APPENDIX A
FIELD EXPLORATION AND LABORATORY TESTING

Field Explorations

Subsurface soil and groundwater conditions were evaluated by drilling four geotechnical borings. The borings were completed to depths of 11½ to 66½ feet below the existing ground surface (bgs) on February 25 and 26, 2016 using a track-mounted drill rig subcontracted to GeoEngineers. The approximate locations of the explorations are shown in Figure 2. The locations of the borings were determined by pacing and taping from existing site features and by recreational grade GPS; therefore, the locations shown in Figure 2 should be considered approximate.

Disturbed soils samples were obtained using Standard Penetration Test (SPT) methodology with the standard split spoon sampler in the borings with a rope and cathead driven 140-pound hammer with 30-inch drop. The samples were placed in plastic bags to maintain the moisture content and transported back to our laboratory for analysis and testing.

The explorations were continuously monitored by a geotechnical engineer from our firm who examined and classified the soils/rock encountered, obtained representative soil/rock samples, observed groundwater conditions and prepared a detailed log of each exploration. Soils were visually classified in general accordance with ASTM International (ASTM) D 2488-90, which is described in Figure A-1. An explanation of our boring log symbols is also shown on Figure A-1.

The logs of the borings are presented in Figures A-2 through A-6. The exploration logs are based on our interpretation of the field and laboratory data and indicate the various types of soils encountered. It also indicates the depths at which these soils or their characteristics change, although the change might actually be gradual. If the change occurred between samples in the boring, it was interpreted.

Observations of groundwater conditions were made during exploration. The groundwater conditions observed are presented on the logs. Groundwater conditions observed during exploration represent a short-term condition and may or may not be representative of the long-term groundwater conditions at the site. A monitoring well was installed in B-3 and water level measured on March 25, 2016 with the water level indicated on the well log.

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Laboratory Testing

Soil samples obtained from the explorations were transported to our laboratory and examined to confirm or modify field classifications, as well as to evaluate index properties of the soil samples. Representative samples were selected for laboratory testing consisting of the determination of the moisture content, Atterberg limits, and consolidation testing. The tests were performed in general accordance with test methods of ASTM or other applicable procedures.

Moisture Content Testing

Moisture content tests were completed in general accordance with ASTM D 2216 for representative samples obtained from the explorations. The results of these tests are presented on the exploration logs at the depths at which the samples were obtained.

Atterberg Limits

Atterberg limit tests were completed for four soil samples. The tests were used to classify the soil as well as to aid in evaluating index properties and consolidation characteristics of the fine-grained soil deposits. The liquid limit and the plastic limit were obtained in general accordance with ASTM D 318. The results of the Atterberg limits are summarized in Figure A-6.

Consolidation Test

One consolidation test was completed on a thin-walled sample collected from the younger alluvial deposits in accordance with ASTM D 2435. The consolidation test is used to estimate the magnitude and rate of consolidation of soil restrained laterally and drained axially while subjected to incrementally applied controlled-stress loading. The test results are shown on Figure A-7.

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SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	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, GRAVEL - 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 SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS
				SP	POORLY-GRADED SANDS, GRAVELLY SAND
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY 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

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	AC	Asphalt Concrete
	CC	Cement Concrete
	CR	Crushed Rock/ Quarry Spalls
	TS	Topsoil/ Forest Duff/Sod

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Sampler Symbol Descriptions

- **2.4-inch I.D. split barrel**
- **Standard Penetration Test (SPT)**
- **Shelby tube**
- **Piston**
- **Direct-Push**
- **Bulk or grab**
- **Continuous Coring**

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

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

A "WOH" indicates sampler pushed using the weight of the hammer.

Laboratory / Field Tests

%F	Percent fines
%G	Percent gravel
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
OC	Organic content
PM	Permeability or hydraulic conductivity
PI	Plasticity index
PP	Pocket penetrometer
PPM	Parts per million
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen
NT	Not Tested

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

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KEY TO EXPLORATION LOGS

GEOENGINEERS

FIGURE A-1

Rev. 02/16

Drilled Start 2/25/2016 End 2/25/2016	Total Depth (ft) 66.5	Logged By KMS Checked By AJH	Driller Boretec1, Inc.	Drilling Method Hollow-Stem Auger
Surface Elevation (ft) 7.5 Vertical Datum NAVD88	Hammer Data 140 (lbs) / 30 (in) Drop		Drilling Equipment Track Mounted Drill Rig	
Latitude Longitude	System Datum Geographic		Groundwater Date Measured / Depth to Water (ft) / Elevation (ft): None observed	
Notes:				

Elevation (feet)	Depth (feet)	Interval Recovered (in)	Blows/foot	Collected Sample Sample Name Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0						ML	Gray silt with sand and occasional organic matter (very soft, moist) (fill)			
5		18	2	1 MC					40		
	5	18	2	2A MC			PT	Dark brown fibrous peat (very soft, wet)	442		
0							ML	Gray silt with trace organic matter (very soft, wet) (alluvium)			
							ML	Gray silt (very soft, wet)			
	10	18	2	3 AL					46		AL (LL = 45, PI = 16)
-5											
	15	18	2	4 MC					41		
-10											
	20	18	2	5 MC					39		
-15											
	25	18	2	6 MC					36		
-20											
	30	18	2	7 AL				Grades to silt with sand	40		AL (LL = 39, PI = 10)

Note: See Figure A-1 for explanation of symbols.

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Log of Boring B-1

GEOENGINEERS

Project: McDay Holdings, Septage Facility
Project Location: Stanwood, Washington
Project Number: 22160-001-00

Figure A-2
Sheet 1 of 2

Elevation (feet)	Depth (feet)	FIELD DATA: Interval Recovered (in)	Blows/foot	Collected Sample: Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
25											
	35	18	2	8 / MC					42		
30											
	40	18	2	9 / MC					43		
35											
	45	18	3	10 / MC				Becomes soft	67		
40											
	50	18	10	11 / MC					66		
45							SM	Dark brown silty fine to medium sand with occasional gravel (medium dense, wet)			
	55	12	18	12			SP-SM	Gray fine to coarse sand with silt and occasional gravel (medium dense, wet)			10 feet of heave from 45 to 55 feet Drilling mud used
50							SP	Gray fine to medium sand with occasional gravel (medium dense, wet)			
	60	12	26	13							
55							SP	Gray fine to coarse sand with occasional gravel (medium dense, wet)			
	65	12	18	14							

Note: See Figure A-1 for explanation of symbols.

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Log of Boring B-1 (continued)

GeoEngineers

Project: McDay Holdings, Septage Facility
Project Location: Stanwood, Washington
Project Number: 22160-001-00

Figure A-2
Sheet 2 of 2

Drilled Start 2/26/2016	End 2/26/2016	Total Depth (ft) 51.5	Logged By MWR / Checked By AJH	Driller Boretec1, Inc.	Drilling Method Hollow-Stem Auger
Surface Elevation (ft) 7.5 / Vertical Datum NAVD88			Hammer Data 140 (lbs) / 30 (in) Drop		Drilling Equipment Track Mounted Drill Rig
Latitude / Longitude			System Datum Geographic		Groundwater: Date Measured / Depth to Water (ft) / Elevation (ft)
Notes:					None observed

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0		3			1			ML	Brown silt with occasional sand and gravel (very soft to soft, moist) (fill)			
5			10	2		2 MC					39		
									PT	Dark brown fibrous peat (very soft to soft, wet)			
	5		18	2		3 MC					270		
0			12	2		4A MC			ML	Gray-blue silt with occasional sand and organic matter (very soft to soft, wet) (alluvium)	369		
	10		18	1		5 MC					46		
-5			18	2		6 MC					43		TV = 1.2 tsf PP = 0.2 tsf
	15		18	12		7 AL					42		AL (LL = 41, PI = 13)
-10													
	20		24			8 AL, consol					42		AL (LL = 37 , PI = 4)
-15													
	25		18	3		9 MC					40		
-20													
	30		18	2		10 MC					42		

Note: See Figure A-1 for explanation of symbols.

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Log of Boring B-2

GEOENGINEERS

Project: McDay Holdings, Septage Facility
Project Location: Stanwood, Washington
Project Number: 22160-001-00

Figure A-3
Sheet 1 of 2

Elevation (feet)	Depth (feet)	FIELD DATA: Interval Recovered (in)	Blows/foot	Collected Sample: Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
25											
	35		3	11 / MC					41		
30											
	40	18	2	12 / MC					49		
35											
	45	18	2	13 / MC					55		
40							SP-SM	Gray fine to coarse sand with silt and gravel (medium dense, wet)			
	50	12	14	14 / MC					14		

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Note: See Figure A-1 for explanation of symbols.

Log of Boring B-2 (continued)

GeoEngineers

Project: McDay Holdings, Septage Facility
Project Location: Stanwood, Washington
Project Number: 22160-001-00

Figure A-3
Sheet 2 of 2

Redmond: Date:4/18/16 Path:W:\PROJECTS\22\22160001\GINT\22160001GPJ DBTemplate/LibTemplate:GEOENGINEERS8.GDT/GEI8_GEOTECH_STANDARD_%F

Drilled Start 2/26/2016	End 2/26/2016	Total Depth (ft) 21.5	Logged By MWR / Checked By AJH	Driller Boretec1, Inc.	Drilling Method Hollow-Stem Auger

Hammer Data	140 (lbs) / 30 (in) Drop	Drilling Equipment	Track Mounted Drill Rig
Surface Elevation (ft)	7.5	Top of Casing Elevation (ft)	
Vertical Datum	NAVD88		
Latitude / Longitude		Horizontal Datum	Geographic

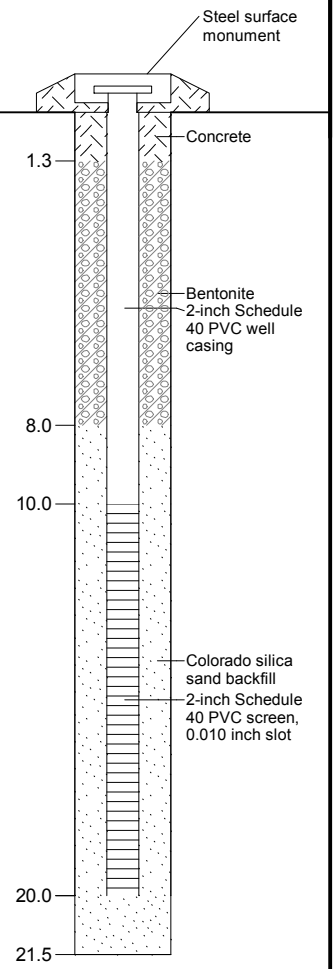
DOE Well I.D.: BIS-604
A 2 (in) well was installed on 2/26/2016 to a depth of 21.5 (ft).

Groundwater		
Date Measured	Depth to Water (ft)	Elevation (ft)
3/25/2016	3.1	4.4

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
	0		3			1			SM	Brown silty fine to medium sand with gravel (loose, moist) (fill)		
5			6	4		2 MC					23	
	5		18	2		3A MC			PT	Dark brown fibrous peat (very soft to soft, wet)	400	
						3B MC			ML	Gray silt with organic matter (very soft, wet) (alluvium)	52	
0			18	2		4 MC					48	
	10		18	2		5 MC				(PP = 0 tsf, TV = 1.1 tsf)	53	
-5									ML	Gray silt with trace shell fragments (very soft, wet)		
	15		18	2		6 MC				(PP = 0.5 tsf, TV = 1.3 tsf)	42	
-10												
	20		18	2		7 MC					37	

WELL LOG



Note: See Figure A-1 for explanation of symbols.

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Log of Monitoring Well B-3

GeoEngineers

Project: McDay Holdings, Septage Facility
Project Location: Stanwood, Washington
Project Number: 22160-001-00

Figure A-4
Sheet 1 of 1

Redmond: Date:4/18/16 Path:W:\PROJECTS\22\22160001\GINT\22160001GPJ DBTemplate\LibTemplate:GEOENGINEERS8.GDT/GEI8_GEOTECH_WELL_%F

Drilled Start 2/26/2016	End 2/26/2016	Total Depth (ft) 11.5	Logged By MWR / Checked By AJH	Driller Boretec1, Inc.	Drilling Method Hollow-Stem Auger
Surface Elevation (ft) 7.75 / Vertical Datum NAVD88			Hammer Data 140 (lbs) / 30 (in) Drop	Drilling Equipment Track Mounted Drill Rig	
Latitude / Longitude			System Datum Geographic	Groundwater: Date Measured / Depth to Water (ft) / Elevation (ft)	
Notes:				See Remarks	

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0								SM	Brown silty fine ot coarse sand with gravel (loose, moist) (fill)			
			3			1							
5			2			2 MC					55		
									ML	Brown silt with occasional sand (soft, wet)			Groundwater observed at approximately 4 feet during drilling
	5		18	3		3A MC					51		
						3B MC			PT	Dark brown fibrous peat (soft, wet)	312		
0			18	2		4A MC 4B MC					312		
									ML	Gray silt with organic matter (very soft to soft, wet) (alluvium)	45		
	10		18	2		5 MC					53		

Note: See Figure A-1 for explanation of symbols.

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Log of Boring B-4

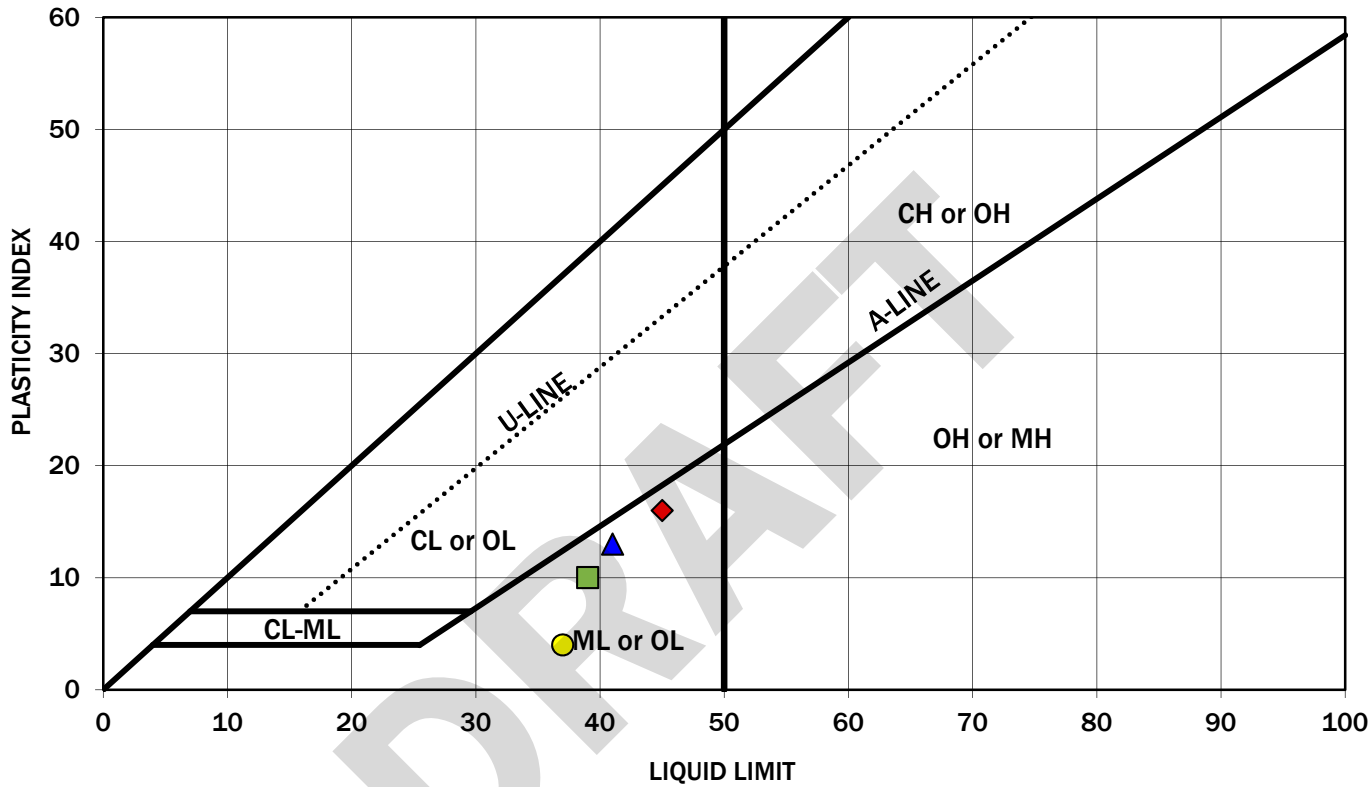
GeoEngineers

Project: McDay Holdings, Septage Facility
Project Location: Stanwood, Washington
Project Number: 22160-001-00

Figure A-5
Sheet 1 of 1

Redmond: Date:4/18/16 Path:W:\PROJECTS\22\22160001\GINT\22160001GPJ DBTemplate/LibTemplate:GEOENGINEERS8.GDT/GEI8_GEOTECH_STANDARD_%F

PLASTICITY CHART



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Symbol	Boring Number	Depth (feet)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Soil Description
◆	B-1	10	46	45	16	Gray silt (ML)
■	B-1	30	40	39	10	Gray silt (ML)
▲	B-2	15	42	41	13	Gray silt (ML)
●	B-2	20	42	37	4	Gray silt (ML)

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The liquid limit and plasticity index were obtained in general accordance with ASTM D 4318.

Atterberg Limits Test Results

McDay Holdings, Septage Facility
Stanwood, Washington

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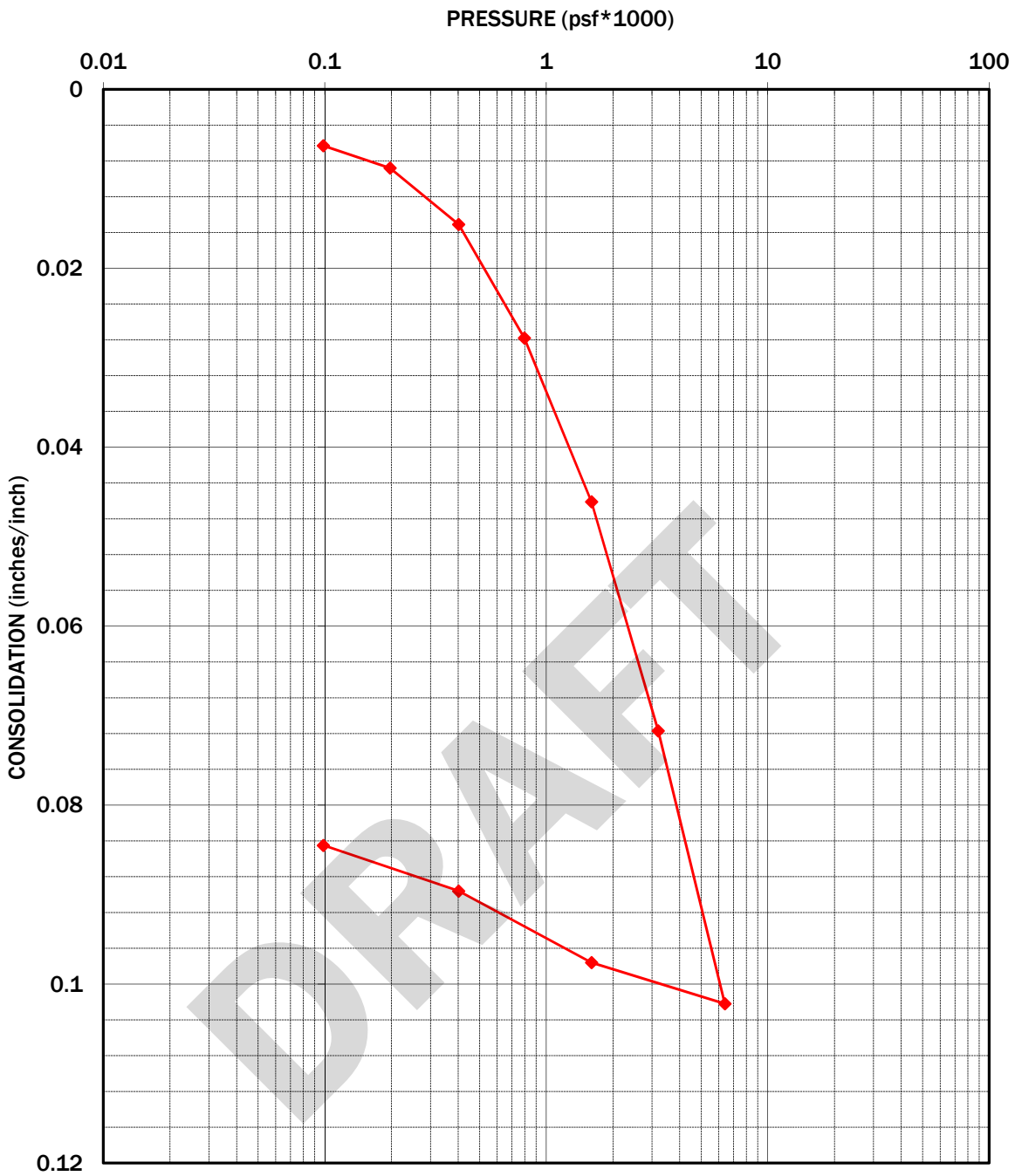
Figure A-6



Boring Number	Depth (feet)	Soil Description	Initial Moisture (%)	Initial Dry Density (lbs/ft^3)
B-2	19.5	Silt (ML)	42.0	80.4

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The consolidation test results were obtained in general accordance with ASTM D 2435.

Consolidation Test Results

McDay Holdings Septage Facility
Stanwood, Washington

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Figure A-7

22160-001-00 Date Exported: 03/11/16

APPENDIX B
Report Limitations and Guidelines for Use

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APPENDIX B
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This appendix provides information to help you manage your risks with respect to the use of this report.

Read These Provisions Closely

It is important to recognize that the geoscience practices (geotechnical engineering, geology and environmental science) rely on professional judgment and opinion to a greater extent than other engineering and natural science disciplines, where more precise and/or readily observable data may exist. To help clients better understand how this difference pertains to our services, GeoEngineers includes the following explanatory "limitations" provisions in its reports. Please confer with GeoEngineers if you need to know more how these "Report Limitations and Guidelines for Use" apply to your project or site.

Geotechnical Services Are Performed for Specific Purposes, Persons and Projects

This report has been prepared for McDay Holdings, LLC and for the Project(s) specifically identified in the report. The information contained herein is not applicable to other sites or projects.

GeoEngineers structures its services to meet the specific needs of its clients. No party other than the party to whom this report is addressed may rely on the product of our services unless we agree to such reliance in advance and in writing. Within the limitations of the agreed scope of services for the Project, and its schedule and budget, our services have been executed in accordance with our Agreement with McDay Holdings, LLC dated November 13, 2015 and authorized November 16, 2015 and generally accepted geotechnical practices in this area at the time this report was prepared. We do not authorize, and will not be responsible for, the use of this report for any purposes or projects other than those identified in the report.

A Geotechnical Engineering or Geologic Report is Based on a Unique Set of Project-Specific Factors

This report has been prepared for the Stanwood Septage Facility project in Stanwood, Washington. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, it is important not to rely on this report if it was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

[1] Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org

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For example, changes that can affect the applicability of this report include those that affect:

- the function of the proposed structure;
- elevation, configuration, location, orientation or weight of the proposed structure;
- composition of the design team; or
- project ownership.

If changes occur after the date of this report, GeoEngineers cannot be responsible for any consequences of such changes in relation to this report unless we have been given the opportunity to review our interpretations and recommendations. Based on that review, we can provide written modifications or confirmation, as appropriate.

Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by man-made events such as construction on or adjacent to the site, new information or technology that becomes available subsequent to the report date, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. If more than a few months have passed since issuance of our report or work product, or if any of the described events may have occurred, please contact GeoEngineers before applying this report for its intended purpose so that we may evaluate whether changed conditions affect the continued reliability or applicability of our conclusions and recommendations.

Geotechnical and Geologic Findings Are Professional Opinions

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies the specific subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied its professional judgment to render an informed opinion about subsurface conditions at other locations. Actual subsurface conditions may differ, sometimes significantly, from the opinions presented in this report. Our report, conclusions and interpretations are not a warranty of the actual subsurface conditions.

Geotechnical Engineering Report Recommendations Are Not Final

The recommendations included in this report are preliminary and should not be considered final. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for the recommendations in this report if we do not perform construction observation.

We recommend that you allow sufficient monitoring, testing and consultation during construction by GeoEngineers to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes if the conditions revealed during the work differ from those anticipated, and to evaluate whether earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective means of managing the risks associated with unanticipated conditions.

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A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

Misinterpretation of this report by members of the design team or by contractors can result in costly problems. GeoEngineers can help reduce the risks of misinterpretation by conferring with appropriate members of the design team after submitting the report, reviewing pertinent elements of the design team's plans and specifications, participating in pre-bid and preconstruction conferences, and providing construction observation.

Do Not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. The logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Photographic or electronic reproduction is acceptable, but separating logs from the report can create a risk of misinterpretation.

Give Contractors a Complete Report and Guidance

To help reduce the risk of problems associated with unanticipated subsurface conditions, GeoEngineers recommends giving contractors the complete geotechnical engineering or geologic report, including these "Report Limitations and Guidelines for Use." When providing the report, you should preface it with a clearly written letter of transmittal that:

- advises contractors that the report was not prepared for purposes of bid development and that its accuracy is limited; and
- encourages contractors to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer.

Contractors Are Responsible for Site Safety on Their Own Construction Projects

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and adjacent properties.

Biological Pollutants

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings or conclusions regarding the detecting, assessing, preventing or abating of Biological Pollutants, and no conclusions or inferences should be drawn regarding Biological Pollutants as they may relate to this project. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria and viruses, and/or any of their byproducts.

A Client that desires these specialized services is advised to obtain them from a consultant who offers services in this specialized field.

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APPENDIX B

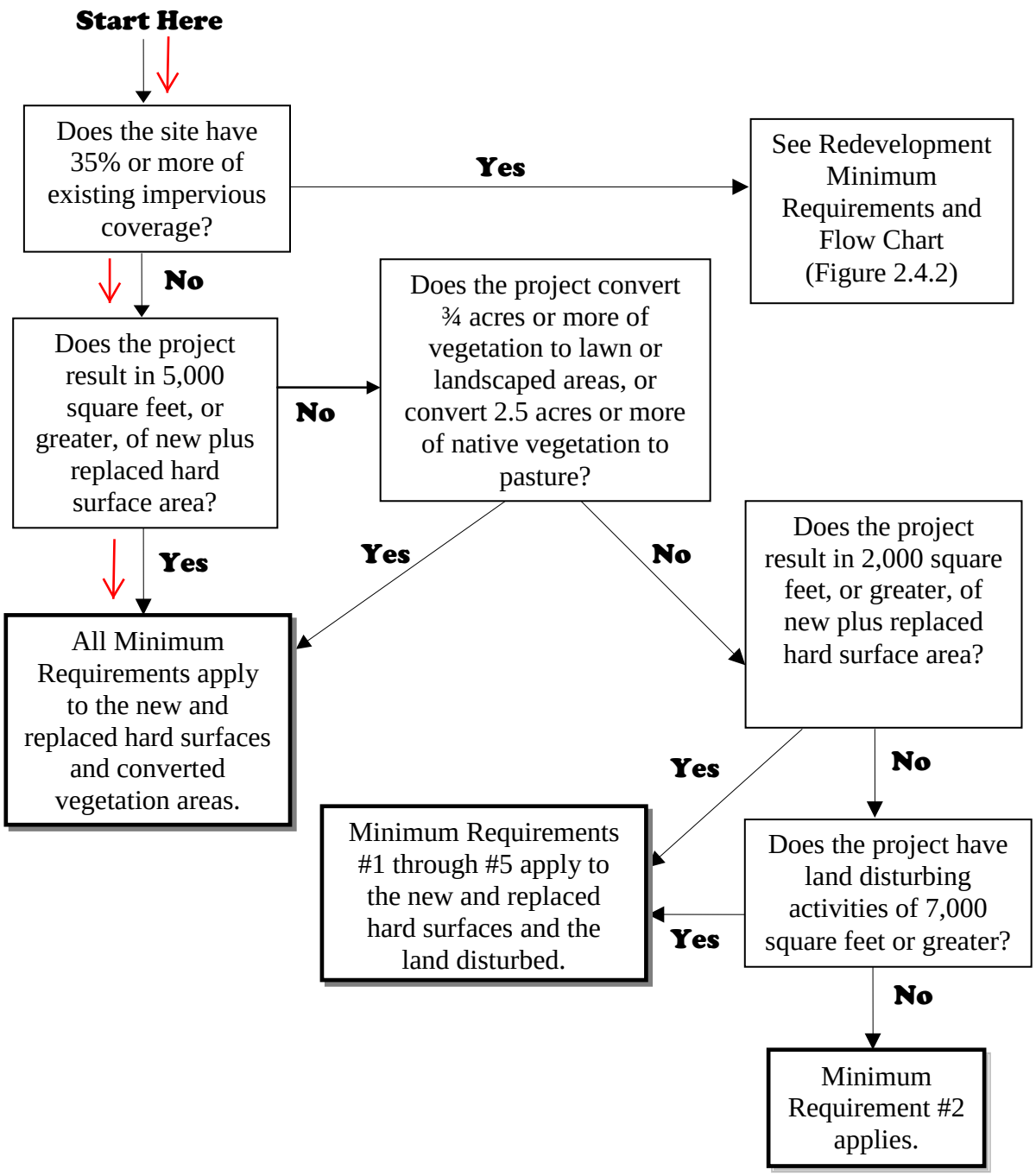
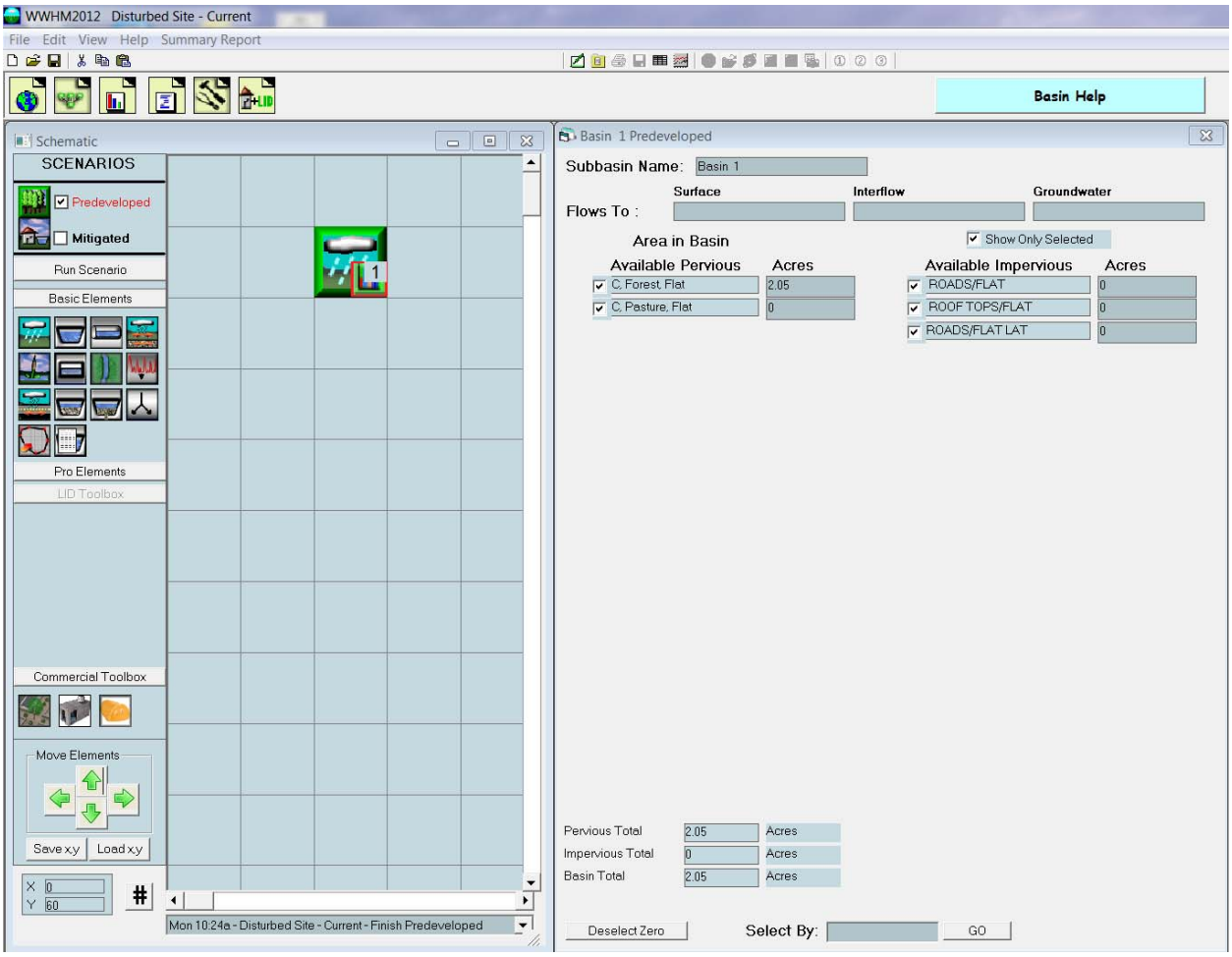


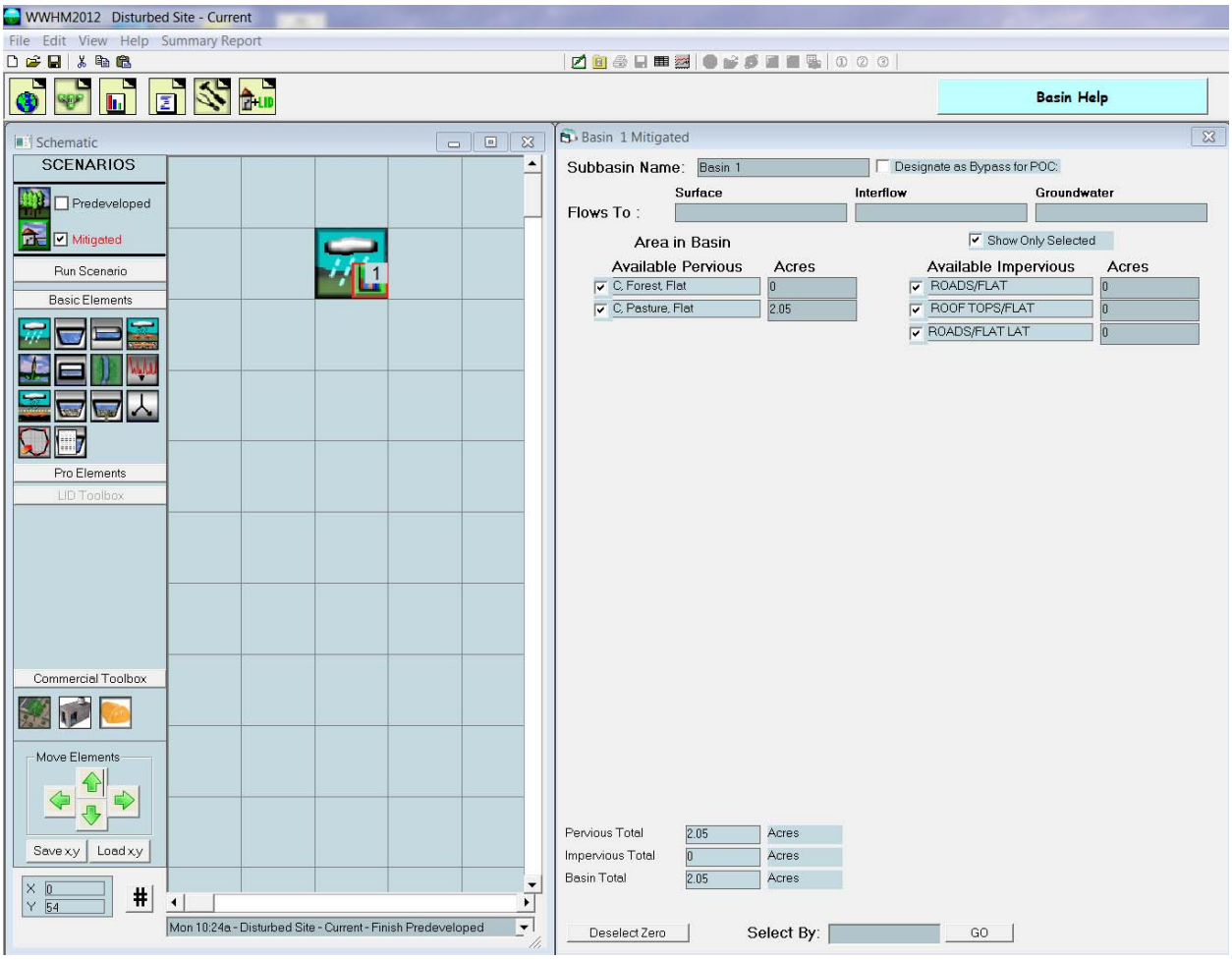
Figure 2.4.1 – Flow Chart for Determining Requirements for New Development

Existing Flowrates for Site

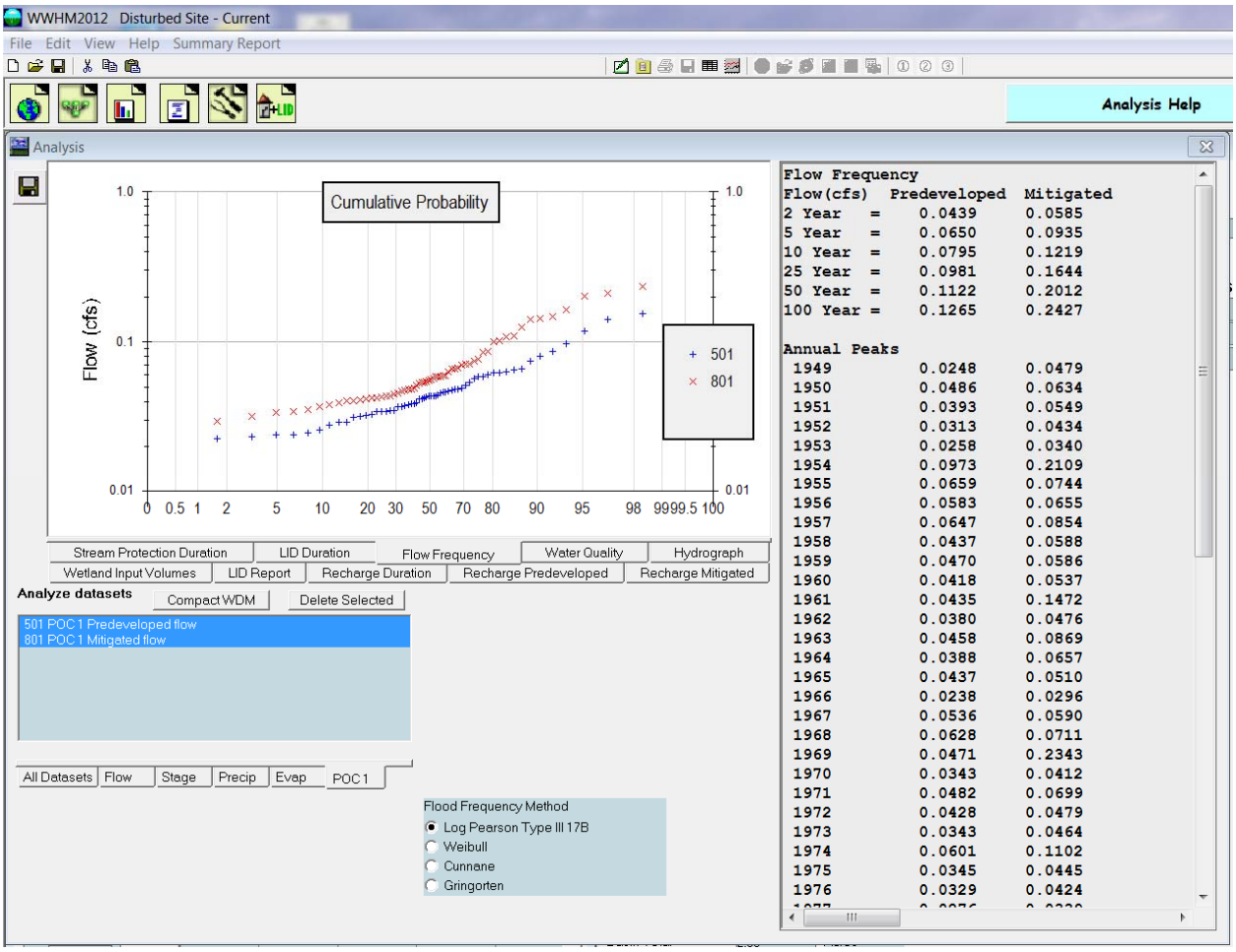
Total site is 3.3918 acres in size, and currently is pasture/cleared land with grasses and shrubs. The total area being disturbed by development for this project is 2.05 acres. Predevelopment assumes all forested land.



Post Development site showing pasture (cleared land with pioneering vegetation)

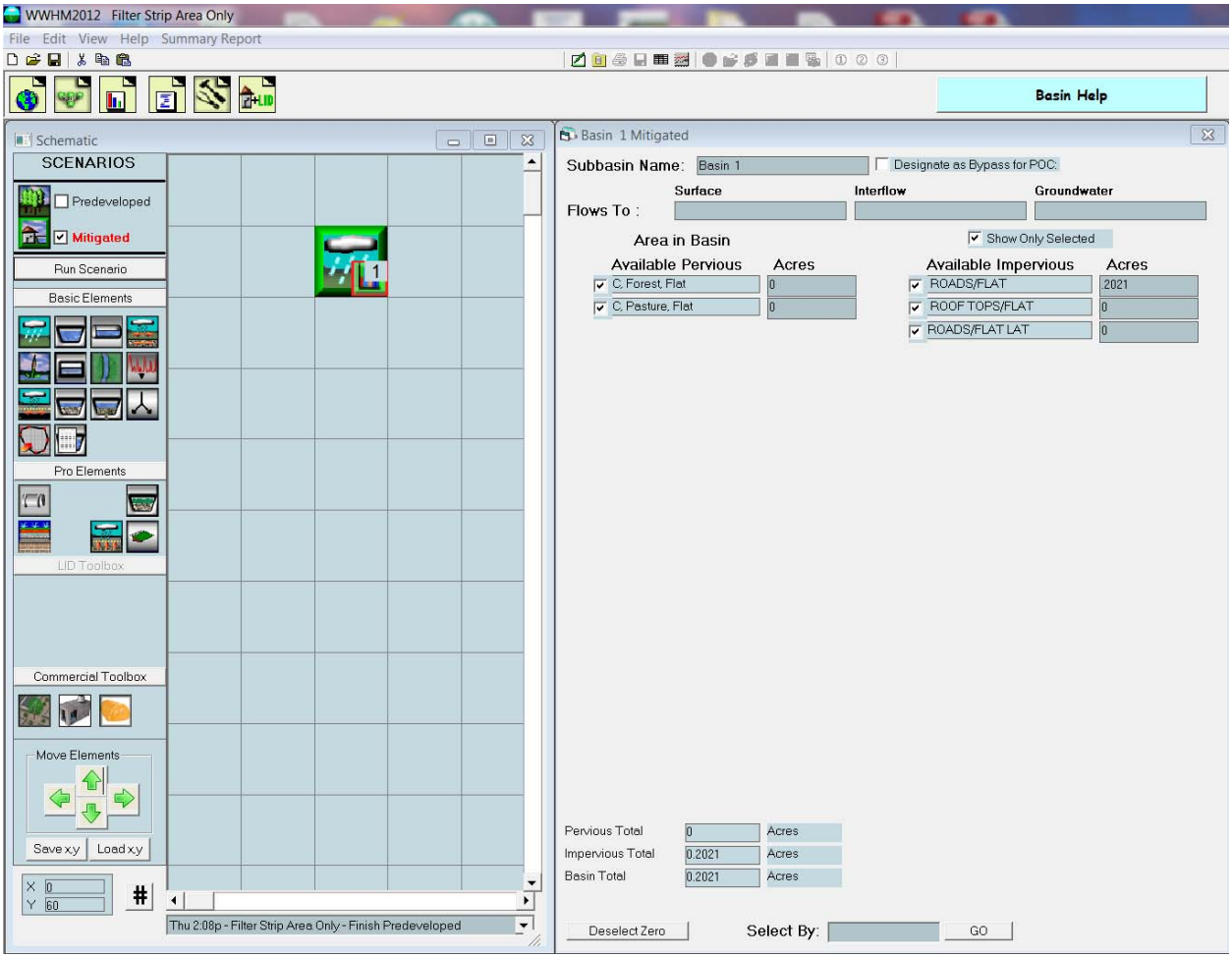


Flowrates for entire site under existing conditions (pasture land).

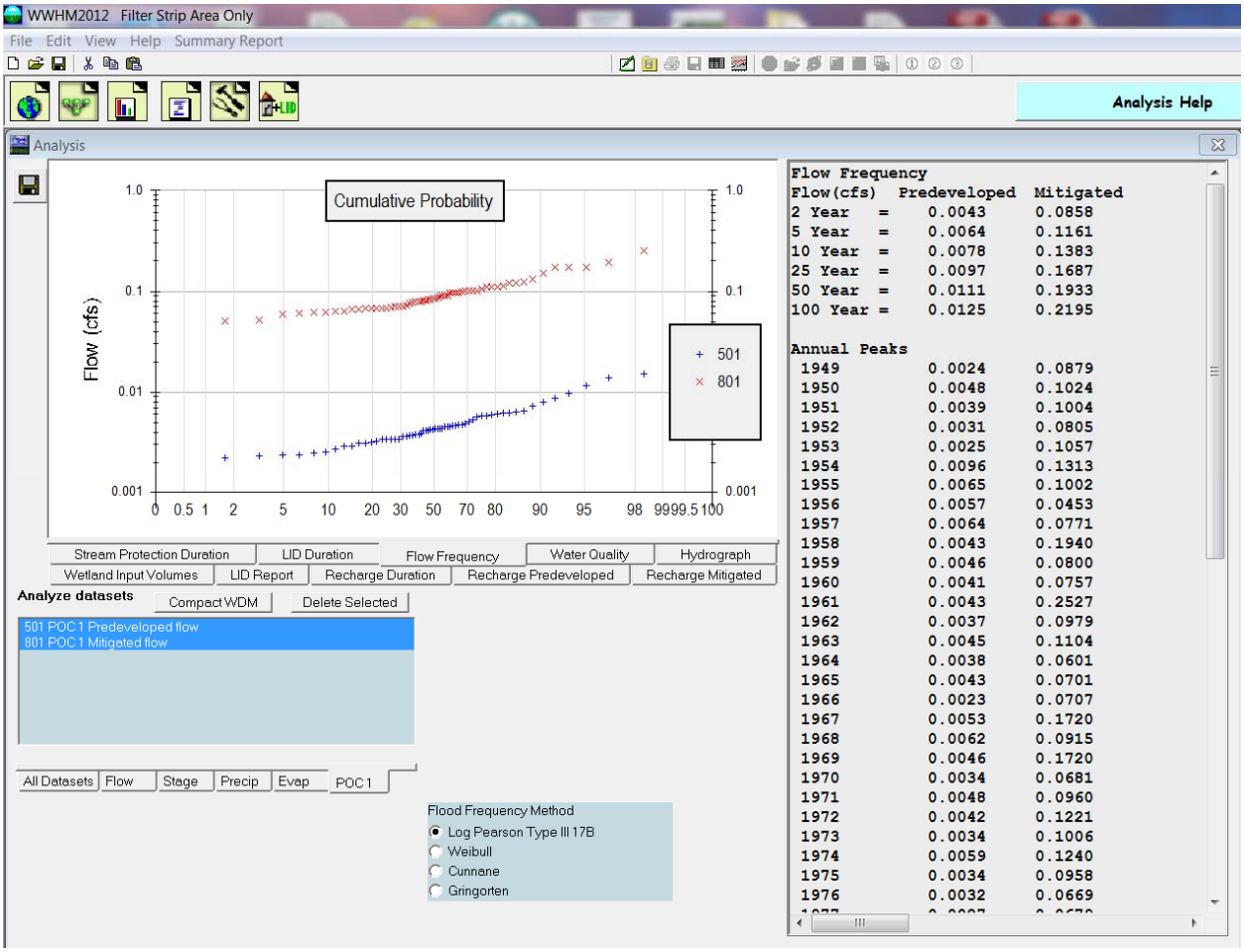


For Water Quality (Filter Strip)

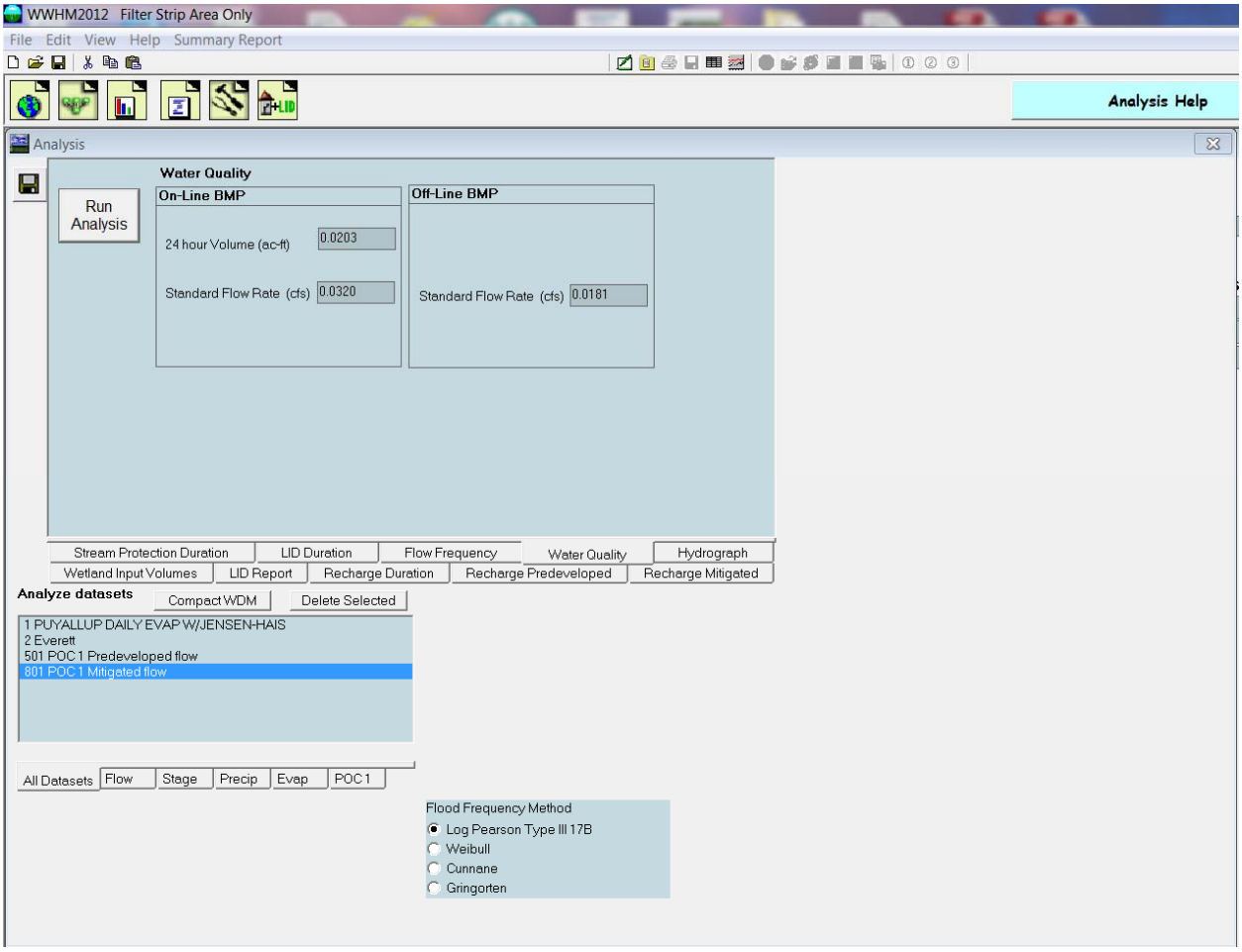
Pavement areas are only pollution generating surfaces. The total pavement area is 0.2021 acres.



Flowrates for the pavement area as shown below as “Mitigated”.

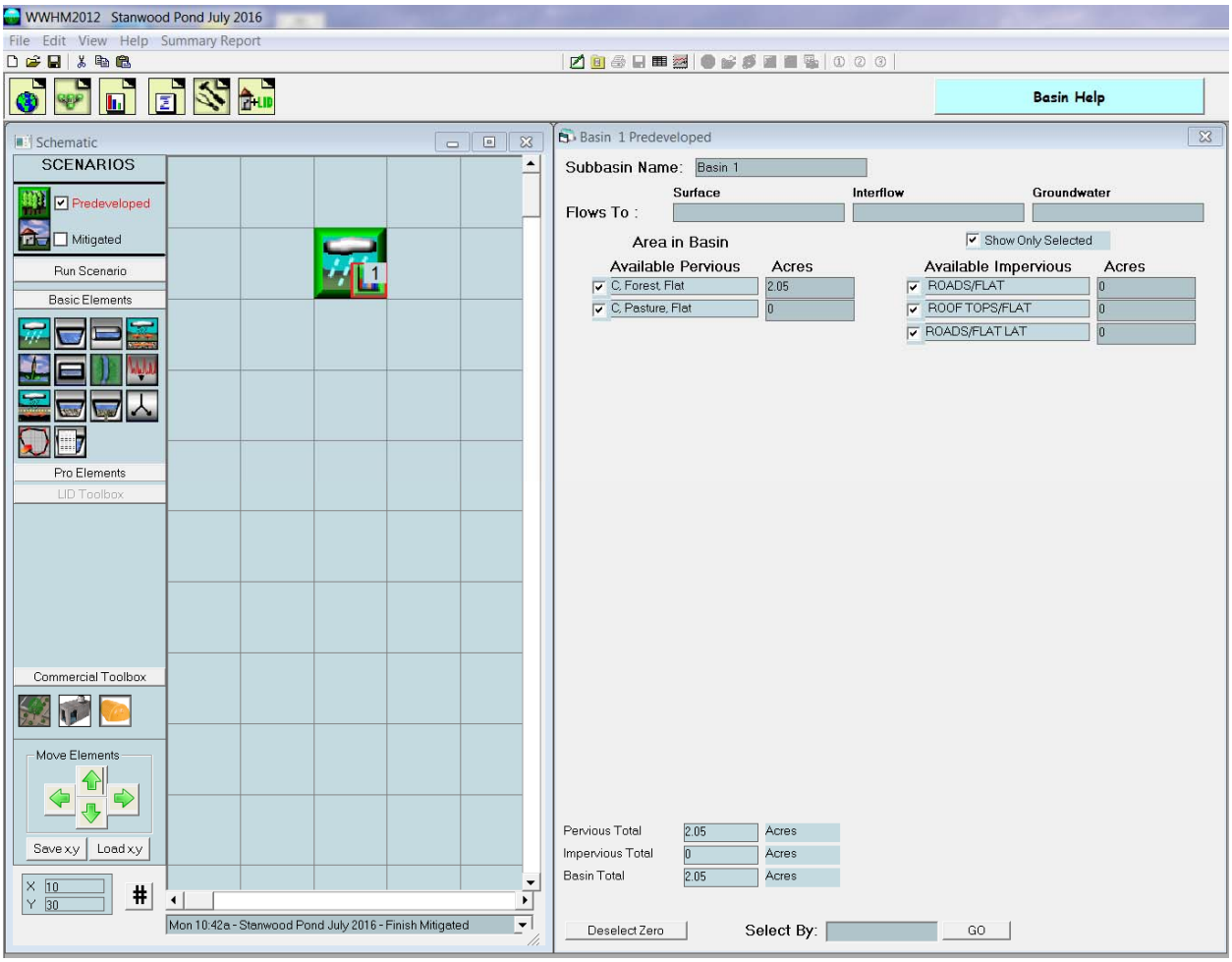


Water quality flow rates for pavement area only.

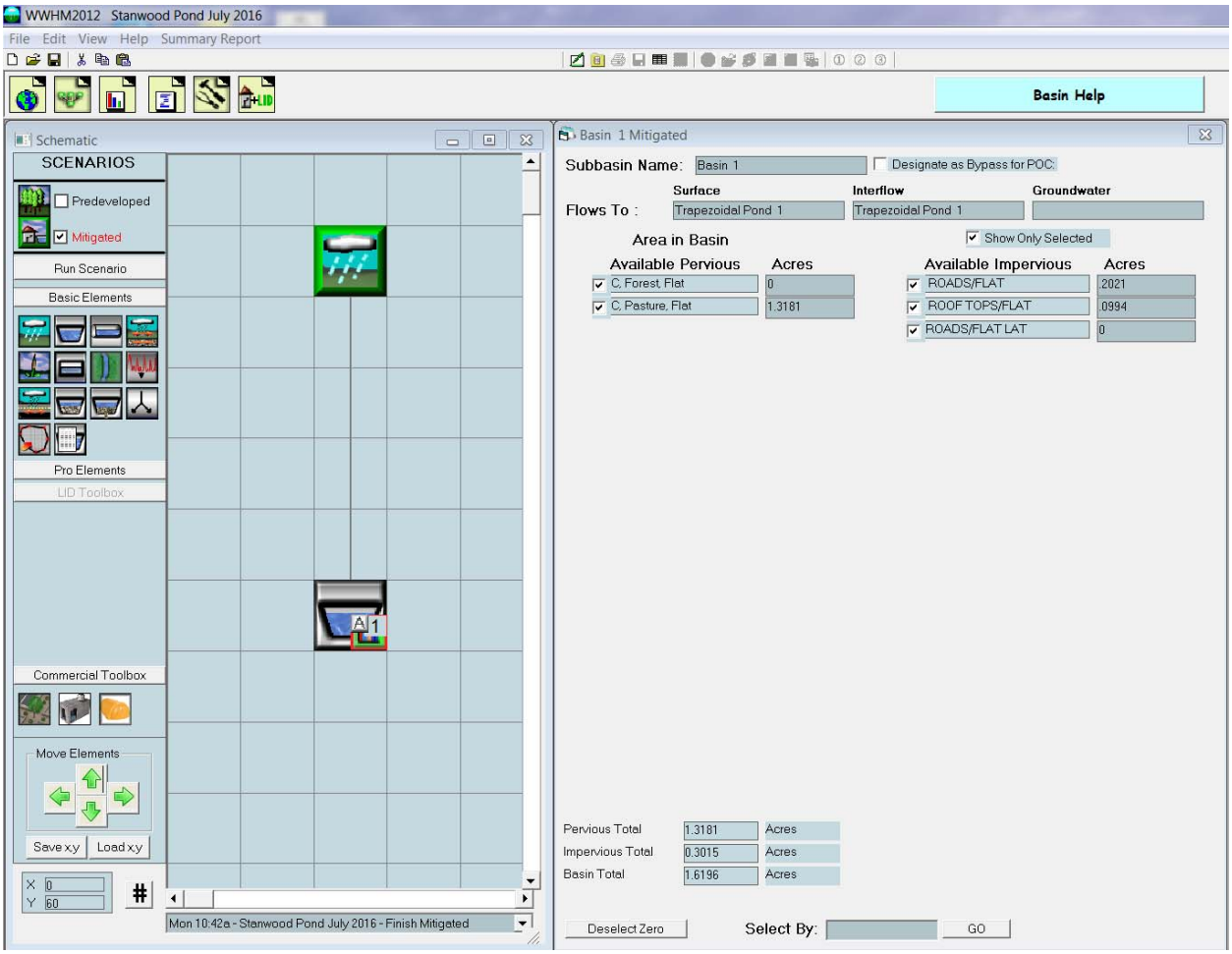


For Flow Control (Detention Pond)

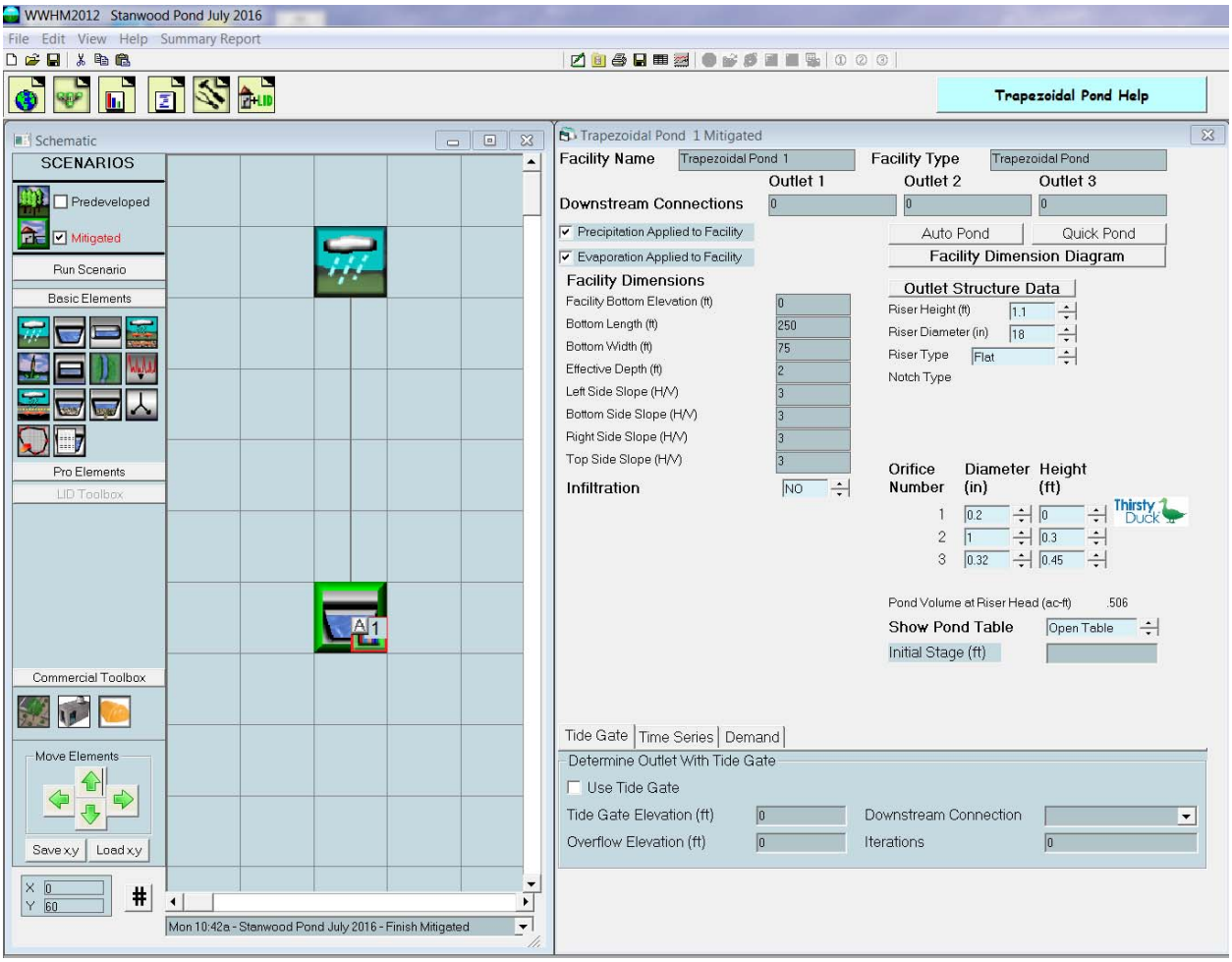
Total disturbed portion of the site is 2.05 acres in size. Predevelopment assumes all forested land.



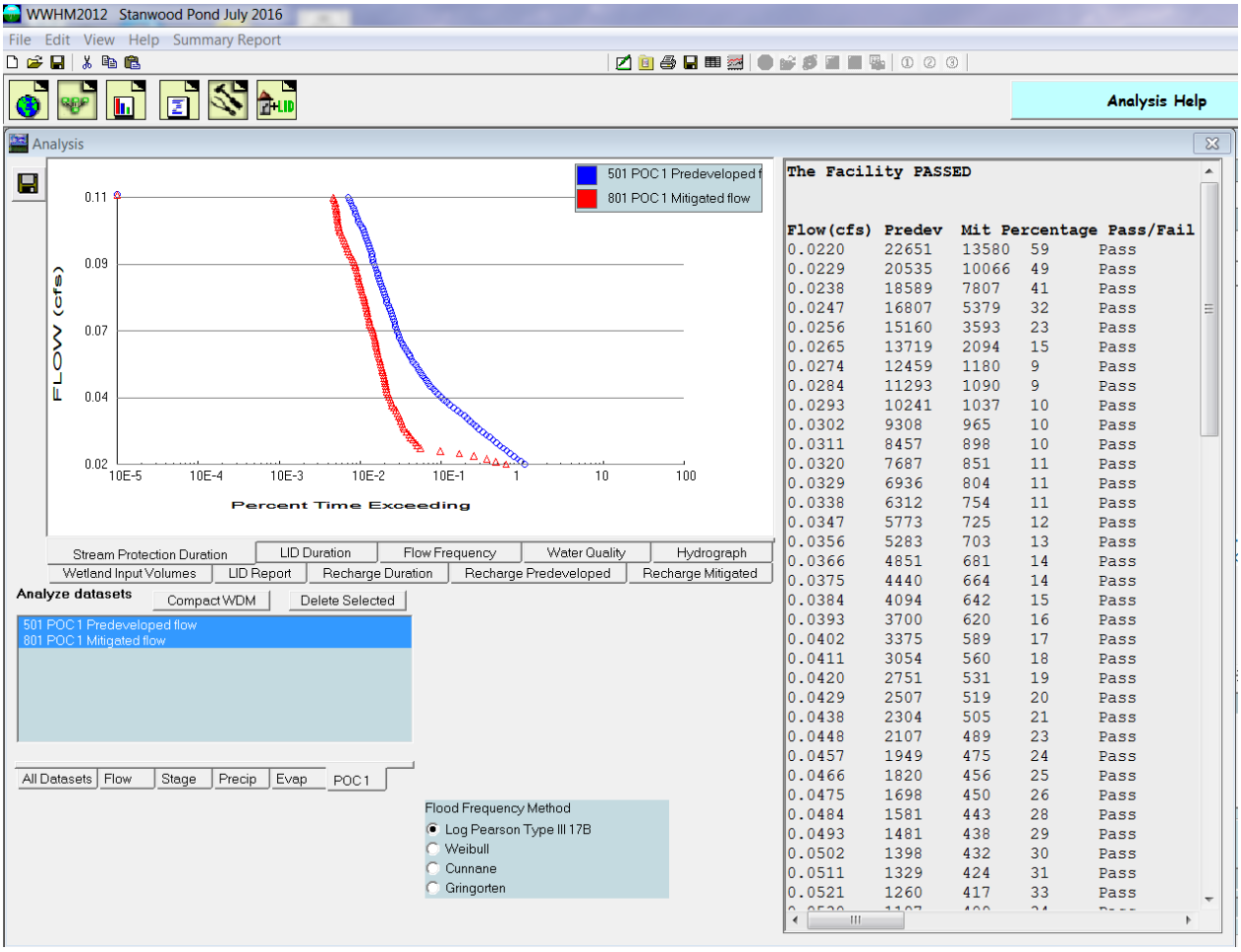
Developed site will include roads, roofs and pasture land. Since the site will not have significant areas of landscaping, a majority of the site will be maintained “as is”, which is cleared with grasses and shrubs which have naturally grown in the area. This is considered “pature” for modeling. Note the detention pond area is not included as it has “precipiation applied” as part of its modeling.



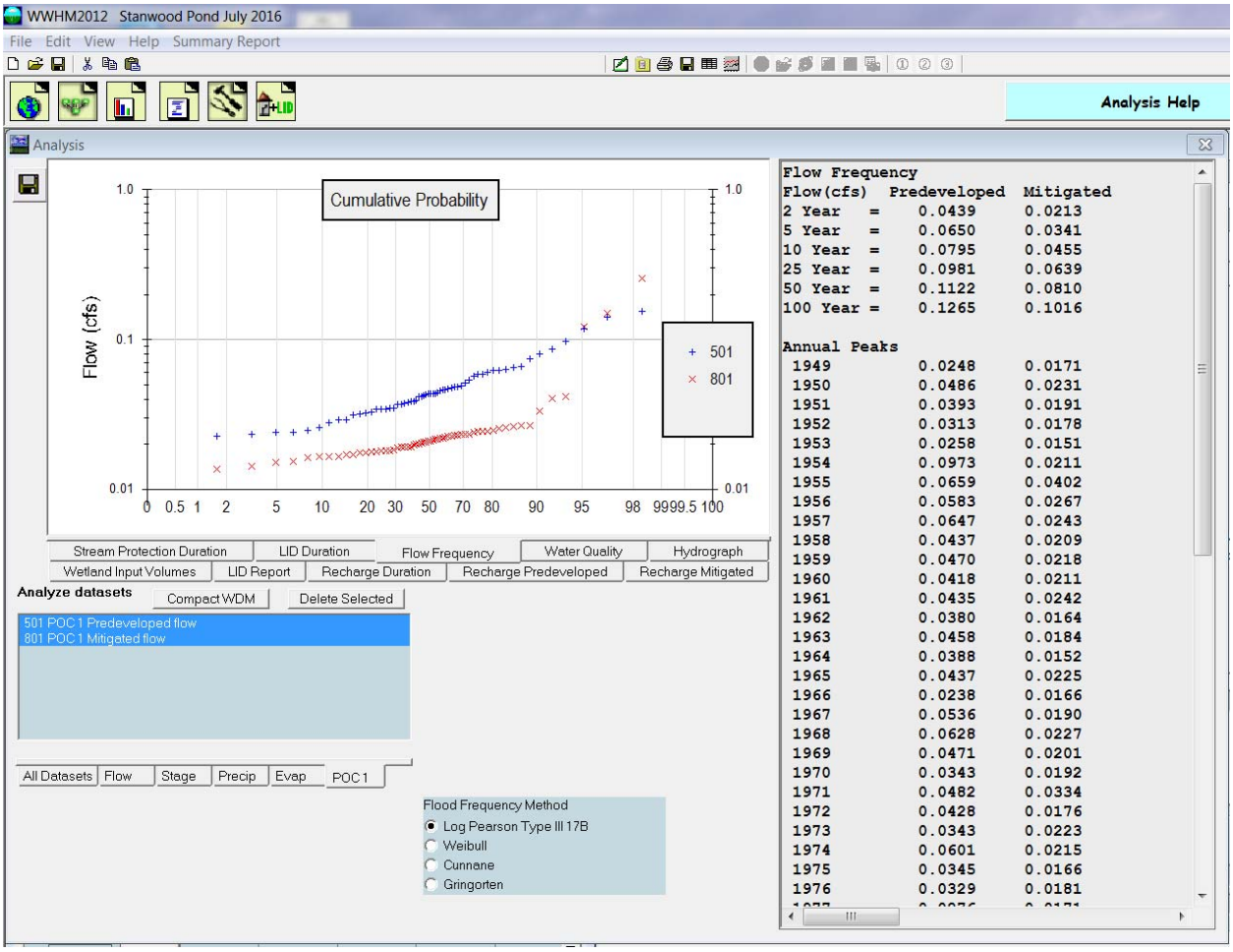
The detention pond is modeled as receiving both precipitation and evaporation. Modeling the pond receiving precipitation is equivalent to modeling the pond as a detention pond surface. Note the orifice sizes of the 1st and 3rd orifice are too small for construction. Department of Ecology requires the orifice sizes to be no less than 0.5 inches in diameter. The model has used these smaller orifice sizes in order to ensure a pond with the required detention volume.



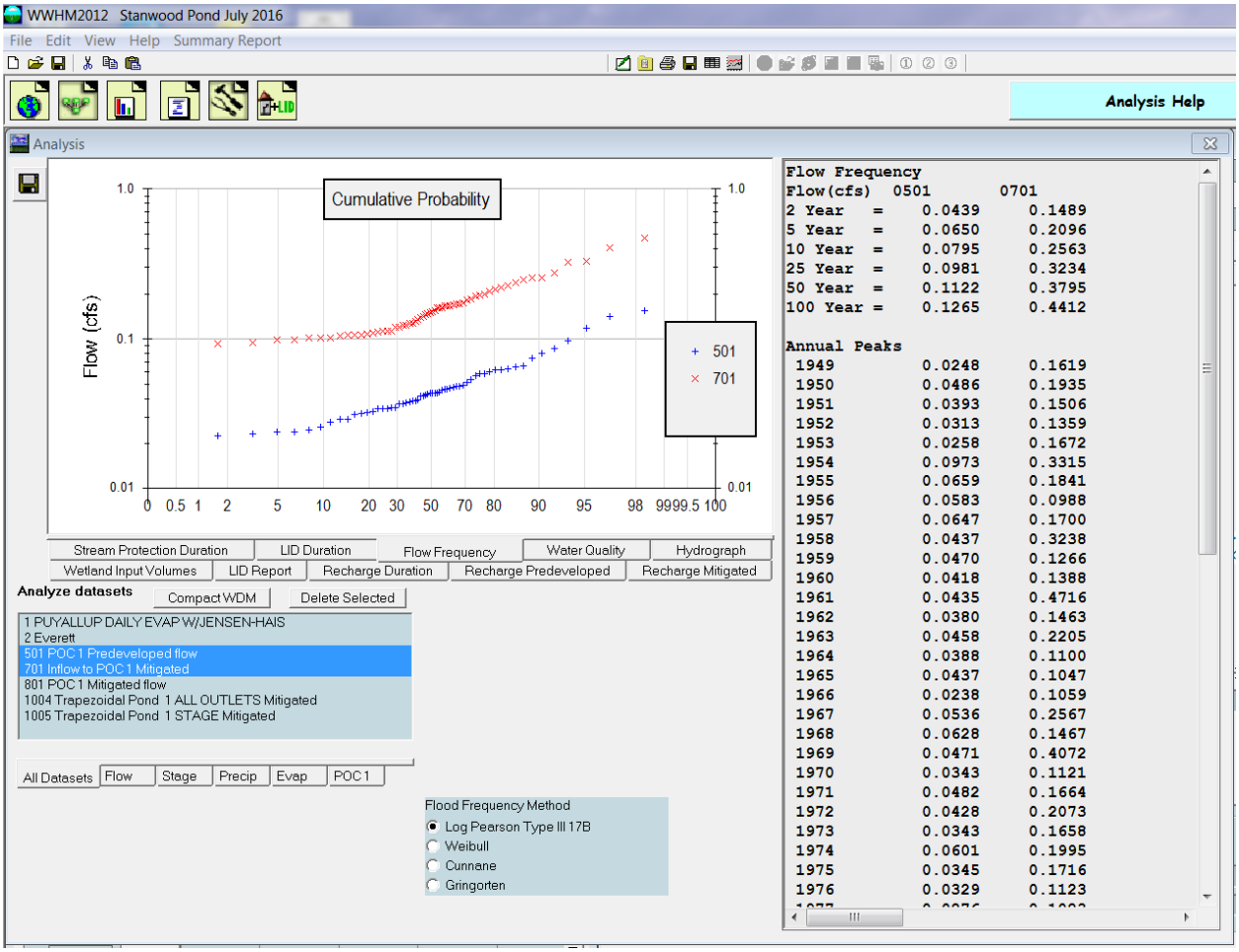
The detention pond meets the predevelopment conditions.



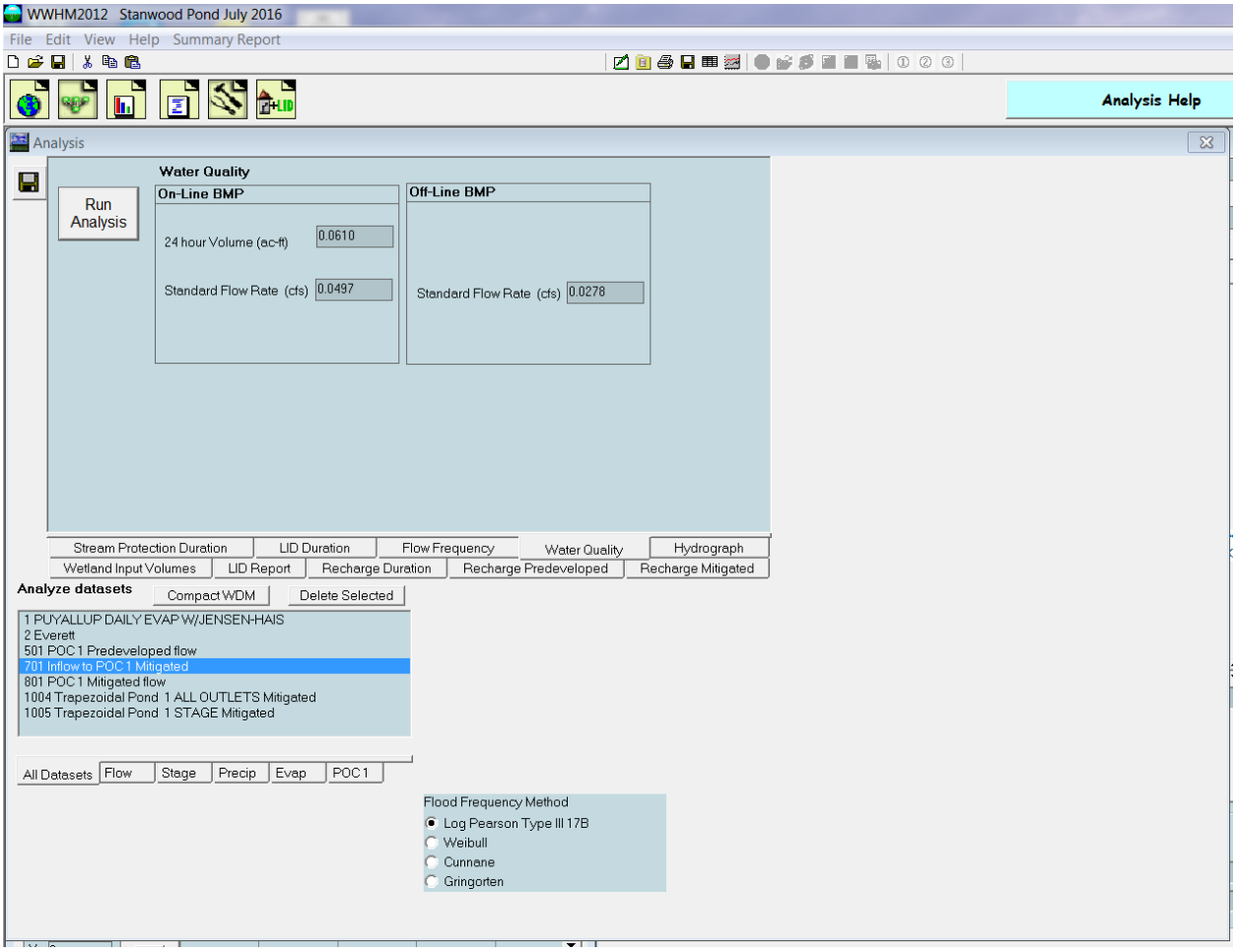
Flowrates leaving the site after the detention pond (for mitigated scenario).



Flowrates TO the detention pond (before detention) are shown as “0701 Inflow to POC Mitigated” (note “0501” is the predevelopment flow).



Water quality flowrate (measured before inflow into detention pond).



WWHM2012
PROJECT REPORT

Project Name: Stanwood Pond July 2016
Site Name: Stanwood Septage
Site Address:
City :
Report Date: 7/18/2016
Gage : Everett
Data Start : 1948/10/01
Data End : 2009/09/30
Precip Scale: 1.00
Version Date: 2016/02/25
Version : 4.2.12

Low Flow Threshold for POC 1 : 50 Percent of the 2 Year

High Flow Threshold for POC 1: 50 year

PREDEVELOPED LAND USE

Name : Basin 1
Bypass: No

GroundWater: No

Pervious Land Use	acre
C, Forest, Flat	2.05
Pervious Total	2.05

Impervious Land Use	acre
Impervious Total	0
Basin Total	2.05

Element Flows To:

Surface	Interflow	Groundwater

MITIGATED LAND USE

Name : Basin 1
Bypass: No

GroundWater: No

Pervious Land Use	acre
C, Pasture, Flat	1.3181
Pervious Total	1.3181

Impervious Land Use	acre
ROADS FLAT	0.2021
ROOF TOPS FLAT	0.0994
Impervious Total	0.3015
Basin Total	1.6196

Element Flows To:

Surface	Interflow	Groundwater
Trapezoidal Pond 1	Trapezoidal Pond 1	

Name : Trapezoidal Pond 1
Bottom Length: 250.00 ft.
Bottom Width: 75.00 ft.
Depth: 2 ft.
Volume at riser head: 0.5063 acre-feet.
Side slope 1: 3 To 1
Side slope 2: 3 To 1
Side slope 3: 3 To 1
Side slope 4: 3 To 1
Discharge Structure
Riser Height: 1.1 ft.
Riser Diameter: 18 in.
Orifice 1 Diameter: 0.2 in. **Elevation:** 0 ft.
Orifice 2 Diameter: 1 in. **Elevation:** 0.3 ft.
Orifice 3 Diameter: 0.32 in. **Elevation:** 0.45 ft.

Element Flows To:

Outlet 1	Outlet 2

Pond Hydraulic Table

Stage(feet)	Area(ac.)	Volume(ac-ft.)	Discharge(cfs)	Infilt(cfs)
0.0000	0.430	0.000	0.000	0.000
0.0222	0.431	0.009	0.000	0.000
0.0444	0.432	0.019	0.000	0.000
0.0667	0.433	0.028	0.000	0.000
0.0889	0.434	0.038	0.000	0.000
0.1111	0.435	0.048	0.000	0.000
0.1333	0.436	0.057	0.000	0.000

0.1556	0.437	0.067	0.000	0.000
0.1778	0.438	0.077	0.000	0.000
0.2000	0.439	0.087	0.000	0.000
0.2222	0.440	0.096	0.000	0.000
0.2444	0.441	0.106	0.000	0.000
0.2667	0.442	0.116	0.000	0.000
0.2889	0.443	0.126	0.000	0.000
0.3111	0.444	0.136	0.003	0.000
0.3333	0.445	0.146	0.005	0.000
0.3556	0.446	0.155	0.007	0.000
0.3778	0.447	0.165	0.008	0.000
0.4000	0.448	0.175	0.009	0.000
0.4222	0.449	0.185	0.010	0.000
0.4444	0.450	0.195	0.011	0.000
0.4667	0.451	0.205	0.012	0.000
0.4889	0.452	0.215	0.013	0.000
0.5111	0.453	0.225	0.013	0.000
0.5333	0.454	0.236	0.014	0.000
0.5556	0.455	0.246	0.015	0.000
0.5778	0.456	0.256	0.016	0.000
0.6000	0.457	0.266	0.016	0.000
0.6222	0.458	0.276	0.017	0.000
0.6444	0.459	0.286	0.018	0.000
0.6667	0.460	0.297	0.018	0.000
0.6889	0.461	0.307	0.019	0.000
0.7111	0.462	0.317	0.019	0.000
0.7333	0.463	0.327	0.020	0.000
0.7556	0.464	0.338	0.020	0.000
0.7778	0.465	0.348	0.021	0.000
0.8000	0.466	0.358	0.021	0.000
0.8222	0.467	0.369	0.022	0.000
0.8444	0.468	0.379	0.022	0.000
0.8667	0.469	0.390	0.023	0.000
0.8889	0.470	0.400	0.023	0.000
0.9111	0.471	0.411	0.024	0.000
0.9333	0.472	0.421	0.024	0.000
0.9556	0.474	0.432	0.025	0.000
0.9778	0.475	0.442	0.025	0.000
1.0000	0.476	0.453	0.025	0.000
1.0222	0.477	0.463	0.026	0.000
1.0444	0.478	0.474	0.026	0.000
1.0667	0.479	0.484	0.027	0.000
1.0889	0.480	0.495	0.027	0.000
1.1111	0.481	0.506	0.046	0.000
1.1333	0.482	0.517	0.125	0.000
1.1556	0.483	0.527	0.236	0.000
1.1778	0.484	0.538	0.373	0.000
1.2000	0.485	0.549	0.531	0.000
1.2222	0.486	0.560	0.707	0.000
1.2444	0.487	0.570	0.899	0.000
1.2667	0.488	0.581	1.104	0.000
1.2889	0.489	0.592	1.322	0.000
1.3111	0.490	0.603	1.550	0.000
1.3333	0.491	0.614	1.787	0.000
1.3556	0.492	0.625	2.031	0.000
1.3778	0.493	0.636	2.281	0.000
1.4000	0.494	0.647	2.533	0.000

1.4222	0.495	0.658	2.788	0.000
1.4444	0.496	0.669	3.042	0.000
1.4667	0.497	0.680	3.294	0.000
1.4889	0.498	0.691	3.543	0.000
1.5111	0.500	0.702	3.787	0.000
1.5333	0.501	0.713	4.023	0.000
1.5556	0.502	0.724	4.250	0.000
1.5778	0.503	0.735	4.468	0.000
1.6000	0.504	0.747	4.674	0.000
1.6222	0.505	0.758	4.868	0.000
1.6444	0.506	0.769	5.048	0.000
1.6667	0.507	0.780	5.214	0.000
1.6889	0.508	0.792	5.367	0.000
1.7111	0.509	0.803	5.505	0.000
1.7333	0.510	0.814	5.629	0.000
1.7556	0.511	0.826	5.740	0.000
1.7778	0.512	0.837	5.840	0.000
1.8000	0.513	0.848	5.930	0.000
1.8222	0.514	0.860	6.013	0.000
1.8444	0.515	0.871	6.091	0.000
1.8667	0.516	0.883	6.243	0.000
1.8889	0.517	0.894	6.333	0.000
1.9111	0.519	0.906	6.421	0.000
1.9333	0.520	0.917	6.508	0.000
1.9556	0.521	0.929	6.594	0.000
1.9778	0.522	0.941	6.679	0.000
2.0000	0.523	0.952	6.763	0.000
2.0222	0.524	0.964	6.846	0.000

ANALYSIS RESULTS

Stream Protection Duration

Predeveloped Landuse Totals for POC #1
Total Pervious Area:2.05
Total Impervious Area:0

Mitigated Landuse Totals for POC #1
Total Pervious Area:1.3181
Total Impervious Area:0.3015

Flow Frequency Return Periods for Predeveloped. POC #1

Return Period	Flow(cfs)
2 year	0.043944
5 year	0.065043
10 year	0.079489
25 year	0.09812
50 year	0.112215
100 year	0.126463

Flow Frequency Return Periods for Mitigated. POC #1

Return Period	Flow(cfs)
2 year	0.021286
5 year	0.034109
10 year	0.045484
25 year	0.063876
50 year	0.081024
100 year	0.101622

Stream Protection Duration
Annual Peaks for Predeveloped and Mitigated. POC #1

Year	Predeveloped	Mitigated
1949	0.025	0.017
1950	0.049	0.023
1951	0.039	0.019
1952	0.031	0.018
1953	0.026	0.015
1954	0.097	0.021
1955	0.066	0.040
1956	0.058	0.027
1957	0.065	0.024
1958	0.044	0.021
1959	0.047	0.022
1960	0.042	0.021
1961	0.043	0.024
1962	0.038	0.016
1963	0.046	0.018
1964	0.039	0.015
1965	0.044	0.022
1966	0.024	0.017
1967	0.054	0.019
1968	0.063	0.023
1969	0.047	0.020
1970	0.034	0.019
1971	0.048	0.033
1972	0.043	0.018
1973	0.034	0.022
1974	0.060	0.021
1975	0.035	0.017
1976	0.033	0.018
1977	0.028	0.017
1978	0.034	0.018
1979	0.062	0.017
1980	0.039	0.018
1981	0.032	0.016
1982	0.042	0.022
1983	0.059	0.020
1984	0.043	0.026
1985	0.057	0.026
1986	0.141	0.150
1987	0.063	0.041
1988	0.035	0.020
1989	0.029	0.014
1990	0.045	0.023
1991	0.048	0.022
1992	0.037	0.023

1993	0.024	0.014
1994	0.022	0.021
1995	0.046	0.025
1996	0.080	0.025
1997	0.154	0.256
1998	0.029	0.019
1999	0.042	0.021
2000	0.023	0.023
2001	0.008	0.010
2002	0.044	0.023
2003	0.032	0.018
2004	0.051	0.024
2005	0.037	0.020
2006	0.087	0.027
2007	0.074	0.024
2008	0.118	0.121
2009	0.037	0.019

Stream Protection Duration
Ranked Annual Peaks for Predeveloped and Mitigated. POC #1

Rank	Predeveloped	Mitigated
1	0.1536	0.2563
2	0.1406	0.1497
3	0.1176	0.1214
4	0.0973	0.0414
5	0.0868	0.0402
6	0.0805	0.0334
7	0.0739	0.0267
8	0.0659	0.0267
9	0.0647	0.0263
10	0.0628	0.0259
11	0.0625	0.0253
12	0.0619	0.0248
13	0.0601	0.0243
14	0.0588	0.0242
15	0.0583	0.0242
16	0.0570	0.0239
17	0.0536	0.0233
18	0.0508	0.0232
19	0.0486	0.0231
20	0.0484	0.0230
21	0.0482	0.0227
22	0.0471	0.0225
23	0.0470	0.0225
24	0.0463	0.0223
25	0.0458	0.0218
26	0.0455	0.0218
27	0.0444	0.0217
28	0.0437	0.0215
29	0.0437	0.0211
30	0.0435	0.0211
31	0.0432	0.0209
32	0.0428	0.0207
33	0.0422	0.0206
34	0.0418	0.0202
35	0.0416	0.0201

36	0.0393	0.0201
37	0.0388	0.0196
38	0.0387	0.0192
39	0.0380	0.0191
40	0.0374	0.0191
41	0.0369	0.0190
42	0.0369	0.0190
43	0.0346	0.0184
44	0.0345	0.0181
45	0.0344	0.0180
46	0.0343	0.0180
47	0.0343	0.0178
48	0.0329	0.0177
49	0.0323	0.0176
50	0.0316	0.0174
51	0.0313	0.0171
52	0.0292	0.0171
53	0.0291	0.0166
54	0.0276	0.0166
55	0.0258	0.0164
56	0.0248	0.0163
57	0.0241	0.0152
58	0.0238	0.0151
59	0.0233	0.0142
60	0.0225	0.0135
61	0.0076	0.0097

Stream Protection Duration
POC #1
The Facility PASSED

The Facility PASSED.

Flow(cfs)	Predev	Mit	Percentage	Pass/Fail
0.0220	22651	13580	59	Pass
0.0229	20535	10066	49	Pass
0.0238	18589	7807	41	Pass
0.0247	16807	5379	32	Pass
0.0256	15160	3593	23	Pass
0.0265	13719	2094	15	Pass
0.0274	12459	1180	9	Pass
0.0284	11293	1090	9	Pass
0.0293	10241	1037	10	Pass
0.0302	9308	965	10	Pass
0.0311	8457	898	10	Pass
0.0320	7687	851	11	Pass
0.0329	6936	804	11	Pass
0.0338	6312	754	11	Pass
0.0347	5773	725	12	Pass
0.0356	5283	703	13	Pass
0.0366	4851	681	14	Pass
0.0375	4440	664	14	Pass
0.0384	4094	642	15	Pass
0.0393	3700	620	16	Pass
0.0402	3375	589	17	Pass
0.0411	3054	560	18	Pass

0.0420	2751	531	19	Pass
0.0429	2507	519	20	Pass
0.0438	2304	505	21	Pass
0.0448	2107	489	23	Pass
0.0457	1949	475	24	Pass
0.0466	1820	456	25	Pass
0.0475	1698	450	26	Pass
0.0484	1581	443	28	Pass
0.0493	1481	438	29	Pass
0.0502	1398	432	30	Pass
0.0511	1329	424	31	Pass
0.0521	1260	417	33	Pass
0.0530	1197	409	34	Pass
0.0539	1138	401	35	Pass
0.0548	1081	393	36	Pass
0.0557	1025	385	37	Pass
0.0566	952	379	39	Pass
0.0575	914	372	40	Pass
0.0584	879	363	41	Pass
0.0593	845	354	41	Pass
0.0603	808	349	43	Pass
0.0612	767	343	44	Pass
0.0621	731	338	46	Pass
0.0630	700	332	47	Pass
0.0639	676	328	48	Pass
0.0648	655	325	49	Pass
0.0657	639	317	49	Pass
0.0666	620	310	50	Pass
0.0675	604	303	50	Pass
0.0685	587	293	49	Pass
0.0694	573	281	49	Pass
0.0703	560	278	49	Pass
0.0712	551	274	49	Pass
0.0721	539	268	49	Pass
0.0730	523	263	50	Pass
0.0739	510	261	51	Pass
0.0748	496	255	51	Pass
0.0758	473	251	53	Pass
0.0767	457	246	53	Pass
0.0776	448	243	54	Pass
0.0785	438	238	54	Pass
0.0794	426	231	54	Pass
0.0803	417	226	54	Pass
0.0812	402	220	54	Pass
0.0821	396	216	54	Pass
0.0830	385	211	54	Pass
0.0840	374	207	55	Pass
0.0849	362	203	56	Pass
0.0858	355	201	56	Pass
0.0867	349	197	56	Pass
0.0876	338	193	57	Pass
0.0885	329	189	57	Pass
0.0894	320	185	57	Pass
0.0903	310	179	57	Pass
0.0912	306	173	56	Pass
0.0922	300	168	56	Pass
0.0931	296	159	53	Pass

0.0940	288	153	53	Pass
0.0949	283	149	52	Pass
0.0958	276	146	52	Pass
0.0967	270	140	51	Pass
0.0976	260	136	52	Pass
0.0985	252	132	52	Pass
0.0995	245	127	51	Pass
0.1004	239	121	50	Pass
0.1013	234	119	50	Pass
0.1022	227	116	51	Pass
0.1031	216	115	53	Pass
0.1040	205	114	55	Pass
0.1049	200	112	56	Pass
0.1058	195	112	57	Pass
0.1067	188	110	58	Pass
0.1077	184	109	59	Pass
0.1086	176	109	61	Pass
0.1095	170	106	62	Pass
0.1104	165	106	64	Pass
0.1113	158	102	64	Pass
0.1122	152	100	65	Pass

Water Quality BMP Flow and Volume for POC #1
On-line facility volume: 0.061 **acre-feet**
On-line facility target flow: 0.0497 **cfs.**
Adjusted for 15 min: 0.0497 **cfs.**
Off-line facility target flow: 0.0278 **cfs.**
Adjusted for 15 min: 0.0278 **cfs.**

LID Report

LID Technique	Used for Water Quality?	Total Volumn Needs Treatment (ac-ft)	Volumn Through Facility (ac-ft)	Infiltration Volumn (ac-ft.)	Cumulative Volumn Infiltration Credit	Percent Volumn Infiltrated	Water Quality Percent Treated	Comment
Trapezoidal Pond 1 POC	N	142.57				N	0.00	
Total Volume Infiltrated		142.57	0.00	0.00	0.00	0.00	0%	No Treat. Credit

Compliance with LID Standard 8
Duration Analysis Result = Failed

Perlnd and Implnd Changes

No changes have been made.

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2005 DOE STORMWATER MANUAL COMPOST AMENDED FILTER STRIP : WORKSHEET

Project name: STANWOOD SEPTAGE FACILITY — Date: 07/06/2016 — Legend

Project number: 2015-135 — Designer: JAR

	entry block
	calculated values
	design values
	constant

Compost Amended Filter Strip Sizing and Stability Check

	WWHM12 WQ Flows			
$Q_{100\text{-yr}}$:	15-minute peak	0.2195	cfs	If high flows flow through
$Q_{25\text{-yr}}$:	15-minute peak	0.1687	cfs	If high flows bypass
$Q_{2\text{-yr}}$:	15-minute peak	0.0858	cfs	
Q_{wq}:	**WQ 15-minute Flows (WWHM12)**	0.032	cfs	Water quality design flow
	***k*=**	1.30		per Fig. 9.6a or 9.6b, Volume V 2005 DOE

Find: depth at Design Flow of $Q_{biofilterstrip}$				**2005 DOE**	
SLOPE	0.015	*ft/ft*	*Mannings n*	0.45	Table 9.1 (compost amended and mowed)
y=	*0.0127719*	design depth (feet)			
W (T width) =	147	perimeter length perpendicular to flow (feet)			
k =	1.30	per *Fig. 9.6a or 9.6b, Volume V 2005 DOE*			
A=	1.8774626	sf	A=T*y	y =	0.1532623 inches
$Q_{treated}$=	0.0315382	cfs	*Calculated Using:*	$Q_{biofilterstrip}$ =(1.49Ty^1.67*S^1/2)/(K*n)	
V=	0.0218378	fps	Calculated Using:	V = kQ/Ty	
L =	11.79243	ft	Calculated Using:	L = 540*V (length needed for 9 min residence time)	

Filter Strip Check

Max Depth y < 1 inch for WQ	okay
Design WQ velocity < .5 fps using appropiate correction factor ***k*** for WWHM12	okay
Max Velocity < 3 fps with a conservatively mannings (n) and appropiate correction factor K for WWHM12	'okay
Total Residence Time during WQ flow > or = to 9 minutes	'okay
Appropiate mannings (n) for WQ	'okay
Geometric Dimensions within **Table 9.1** of 2005 DOE	'okay

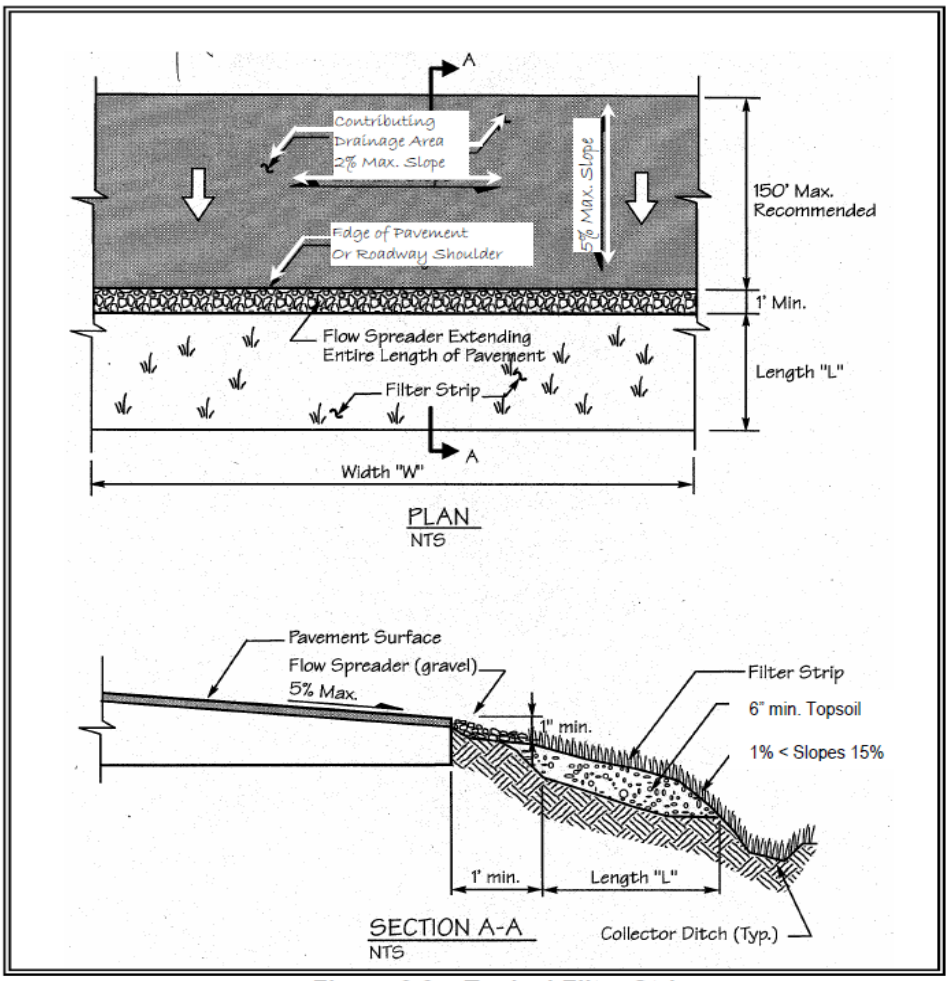
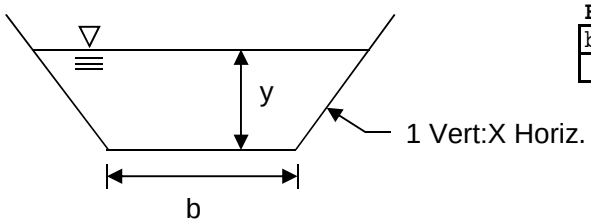


Figure 9.9 – Typical Filter Strip

MANNING'S EQUATION FOR OPEN CHANNEL FLOW
TRAPEZOIDAL SWALE CAPACITY

$$Q = \frac{1.49 A R^{\frac{2}{3}} S^{\frac{1}{2}}}{n}$$

	Design Input
	Calculated from Manning's
	Calculated from WWHM4 15-min. peaks



Build Dimensions

base (ft)	top (ft)	depth (ft)	freeboard(ft)
2.00	4.12	0.53	0.25

Input Values

b = base width	2.00 ft	V-shaped
y = depth of flow	0.28 ft	
Freeboard	0.25 ft	
x = horizontal sideslope	2.0 ft	Max.
S = longitudinal slope	0.00500 ft/ft	
n = roughness coefficient	0.045	

n= 0.045 (grass)

Calculated Variables

T = Top Width	4.12 ft	
A = Area	0.717 ft^2	
WP = Wetted Perimeter	3.252 ft	
R = Hydraulic Radius	0.220 ft	
V = Velocity	0.854 fps	@ design depth
Q = Quantity of flow	0.612 cfs	capacity @ build dimensions for min. slope

From **WWHM4** Q_{100} = 0.59 cfs

Capacity Check for 100-yr storm

100-YEAR STORM WATER SURFACE=	0.23	ft'@ min. slope (rock lined)
100-YEAR STORMWATER Max Velocity =	1.32	fps @ min. slope (rock lined)

Pipe Capacity for Kevos Pond Basin

Manning's Equation for Flow Capacity

$$Q = \frac{1.49\ AR^{\frac{2}{3}} S^{\frac{1}{2}}}{n}$$

$$V = \frac{1.49}{n} R^{2/3} S^{1/2}$$

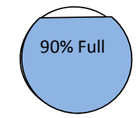
Where: Q = Discharge
V = Velocity
A = Wetted Area
n = Manning's roughness coefficient
R = Hydraulic Radius = WA/WP
S = Slope of energy grade line (pipe)

Mannings n Values for Pipe Types

Concrete	0.014
CMP	0.028
CPE	0.028
PVC	0.013

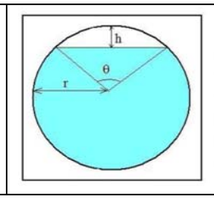
Values from Table 4.2 of Kitsap County SWMM

Flow Depth of Pipe



must have y > D/2

For a partially full pipe (> half full)



θ = 2 arccos (r - h / r)
WA = πr^2 - (r^2(θ - sinθ)/2)
WP = 2πr - rθ
h = 2r - d
D = 2r
d = D*(percent full)/100

Enter Data

0 cfs Basin Flow

Pipe ID#	CB Upstream ID#	CB Downstream ID#	Invert Elevation IN (ft)	Invert Elevation OUT (ft)	Pipe Length (ft)	Pipe Slope S (ft/ft)	Pipe Diameter D (ft)	Pipe Material	Manning's n (roughness coefficent) n	Height of Flow in Pipe d (ft)	Velocity V (fps)	Area A ft^2	Perimeter P	Wetted Area WA ft^2	Wetted Perimeter WP ft^2	Hydraulic Radius R ft^2	Percent Full (percent)	Quanity of Flow Q cfs	Basin Runoff Flowrate Q_{basin} cfs	Capacity in Pipe? No / Yes
Conveyance					75	0.0040	0.67	PVC	0.013	0.603	2.48	0.35	2.10	0.33	1.67	0.20	90	0.83	0	Yes
							1	CMP	0.028	0.9	0.00	0.79	3.14	0.74	2.50	0.30	90	0.00	0	No
							1	CPE	0.028	0.9	0.00	0.79	3.14	0.74	2.50	0.30	90	0.00	0	No
							1	PVC	0.013	0.9	0.00	0.79	3.14	0.74	2.50	0.30	90	0.00	0	No

APPENDIX C

(see Full Plan Set – provided as stand-alone product)