STORMWATER CALCULATIONS

Prepared For:

Salem Watumull LLC

9450 SW Gemini Drive #31339

Beaverton, OR 97008

Site Address:

Oxford St. and 20th St. Site Improvements

NW Intersection of Oxford St. SE and 20th St. SE

Salem, OR 97302

Permit Number: CO -

Prepared By:

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TABLE OF CONTENTS

List of Tables

Appendices

PROJECT OVERVIEW & DESCRIPTION SECTION 1

1.1 SIZE & LOCATION OF PROJECT

The proposed project is located at the NW corner of the intersection of Oxford and 20th Street SE. The site area is approximately 2.23 acres. Refer to the Civil Drawing for a site map of the project area.

1.2 BRIEF DESCRIPTION OF PROJECT SCOPE AND PROPOSED IMPROVEMENTS

The project scope is to expand the existing facility with a 63,170 square foot building, and landscaping. The project includes site preparation and construction of the facilities and associated improvements.

Public improvements along Oxford Street, Lewis Street, and 20th Street are proposed which include sidewalk, and landscaping. New and replaced impervious area within the Oxford Street, Lewis Street, and 20th Street right-of-way are less than 10,000 square feet.

1.3 DESCRIPTION OF SIZE OF WATERSHED DRAINING TO THE SITE

No additional drainage area drains to the project site.

1.4 DESCRIPTION OF THE EXISTING SITE CONDITIONS, CONSTRAINTS, SENSITIVE AREAS & WATERWAYS

The existing site is predominately covered in grasses, shrubs, several trees, and existing buildings that are proposed to be removed. The project site does not contain any existing sensitive areas, waterways, etc.

1.5 SUMMARY OF EXISTING TREES & NATIVE VEGETATION

The existing site as previously mentioned contains grasses, shrubs, and several trees within the site to be removed

1.6 SUMMARY OF GREEN STORMWATER INFRASTRUCTURE

Per Appendix 4E of the City of Salem Design Standards, a large project will be considered to have met the maximum extent feasible (MEF) requirement when the stormwater runoff from the total amount of new plus replaced impervious surfaces flows into an area set aside for GSI that is at least 10% of the total area of the new plus replaced impervious surfaces or up to 80% of all impervious area must be treated. The design implements GSI for the entire disturbed area and therefore meets MEF for GSI.

1.7 REGULATORY PERMITS REQUIRED

A 1200-C permit from DEQ will be obtained for the project. Additional City of Salem permits are also required. No other permits are required for this project.

1.8 100 YEAR STORM ESCAPE ROUTES

Please refer to the Basin Map for 100 year storm overflow routes. It also should be noted that the entire site minus the buildings is located within the 100 year floodplain. During a 100 year storm event much of the site will likely be underwater. However, if not flooded, floodwater will overflow per the routes notes on the Basin map in Appendix C. See the Civil Drawings and Basin Map for floodway boundaries.

METHODOLOGY

SECTION 2

2.1 DEPTH TO GROUNDWATER

Per the attached Geotechnical Report groundwater was encountered at 5 to 9.5 ft bgs (see Appendix A). Due to the shallow groundwater levels found at the site, the design engineer proposes to use an infiltration rain garden without drain rock. However, the bottom of the media will be within the 3 feet seasonal high groundwater separation. A design exception was previously approved with prior development projects on the subject property and is assumed to apply to the proposed project.

2.2 MAXIMUM INFILTRATION AND VEGETATIVE TREATMENT

GeoEngineers performed two infiltration tests on the site. Per the attached Geotechnical Report the measured infiltration rates were 4.5 and 3.0 in/hr. This results in the media being the limiting factor for controlling infiltration, therefore the design engineer used an infiltration rate of 2 in/hr to size the infiltration rain garden.

The proposed stormwater design will treat and detain the entire site utilizing infiltration rain gardens sized to infiltrate half the 2-year, 24-hour storm event, the water quality storm event, the 10-year, 24-hour storm event, the 25-year, 24-hour storm event and the 100-year, 24-hour storm event.

Since the stormwater for the entire site will be treated and detained via GSI facilities, the GSI has been implemented to the maximum extent feasible.

2.3 SOIL INFORMATION

The pre-developed site contains primarily soil group C/D soils and some D soils in the northeast corner of the project. Conservatively, C/D soils are assumed for pre-developed conditions and D soils are assumed for developed conditions. The pre-developed site was primarily grass covered with various clusters of trees and shrubs which correspond to a City of Salem pre-developed curve number of 72/79 for soil group C and group D per Appendix D of the City of Salem Design Standards. The average of these curve numbers was used to model the pre-developed conditions with C/D soils (i.e., 76).

2.4 HAZARDOUS MATERIAL

The owner is not aware of any hazardous material contamination onsite.

ANALYSIS

SECTION 3

3.1 METHODS & SOFTWARE USED

HydroCAD modeling software was used to size the stormwater facilities. The Santa Barbara Unit Hydrograph Type 1A storm was used to model the required design storms. Per the City of Salem Design Standards the design storms used were the 1.38 inch, 24 hour (water quality storm), half the 2 year, 24 hour and the 10 year, 24 hour storm events.

The site area is zoned IG (General Industrial), therefore a curve number corresponding to industrial use per Salem Design Standards Division 004 Appendix D was used to calculate the developed runoff flows. For soil group D this corresponds to a curve number of 93.

3.2 Conveyance Capacity Calculations

Since we propose to infiltrate up to the 100-year, 24-hour storm event downstream conveyance calculations have not been completed.

3.3 Treatment & Flow Control Sizing Calculations

The site was analyzed as one basin for the stormwater analysis. General basin characteristics of both pre-developed and developed conditions are listed in Table 1 below. For more detail refer to the Basin Map in Appendix C.

Table 1 | General Basin Characteristics - Onsite

Basin ID	Source (Roof/Road/Other)	Industrial Area (AC)	Pervious Area (AC)	Runoff Rates				Weighted CN
				½ 2 Year (cfs)	10 Year (cfs)	25 Year (cfs)	100 Year (cfs)	
PD[1]	Native	-	1.72	0.04	0.21	0.27	0.42	76
Developed	Industrial	1.72	-	0.21	1.10	1.28	1.63	93

[1] PD = pre-developed site conditions (i.e., pre-developed release rates)

The allowable onsite release rates based on pre-developed conditions for the design storms are listed below in Table 2.

Table 2 | Allowable Release Rates

Site Condition	Design Storm (cfs)			
	½ 2 Year	10 Year	25 Year	100 Year
Pre-Developed	0.04	0.21	0.27	0.42

An infiltration rain garden is proposed to fully infiltrate/detain the required storm events for onsite runoff. The proposed facility has been sized to infiltrate half the 2-year, 24-hour storm event, the water quality storm event, the 10-year, 24-hour storm event, the 25-year, 24-hour storm event, and the 100-year, 24-hour storm event with no release to the public storm drain up to the 100-year event. See Table 3 below for a summary of stormwater infiltration and release rates.

Table 3 | Summary of GSI Release Rates

Facility ID	Infiltration Rate (in/hr)	½ 2 Year (cfs)	Water Quality (cfs)	10 Year (cfs)	25 Year (cfs)	100 Year (cfs)
RG	2.0	0	0	0	0	0

A summary of the stormwater infiltration rain garden geometry is provided in Table 4 below.

Table 4 | Facility Sizing Summary

Facility ID[1]	Facility Elevations[2] (SF)		Facility Surface Area (SF)	
	Top	Bottom	Top	Bottom[2]
GSI-1	176	172	6,280	1,930

[1] All facilities are privately owned and maintained Stormwater Planters.
[2] Bottom corresponds to the top of the media.

In conclusion, the onsite stormwater system has been designed to fully infiltrate half the 2-year, 24-hour storm event, the water quality storm event, the 10-year, 24-hour storm event, the 25-year, 24-hour storm event, and the 100-year, 24-hour storm event. Therefore, the project is in conformance with the flow control and treatment requirements as set forth in Administrative Rule 109 Division 004 - Stormwater System.

APPENDIX A

GEOTECHNICAL MEMORANDUM

Geotechnical Engineering Report

Carpenter Commercial Properties—Commercial Building Project
Salem, Oregon

For

AC + Co Architecture | Community

August 6, 2019

GeoEngineers
Earth Science + Technology

Geotechnical Engineering Report

Carpenter Commercial Properties—Commercial Building Project
Salem, Oregon

for
AC + Co Architecture | Community

August 6, 2019

GeoEngineers

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Geotechnical Engineering Report

Carpenter Commercial Properties – Commercial Building Project
Salem, Oregon

File No. 23997-001-00

August 6, 2019

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Table of Contents

LIST OF FIGURES

APPENDICES

1.0 INTRODUCTION

GeoEngineers, Inc. (GeoEngineers) is pleased to submit this geotechnical engineering report for the proposed Carpenter Commercial Properties—Commercial Building Project in Salem, Oregon. The site is primarily located on the undeveloped field on the north side of Oxford Street SE approximately 100 feet west of 20th Street SE. The remainder of the site is located south of two existing commercial buildings and the undeveloped area north of the residence at 1545 SE 20th Street. The location and approximate extent of the project site is shown in the Vicinity Map, Figure 1.

We understand the project will consist of a one-story commercial building with associated parking and drive areas, and a new garage building on the north side of the site, west of the existing commercial buildings. At the time this report was prepared building loads had not been developed, but based on similar structures in the area we assume column loads will be less than 90 kips per column with linear wall loads on the order of 2 to 3 kips per lineal foot (klf), and floor loads less than 100 pounds per square foot (psf).

2.0 SCOPE OF SERVICES

The purpose of our services was to evaluate soil and groundwater conditions for use in design and construction of the proposed project. Our proposed scope of services included the following:

1. Reviewed information regarding subsurface soil and groundwater at the site, including reports in our files, selected geologic maps, and other geotechnical engineering related information.
2. Coordinated and managed the field investigation, including public utility notification, and scheduling of subcontractors and GeoEngineers' field staff.
3. Explored subsurface soil and groundwater conditions at the site by excavating seven test pit explorations to depths between 3½ and 10 feet below ground surface (bgs) at the approximate locations shown in the Site Plan, Figure 2.
4. Obtained samples at representative intervals from the explorations, observed groundwater conditions and maintained detailed logs in general accordance with ASTM International (ASTM) Standard Practices Test Method D 2488. Qualified staff from our office observed and documented field activities.
5. Performed laboratory tests on selected soil samples obtained from the explorations to evaluate pertinent engineering characteristics. Laboratory test results are included in Appendix A.
6. Performed two infiltration tests at depths between 2½ and 3½ feet bgs, using the open pit infiltration test method described in the City of Salem specifications.
7. Provided this geotechnical report that addresses the following geotechnical components:
    a. A general description of site topography, geology and subsurface conditions.
    b. An opinion as to the adequacy of site soil conditions for the proposed site development from a geotechnical engineering standpoint.
    c. Recommendations for site preparation measures, including disposing of undocumented fill and unsuitable native soils, and constraints for wet weather construction.
    d. Recommendations for earthwork construction, including use of on-site and imported structural fill, and fill placement and compaction requirements.

e. Recommendations for foundations to support the proposed structure, including minimum width and embedment, design soil bearing pressures, settlement estimates (total and differential) and coefficient of friction and passive earth pressures for sliding resistance.

f. Recommendations for supporting on-grade slabs, including base rock, capillary break and modulus of subgrade reaction, as appropriate.

g. Recommendations for below grade retaining walls, including static and seismic active earth pressures and drainage and backfill recommendations.

h. Recommendations for management of identified groundwater conditions that may affect the performance of structures or pavement.

i. Seismic design parameters in accordance with the current version of the International Building Code (IBC), and the Oregon Structural Specialty Code (OSSC), including our evaluation of the liquefaction and lateral spreading potential of the on-site soils.

Our geotechnical work has been directly supervised by a professional engineer licensed in the state of Oregon.

3.0 SITE CONDITIONS

3.1. Surface Conditions

The site is approximately 2 acres and is generally level with existing residential buildings, commercial buildings and associated paved and hardscaped areas. The undeveloped portions of the site are surfaced with field grasses and weeds with some fencing.

3.2. Site Geology

Published geologic maps of the area (Tolan and Beeson 2000) indicate that fills and disturbed soils at the site surface are underlain by fine-grained alluvial silt and Linn Gravel. The gravel unit is described as coarse fluvial gravels deposited as an alluvial fan. Our review of the site geology, together with on-site observations, suggests that the underlying site geology generally conforms to the Linn Gravel mapped in the area.

3.3. Subsurface Conditions

We completed field explorations for this study at the site on July 11, 2019. Our explorations included seven test pit excavations (TP-1 through TP-7), three dynamic cone penetration (DCP) tests and two infiltration tests. A summary of our exploration methods, the test pit logs, DCP test results and infiltration test results can be found in Appendix A. Laboratory test results are also provided in the exploration logs and described in Appendix A. The approximate locations of the explorations are shown in Figure 2.

The undeveloped portion of the site is generally surfaced with field grass with an approximate 3- to 4-inch-thick root zone. The surficial material west of the existing commercial buildings consists of angular gravel. Beneath the surficial materials, medium stiff to stiff brown silt with trace amounts of sand was encountered to depths between 1½ to 6 feet bgs, with exception of TP-7. Beneath the silt in explorations TP-1 through TP-6, and beneath the surficial materials in TP-7, dense gravel with various amounts of silt and occasional cobbles was encountered to the maximum depths explored.

3.4. Groundwater

Groundwater seepage was observed in our test pits TP-5 and TP-6 at depths between 5 and 9½ feet bgs. Based on nearby explorations, nearby well logs and our experience in the area, groundwater will likely be encountered at depths between 6 and 10 feet bgs depending on the time of measuring and site elevation relative to areal water levels. Shallow groundwater expected at the site is consistent with nearby ponds and standing water in excavations in former gravel mining sites. Dewatering of trenches may be required when perched or high groundwater is encountered. Groundwater conditions at the site are expected to vary seasonally due to rainfall events and other factors not observed in our explorations.

4.0 CONCLUSIONS

4.1. General

Based on our explorations, testing and analyses, it is our opinion that the site is suitable for the proposed project from a geotechnical standpoint, provided the recommendations in this report are included in design and construction. We offer the following conclusions regarding geotechnical design at the site.

- Structures can be satisfactorily supported on continuous and isolated shallow foundations supported on the firm near-surface silts or dense to very dense native silty gravels, or on structural fill that extends to the suitable native soils. If foundations are supported on the silt, a bearing capacity of 2,500 psf can be used to proportion foundations sizes. If supported directly on the dense native silty gravels, a bearing capacity of 4,000 psf can be used to proportion foundation sizes.
- Because of the silt content in the upper on-site soils, when soils at the site are exposed during excavation or grading, they will be easily disturbed by construction traffic or activities during periods of wet weather or when the moisture content of the soil is more than a few percentage points above optimum. Wet weather construction practices will be required when exposed soils are subject to construction traffic, except during the dry summer months.
- Based on proposed development, we estimate maximum anticipated loads of 90 kips or less for columns, 2 to 3 klf or less for walls, and slab on grade floor loads of 100 psf or less. Based on these assumed design loads, we estimate total settlement to be less than 1 inch. If larger structural loads are anticipated, we should review and reassess the estimated settlement.
- While groundwater was encountered in our explorations at depths between 5 and 9½ feet bgs, based on nearby water well logs, groundwater is likely between 6 and 10 feet bgs, and perched water may be encountered at higher elevations.
- Slabs-on-grade will be satisfactorily supported on firm native soils or structural fill overlying firm soils with a minimum 6-inch layer of compacted crushed rock base overlying approved subgrade or on structural fill over firm native soils.
- Existing utilities, if present across the site, that will be below proposed structural areas, including proposed buildings, should be relocated, abandoned or grouted full if left in place. Based on the location of the site and the previous use, unknown buried features such as tanks could be encountered.

4.2. Infiltration Testing

We conducted on-site infiltration testing to assist in evaluating the site for stormwater infiltration design. We conducted tests south of the proposed new building location as requested.

Testing was conducted using the open pit test procedure consistent with the method outlined in "Division 004" of the City of Salem Department of Public Works *Administrative Rules Design Standards* (COSDS). A 2- to 3-inch layer of clean, washed pea gravel was placed in the base of the test pit prior to adding water to diminish disturbance from flowing water. The test area was pre-soaked over a 4-hour period by repeated addition of water into the pit when necessary.

After the saturation period, the test pits were filled with clean water to at least 1 foot above the bottom of the pit. We observed the drop-in water level for three, 60-minute testing periods. Infiltration rates are based on the final testing period. Field test results are summarized in Table 1. The data and incremental infiltration rate over time are included in the infiltration test data summary in Appendix A, Figures A-12 and A-13.

TABLE 1. INFILTRATION RESULTS

Infiltration Test No.	Depth (feet)	USCS Material Type	Field Measured Infiltration Rate[1] (inches/hour)
IT-1	2½	ML	4.5
IT-2	3½	GP-GW	3.0

Notes:

[1] Field-measured infiltration rates should only be used in design if considerations noted in the section below, including the discussion of variability of fill and concern for limited infiltration as a result of high-water levels and very silty native soils are accounted for in facility design. In addition, appropriate factors should be applied to the field measured infiltration rate, based on the design methodology and specific system used.

USCS = Unified Soil Classification System

Although the infiltration rates south of the proposed building are relatively consistent in the two tests, groundwater was observed across the site at approximate depths of 5 to 9½ feet bgs. Based on the test results, the shallow groundwater, and considering the need to apply factors of safety to the field-measured tests as described below, we do not recommend on-site infiltration as the only means of stormwater disposal unless additional testing is performed and yields higher and more consistent infiltration rates in other areas of the site, or at different elevations.

The infiltration rates shown in Table 1 are field-measured infiltration rates. These represent a relatively short-term measured rate taken after the required saturation period, and factors of safety have not been applied for the type of infiltration system being considered, or for variability that may be present in the on-site soil. In our opinion, and consistent with the state of the practice, correction factors should be applied to this measured rate to reflect the variability in the fill materials that the test were conducted in.

Appropriate correction factors should also be applied by the project civil engineer to account for long-term infiltration parameters. From a geotechnical perspective, we recommend a factor of safety (correction factor) of at least 2 be applied to the infiltration values derived from field observations to account for potential soil variability with depth and location within the area tested. This will result in a recommended infiltration value of 1.5 to 2.2 inches per hour.

In addition, the stormwater system design engineer should determine and apply appropriate remaining correction factor values, or factors of safety, to account for repeated wetting and drying that occur in this area, degree of in-system filtration, frequency and type of system maintenance, vegetation, potential for siltation and bio-fouling, etc., as well as system design correction factors for overflow or redundancy, and base and facility size. Siltation of the upper facility medium is a common problem in new facilities where fine-grained soils are present in uphill sites and can wash into the new facility limiting (at times to zero) the infiltration capacity of designed facilities.

The actual depths, lateral extent and estimated infiltration rates can vary from the values presented above. Field testing/confirmation during construction is often required in large or long systems or other situations where soil conditions may vary within the area where the system is constructed.

Infiltration flow rate of a focused stormwater system typically diminishes over time as suspended solids and precipitates in the stormwater further clog the void spaces between the soil particles or cake on the infiltration surface. The serviceable life of an infiltration media in a stormwater system can be extended by pre-filtering or with on-going accessible maintenance. Eventually, most systems will fail and will need to be replaced or have media regenerated or replaced. We recommend that infiltration systems include an overflow that is connected to a suitable discharge point. Also, infiltration systems can cause localized high groundwater levels and should not be located near basement walls, retaining walls, or other embedded structures unless these are specifically designed to account for the resulting hydrostatic pressure. Infiltration locations should not be located on sloping ground, unless it is approved by a geotechnical engineer, and should not be infiltrated at a location that allows for flow to travel laterally toward a slope face, such as a mounded water condition or too close to a slope face.

4.2.1. Suitability of Infiltration System

Successful design and implementation of stormwater infiltration systems and whether a system is suitable for a development depend on several site-specific factors. Stormwater infiltration systems are generally best suited for sites having sandy or gravelly soil with saturated hydraulic conductivities greater than 2 inches per hour. Sites with silty or clayey soil such as encountered at this site, are generally not well-suited for long-term stormwater infiltration or as a sole method of stormwater infiltration. Soils that have fine-grained matrices are susceptible to volumetric change and softening during wetting and drying cycles. Fine-grained soils also have large variations in the magnitude of infiltration rates because of bedding and stratification that occurs during alluvial deposition, and often have thin layers of less permeable or impermeable soil within a larger layer.

Based on the fine-grained soil conditions and shallow groundwater observed, we recommend infiltration of stormwater not be used as the sole method of stormwater management at this site unless those design factors can be otherwise accounted for.

5.0 STRUCTURAL DESIGN RECOMMENDATIONS

5.1. Shallow Foundation Support Recommendations

Proposed structures can be satisfactorily founded on continuous wall or isolated column footings supported on firm silts or dense native gravels, or on structural fill placed over the firm native soils. We have carefully evaluated foundation support and subgrade preparation to provide adequate performance for the building, while still considering the project schedule, soil conditions and cost of earthwork. We have assumed that building loads will not exceed design load values provided to us as presented above.

Exterior footings should be established at least 18 inches below the lowest adjacent grade. The recommended minimum footing depth is greater than the anticipated frost depth (maximum of 12 inches based on local mapping). Interior footings can be founded a minimum of 12 inches below the top of the floor slab. Isolated column and continuous wall footings should have minimum widths of 24 and 18 inches, respectively.

5.1.1. Foundation Bearing Surface Preparation

Material beneath proposed structural elements should be prepared as described below and in Section 6.1. We recommend loose or disturbed soils resulting from foundation excavation be removed before placing reinforcing steel and concrete. Foundation bearing surfaces should not be exposed to standing water. If water pools in the excavation, the water, along with any disturbed soil, should be removed before placing reinforcing steel. A thin layer of crushed rock can be used to provide protection to the subgrade from weather and light foot traffic. Compaction should be performed as described in Section 6.6.6.

We recommend GeoEngineers observe all foundation excavations before placing concrete forms and reinforcing steel in order to determine that bearing surfaces have been adequately prepared and the soil conditions are consistent with those observed during our explorations.

5.1.2. Bearing Capacity – Spread Footings

We recommend conventional footings bearing on the stiff silt, or on compacted crushed rock backfill overlying the stiff silts be proportioned using a maximum allowable bearing pressure of 2,500 psf. If supported directly on the dense to very dense gravels, or on compacted crushed rock backfill overlying the dense to very dense gravels, the footings can be proportioned using a maximum allowable bearing pressure of 4,000 psf. These bearing pressures apply to the total of dead and long-term live loads and may be increased by one-third when considering earthquake or wind loads. This is a net bearing pressure. The weight of the footing and overlying backfill can be ignored in calculating footing sizes.

5.1.3. Foundation Settlement

Foundations designed and constructed as recommended are expected to experience settlements of less than 1 inch. Differential settlements of up to one-half of the total settlement magnitude can be expected between adjacent footings supporting comparable loads.

5.1.4. Lateral Resistance

The ability of the soil to resist lateral loads is a function of frictional resistance, which can develop on the base of footings and slabs and the passive resistance, which can develop on the face of below-grade elements of the structure as these elements tend to move into the soil. For footings and floor slabs founded in accordance with the recommendations presented above, the allowable frictional resistance may be computed using a coefficient of friction of 0.30 applied to vertical dead-load forces. Our analysis indicates that the available passive earth pressure for footings confined by on-site soil and structural fill is 200 pounds per cubic foot (pcf), modeled as an equivalent fluid pressure. Typically, the movement required to develop the available passive resistance may be relatively large; therefore, we recommend using a reduced passive pressure of 170 pcf equivalent fluid pressure. Adjacent floor slabs, pavements, or the upper 12-inch depth of adjacent unpaved areas should not be considered when calculating passive resistance. In addition, in order to rely on passive resistance, a minimum of 10 feet of horizontal clearance must exist between the face of the footings and any adjacent downslopes.

The passive earth pressure and friction components may be combined provided that the passive component does not exceed two-thirds of the total. The passive earth pressure value is based on the assumptions that the adjacent grade is level and that groundwater remains below the base of the footing throughout the year. The top foot of soil should be neglected when calculating passive lateral earth pressures unless the foundation area is covered with pavement or slab-on-grade. The lateral resistance values include a safety factor of approximately 1.5.

5.2. Floor Slabs

Satisfactory subgrade support for slab on grade floor slabs supporting up to 100 psf floor loads can be obtained provided the floor slab subgrade is prepared as recommended in Section 6.1 of this report, including compaction of the upper exposed subgrade. Slabs should be reinforced according to their proposed use and per the structural engineer's recommendations. Load-bearing concrete slabs should be designed assuming a modulus of subgrade reaction (k) of 125 pounds per cubic inch (pci).

We recommend that on-grade slabs be underlain by a minimum 6-inch-thick compacted crushed rock base to act as a capillary break and to provide adequate subgrade support for slab design. The capillary break material should consist of Aggregate Base material as described in Section 6.6 of this report. The material should be placed as recommended in Section 6.6.6. If dry slabs are required (e.g., where adhesives are used to anchor carpet or tile to the slab), a waterproof liner may be placed as a vapor barrier below the slab. The vapor barrier should be selected by the structural engineer and should be accounted for in the design floor section and mix design selection for the concrete, to accommodate the effect of the vapor barrier on concrete slab curing.

We estimate that concrete slabs constructed as recommended will settle less than 0.5 inch.

5.3. Drainage Considerations

We recommend the ground surface be sloped away from the buildings at least 2 percent. All downspouts should be tightlined away from the building foundation areas and should be discharged into a stormwater system. Downspouts should not be connected to footing drains.

Although not required based on groundwater depths observed in our explorations, if perimeter footing drains are used for below-grade structural elements to mitigate perched water that may flow on to the site from other sources, or behind walls, they should be installed at the base of the exterior footings. Perimeter footing drains should be provided with cleanouts and should consist of at least 4-inch-diameter perforated pipe placed on a 3-inch bed of, and surrounded by, 6 inches of granular drainage material. Aggregate Base can be used for the granular pipe bedding and drainage materials provided the material has less than 3 percent passing the U.S. No. 200 sieve. The drainage material should be enclosed in a non-woven geotextile such as Mirafi 140N (or approved alternate) to prevent fine soil from migrating into the drain material. We recommend against using flexible tubing for footing drainpipes. The perimeter drains should be sloped to drain by gravity to a suitable discharge, preferably a storm drain. We recommend that the cleanouts be covered and placed in flush-mounted utility boxes. Water collected in roof downspout lines must not be routed to the footing drain lines.

5.4. Retaining Wall Recommendations

5.4.1. General

The following general recommendations can be implemented for wall design where new walls for site access are required or evaluation of existing walls or shoring walls that are not internally braced. Our retaining wall design recommendations are based on the following assumptions: (1) the walls consist of conventional, cantilevered retaining walls; (2) walls are less than 8 feet in height; (3) the backfill is drained; and (4) the backfill has a slope flatter than 2H:1V (horizontal to vertical). Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project varies from these assumptions.

5.4.2. Drainage

Positive drainage is imperative behind retaining structures. This can be accomplished by providing a drainage zone behind the wall consisting of free-draining material and perforated pipes to collect and dispose the water. The drainage material should consist of Aggregate Base having less than 3 percent passing the U.S. No 200 sieve. The wall drainage zone should extend horizontally at least 18 inches from the back of the wall.

A perforated smooth-walled rigid drainpipe having a minimum diameter of 4 inches should be placed at the bottom of the drainage zone along the entire length of the wall, with the pipe invert at or below the base of the wall footing. The drainpipes should discharge to a tightline leading to an appropriate collection and disposal system. An adequate number of cleanouts should be incorporated into the design of the drains to provide access for regular maintenance. Roof downspouts, perimeter drains or other types of drainage systems should not be connected to retaining wall drain systems.

5.4.3. Design Parameters

The lateral pressures presented in this section for retaining walls assume that backfill placed within 2 feet of the wall is compacted by hand-operated equipment to a density of 90 percent of the maximum dry density (MDD) and that wall drainage measures are included as previously recommended. For walls constructed as described above, as with a maximum height of 8 feet, we recommend using an active lateral earth pressure corresponding to an equivalent fluid density of 35 pcf for the level backfill condition. For walls with backfill sloping upward behind the wall at 2H:1V, an equivalent fluid density of 60 pcf should be used. If the slope is shallower than 2H:1V, the active lateral earth pressures can be linearly interpolated between the two values above. This assumes that the tops of the walls are not structurally restrained and are free to rotate.

For the at-rest condition (walls restrained from movement at the top) an equivalent fluid density of 55 pcf for level conditions and 85 pcf for a 2H:1V slope behind the wall, should be used for design.

For seismic conditions, we recommend a uniform lateral pressure of 4H (where H is the height of the wall) psf be added to these lateral pressures. If the retaining system is designed as a braced system but is expected to yield a small amount during a seismic event, an active earth pressure condition may be assumed and combined with the uniform seismic surcharge pressure.

The recommended pressures do not include the effects of surcharges from surface loads. If vehicles will be operated within one-half the height of the wall, a traffic surcharge should be added to the wall pressure.

The traffic surcharge can be approximated by the equivalent weight of an additional 2 feet of backfill behind the wall. Additional surcharge loading conditions should also be considered on a case-by-case basis.

If shallow foundations are located behind the retaining wall within a 1H:1V projection from the base of the wall, foundation loads will impart additional pressures on the retaining wall. If the design of the building requires foundations within the 1H:1V projection, the loads imparted on the wall should be included in the design on the wall. Foundation induced lateral loads imparted on the retaining wall will depend on the size of the load and the distance setback from the back of the wall.

Retaining walls founded on native soil or structural fill extending to these materials may be designed using the allowable soil bearing values and lateral resistance values presented above in Section 5.1 of this report. We estimate settlement of retaining structures will be similar to the values previously presented for building foundations.

5.5. Pavement Recommendations

5.5.1. Dynamic Cone Penetrometer (DCP) Field Testing and Resilient Modulus (M_R)

We conducted DCP tests in general accordance with ASTM D 6951 to estimate M_R at each test location. We recorded penetration depths of the cone versus hammer blow counts and terminated testing at depths between 2 and 3½ feet bgs. We conducted DCP tests beneath the surficial material in explorations TP-2, TP-6 and TP-7. We estimate the resilient modulus of the subgrade materials in general accordance with the Oregon Department of Transportation (ODOT) Pavement Design Guide using a conversion coefficient, C_f, of 0.35. Table 2 lists the estimated subgrade resilient modulus at each test location. Field DCP data are summarized in Figures A-9 through A-11.

TABLE 2. ESTIMATED SUBGRADE RESILIENT MODULI BASED ON DCP TESTING

Boring Number	Estimated Resilient Modulus (psi)
DCP-1	4,800
DCP-2	5,200
DCP-3	4,000

Note:
psi = pounds per square inch

5.5.2. General

Our pavement recommendations are based on the results of our field testing and analysis. The recommended pavement sections assume that final improvements surrounding the pavement will be designed and constructed such that stormwater or excess irrigation water from landscape areas does not infiltrate below the pavement section into the base rock materials.

Pavement subgrades should be prepared in accordance with Section 6.0 of this report. Our pavement design assumes that traffic at the site will consist of occasional truck traffic and passenger cars. We do not have specific information on the frequency and type of vehicles that will use the area; however, we have based our design analysis on traffic consisting of five heavy trucks per day to account for delivery- and service-type vehicles and passenger car traffic for the heavy-duty pavement sections, and passenger car traffic only for the light-duty pavement sections.

Our pavement recommendations are based on the following assumptions and design parameters included in the Asphalt Pavement Association of Oregon (APAO) Design Manual (Hicks, et al. 2003):

- The pavement subgrades, fill subgrades and site earthwork used to establish pavement grades below the Aggregate Subbase and Aggregate Base materials have been prepared as described in Section 6.0 of this report.
- A resilient modulus of 20,000 psi has been estimated for compacted Aggregate Subbase and Aggregate Base materials.
- A resilient modulus of 4,600 psi was estimated for firm native silts based of DCP results.
- Initial and terminal serviceability indices of 4.2 and 2.0, respectively.
- Reliability and standard deviations of 90 percent and 0.49, respectively.
- Structural coefficients of 0.42 and 0.10 for the asphalt and base rock, respectively.
- A 20-year design life without any growth.
- Light-duty areas are for paved areas that only passenger car traffic will load, and heavy-duty areas are for paved areas that will carry passenger traffic and truck traffic.
- Truck traffic consists of five trucks per day with an even distribution of two-axle service trucks/vans and large, four-axle trucks.

If any of the noted assumptions vary from project design use, our office should be contacted with the appropriate information so that the pavement designs can be revised or confirmed adequate. The recommended minimum pavement sections are provided in Table 3.

The alternate pavement section using Aggregate Subbase material is provided because it may be more applicable during wet-weather construction where a gravel haul road or working surface is needed to support construction traffic. Wet weather construction recommendations are provided in Section 6.0 of this report. The subbase material can be incorporated into the gravel working blankets and haul roads provided the material meets the minimum thickness in Table 3 and meets the specifications for Aggregate Subbase. Working blanket and haul road materials that pump excessively or have excessive fines from construction traffic should be removed and replaced with proper materials prior to constructing roadways over those areas.

TABLE 3. MINIMUM PAVEMENT SECTIONS FOR ON-SITE ROADWAYS AND PARKING AREAS

Location	Minimum Asphalt Thickness (inches)	Minimum Aggregate Base Thickness (inches)	Minimum Aggregate Subbase Thickness (inches)
Light Duty	2½	6.0	NA
Light Duty	2½	3.0	6.0
Heavy Duty	3½	8.0	NA
Heavy Duty	3½	4.0	6.0

The aggregate base course should conform to Section 6.6.3 of this report and be compacted to at least 95 percent of the MDD determined in accordance with American Association of State Highway and Transportation Officials (AASHTO) T-180/ASTM Test Method D 1557.

The asphalt concrete (AC) pavement should conform to Section 00745 of the most current edition of the ODOT Standard Specifications for Highway Construction. The Job Mix Formula should meet the requirements for a ½-inch Dense Graded Level 2 Mix. The AC should be PG 64-22 grade meeting the ODOT Standard Specifications for Asphalt Materials. AC pavement should be compacted to 92.0 percent at Maximum Theoretical Unit Weight (Rice Gravity) of AASHTO T-209.

6.0 EARTHWORK RECOMMENDATIONS

6.1. Site Preparation

In general, site preparation will include removing or relocating existing site utilities if present, demolishing hardscaped areas, stripping and site grading. It is possible that site excavations and grading will encounter buried features from previous site uses not observed in our explorations. Site preparation will also include grading the site and excavating for utilities and foundations.

6.1.1. Demolition

All structures and hardscaped areas to be demolished should be completely removed from proposed structural areas. Proposed structural areas are areas where new structures will be built, including building pads and parking areas. Existing utilities that will be abandoned on site should be identified prior to construction. Abandoned utility lines should be completely removed or filled with grout if abandoned and left in-place to reduce potential settlement or caving in the future. Materials generated during demolition should be transported off site and properly disposed.

6.1.2. Stripping

Based on our observations at the site, we estimate that the depth of stripping will generally be on the order of about 4 inches. Greater stripping depths may be required to remove localized zones of loose or organic soil. The actual stripping depth should be based on field observations at the time of construction. Stripped material should be transported off site for disposal unless otherwise allowed by project specifications for other uses such as landscaping. Clearing and grubbing recommendations provided below should be used in areas where moderate to heavy vegetation are present, or where surface disturbance from prior use has occurred.

6.2. Subgrade Preparation and Evaluation

Upon completion of site preparation activities, exposed subgrades for at-grade construction should be proof-rolled with a fully loaded dump truck or similar heavy rubber-tired construction equipment to identify soft, loose, or unsuitable areas. Probing may be used for evaluating smaller areas or where proof-rolling is not practical. Proof-rolling and probing should be conducted prior to placing fill and should be performed by a representative of GeoEngineers who will evaluate the suitability of the subgrade and identify areas of yielding that are indicative of soft or loose soil. If soft or loose zones are identified during proof-rolling or probing, these areas should be excavated to the extent indicated by our representative and replaced with structural fill.

As discussed in Section 6.6 of this report, the native soils can be sensitive to small changes in moisture content and will be difficult to compact adequately during wet weather. While tilling and compacting the subgrade is the economical method for subgrade improvement, it will likely only be possible during extended dry periods and following moisture conditioning of the soil.

During wet weather, or when the exposed subgrade is wet or unsuitable for proof-rolling, the prepared subgrade should be evaluated by observing excavation activity and probing with a steel foundation probe. Observations, probing and compaction testing should be performed by a member of our staff. Wet soil that has been disturbed due to site preparation activities, or soft or loose zones identified during probing, should be removed and replaced with compacted structural fill.

6.2.1. Subgrade Protection and Wet Weather Considerations

Portions of the near-surface soils at the site are highly susceptible to moisture. Wet weather construction practices will be necessary if work is performed during periods of wet weather. If site grading will occur during wet weather conditions, it will be necessary to use track-mounted equipment, load removed material into trucks supported on gravel haul roads, use gravel working pads and employ other methods to reduce ground disturbance. The contractor should be responsible to protect the subgrade during construction.

Earthwork planning should include considerations for minimizing subgrade disturbance. We provide the following recommendations if wet weather construction is considered:

- The ground surface in and around the work area should be sloped so that surface water is directed to a sump or discharge location. The ground surface should be graded such that areas of ponded water do not develop. Measures should be taken by the contractor to prevent surface water from collecting in excavations and trenches. Measures should be implemented to remove surface water from the work areas.
- Earthwork activities should not take place during periods of heavy precipitation.
- Slopes with exposed soils should be covered with plastic sheeting or similar means.
- The site soils should not be left in a disturbed or uncompacted state and exposed to moisture. Sealing the surficial soils by rolling with a smooth-drum roller prior to periods of precipitation may reduce the extent to which these soils become wet or unstable.
- Construction activities should be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practicable.
- Construction traffic should be restricted to specific areas of the site, preferably areas that are not susceptible to wet weather disturbance such as haul roads and areas that are adequately surfaced with working pad materials.
- When on-site soils are wet of optimum, they are easily disturbed and will not provide adequate support for construction traffic for the proposed development. The use of granular haul roads and staging areas will be necessary to support heavy construction traffic. Generally, a 12- to 16-inch-thick mat of Imported Select Structural Fill should be sufficient for light staging areas for the building pad and light staging activities but is not expected to be adequate to support repeated heavy equipment or truck traffic. The thickness of the Imported Select Structural Fill for haul roads and areas with repeated heavy construction traffic should be increased to between 18 and 24 inches. The actual thickness of haul

roads and staging areas should be determined at the time of construction and based on the contractor's approach to site development and the amount and type of construction traffic.

- The base rock (Aggregate Base and Aggregate Subbase) thicknesses described in Section 5.5 of this report is intended to support post-construction design traffic loads. The design base rock thicknesses will likely not support repeated heavy construction traffic during site construction, or during pavement construction. A thicker base rock section, as described above for haul roads, will likely be required to support construction traffic.
- During periods of wet weather, concrete should be placed as soon as practical after preparing foundation excavations. Foundation bearing surfaces should not be exposed to standing water. Should water infiltrate and pool in the excavation, the water should be removed, and the foundation subgrade should be re-evaluated before placing reinforcing steel or concrete. Foundation subgrade protection, such as a 3- to 4-inch thickness of Aggregate Base/Aggregate Subbase or lean concrete, may be necessary if footing excavations are exposed to extended wet weather conditions.

During wet weather, or when the exposed subgrade is wet or unsuitable for proof-rolling, the prepared subgrade should be evaluated by observing excavation activity and probing with a steel foundation probe. Observations and probing should be performed by a member of our staff. Wet soil that has been disturbed due to site preparation activities, or soft or loose zones identified during probing, should be removed and replaced with Imported Select Structural Fill.

6.3. Excavation

Based on the materials encountered in our subsurface exploration, it is our opinion that conventional earthmoving equipment in proper working condition should be capable of making necessary general excavations.

The earthwork contractor should be responsible for reviewing this report, including the boring logs, providing their own assessments, and providing equipment and methods needed to excavate the site soils while protecting subgrades.

6.4. Dewatering

As discussed in Section 3.4 of this report, groundwater was encountered in our explorations at depths between 5 and 9½ feet bgs. If excavations extend below these depths, groundwater may be a factor and may require dewatering. Excavations that extend into saturated/wet soils, or excavations that extend into perched groundwater, may require significant effort to dewater. Sump pumps are expected to adequately address perched water encountered in shallow excavations and a more intensive use of sumps will likely suffice in deeper explorations. In addition to groundwater seepage, surface water inflow to the excavations during the wet season can be problematic. Provisions for surface water control during earthwork and excavations should be included in the project plans and should be installed prior to commencing earthwork.

6.5. Shoring

All trench excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations. In our opinion, native soils are generally OSHA Type C. Excavations deeper than 4 feet should be shored or laid back at an inclination of 1.5H:1V or flatter if workers are required to enter. Excavations made to construct footings or other structural elements should be laid back or shored at the surface as necessary to prevent soil from falling into excavations.

Shoring for trenches less than 6 feet deep that are above the effects of groundwater should be possible with a conventional box system. Moderate sloughing should be expected outside the box. Shoring deeper than 6 feet or below the groundwater table should be designed by a registered engineer before installation. Further, the shoring design engineer should be provided with a copy of this report.

The site earthwork contractor should expect that unsupported cut slopes will likely experience some sloughing and raveling if exposed to water. Plastic sheeting, placed over the exposed slope and directing water away from the slope, will reduce the potential for sloughing and erosion of cut slopes during wet weather.

In our opinion, the contractor will be in the best position to observe subsurface conditions continuously throughout the construction process and to respond to the soil and groundwater conditions. Construction site safety is generally the sole responsibility of the contractor, who also is solely responsible for the means, methods and sequencing of the construction operations and choices regarding excavations and shoring. Under no circumstances should the information provided by GeoEngineers be interpreted to mean that GeoEngineers is assuming responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred.

6.6. Structural Fill and Backfill

Structural areas include areas beneath foundations, floor slabs and any other areas intended to support structures or within the influence zone of structures.

All structural fill soils should be free of debris, clay balls, roots, organic matter, frozen soil, man-made contaminants, particles with greatest dimension exceeding 6 inches (3-inch-maximum particle size in building footprints) and other deleterious materials. The suitability of soil for use as structural fill will depend on the gradation and moisture content of the soil. As the amount of fines in the soil matrix increases, the soil becomes increasingly more sensitive to small changes in moisture content and achieving the required degree of compaction becomes more difficult or impossible. Recommendations for suitable fill material are provided in the following sections.

6.6.1. On-Site Soils

The on-site silt with trace sand and gravels with silt is generally suitable for use as structural fill, provided it meets the requirements set forth in OSSC 00330.12 (Borrow Material). When the on-site material is used as structural fill, it should be placed in lifts with a maximum uncompacted thickness of 12 inches and should be compacted to not less than 95 percent of the MDD, as determined by ASTM D 1557. The near surface site soil contains a moderate amount of fine-grained material and is sensitive to changes in moisture content and is susceptible to disturbance when wet. Use of the on-site material as structural fill may not be possible during wet weather (see Section 6.2.1 of this report) or when the moisture content of the soils is more than three points above optimum. When wet of optimum, on-site soils will need to be dried back in order to achieve adequate compaction.

6.6.2. Imported Select Structural Fill

Imported Select Structural Fill may be used as structural fill and should consist of pit or quarry run rock, crushed rock, or crushed gravel and sand that is fairly well-graded between coarse and fine sizes (approximately 25 to 65 percent passing the U.S. No. 4 sieve). It should have less than 5 percent passing the U.S. No. 200 sieve and have a minimum of 75 percent fractured particles according to AASHTO TP-61.

6.6.3. Aggregate Base

Aggregate base material located under floor slabs and pavements, crushed rock used in footing over excavations and used as wall backfill should consist of imported clean, durable, crushed angular rock. Such rock should be well-graded, have a maximum particle size of 1 inch, have less than 5 percent passing the U.S. No. 200 sieve (3 percent for retaining walls) and meet the gradation requirements in Table 4. In addition, Aggregate Base shall have a minimum of 75 percent fractured particles according to AASHTO TP-61 and a sand equivalent of not less than 30 percent based on AASHTO T-176.

TABLE 4. RECOMMENDED GRADATION FOR AGGREGATE BASE

Sieve Size	Percent Passing (by weight)
1 inch	100
¾ inch	80 to 95
½ inch	50 to 80
No. 4	40 to 60
No. 40	5 to 15
No. 200	0 to 5

6.6.4. Trench Backfill

Trench backfill for pipe bedding and in the pipe zone should consist of well-graded granular material with a maximum particle size of ¾ inch and less than 5 percent passing the U.S. No. 200 sieve. The material should be free of organic matter and other deleterious materials. Further, the backfill should meet the pipe manufacturer's recommendations. Above the pipe zone backfill, Imported Select Structural Fill may be used as described above.

6.6.5. Aggregate Subbase

Aggregate Subbase material should consist of imported, clean, durable, crushed angular rock. Such rock should be well-graded, have a maximum particle size of 1½ inches, have less than 5 percent passing the U.S. No. 200 sieve and meet the gradation requirements in the ODOT Standard Section 00331. In addition, aggregate base shall have a minimum of 75 percent fractured particles according to AASHTO TP-61 and a sand equivalent of not less than 30 percent based on AASHTO T-176.

6.6.6. Fill Placement and Compaction

Structural fill should be compacted at moisture contents that are within 3 percent of the optimum moisture content as determined by ASTM D 1557 (Modified Proctor). The optimum moisture content varies with gradation and should be evaluated during construction. Fill material that is not near the optimum moisture content should be moisture conditioned prior to compaction.

Fill and backfill material should be placed in uniform, horizontal lifts and compacted with appropriate equipment. The appropriate lift thickness will vary depending on the material and compaction equipment used. Fill material should be compacted in accordance with Table 5, below. It is the contractor's responsibility to select appropriate compaction equipment and place the material in lifts that are thin enough to meet these criteria. However, in no case should the loose lift thickness exceed 18 inches.

TABLE 5. COMPACTION CRITERIA

Fill Type	Compaction Requirements			
	Percent Maximum Dry Density Determined by ASTM Test Method D 1557 at ± 3% of Optimum Moisture			
	0 to 2 Feet Below Subgrade	> 2 Feet Below Subgrade	0 to 2 Feet Below Subgrade	
Fine-grained soils (non-expansive)	92	Fine-grained soils (non-expansive)	92	
Imported Granular, maximum particle size < 1¼ inch	95	Imported Granular, maximum particle size < 1¼ inch	95	
Imported Granular, maximum particle size 1¼ inch to 6 inches (3-inch maximum under building footprints)	n/a (proof-roll)	Imported Granular, maximum particle size 1¼ inch to 6 inches (3-inch maximum under building footprints)	n/a (proof-roll)	
Retaining Wall Backfill*	92	Retaining Wall Backfill*	92	
Nonstructural Zones	90	Nonstructural Zones	90	

Note:

*Measures should be taken to prevent overcompaction of the backfill behind retaining walls. We recommend placing the zone of backfill located within 5 feet of the wall in lifts not exceeding about 6 inches in loose thickness and compacting this zone with hand-operated equipment such as a vibrating plate compactor and a jumping jack.

A representative from GeoEngineers should evaluate compaction of each lift of fill. Compaction should be evaluated by compaction testing, unless other methods are proposed for oversized materials and are approved by GeoEngineers during construction. These other methods typically involve procedural placement and compaction specifications together with verifying requirements such as proof-rolling.

6.7. Seismic Design

We recommend seismic design be performed using the procedure outlined in the 2015 IBC and the 2014 OSSC. The parameters provided in Table 6 are based on the conditions encountered during our subsurface exploration program, during previous exploration programs, and the mapped local geology, and should be used in preparation of response spectra for the proposed structures.

TABLE 6. SEISMIC DESIGN PARAMETERS

Parameter	Value
Site Class	C
Site Modified Peak Ground Acceleration, PGA_M	0.43 g
Spectral Response Acceleration, S_s	0.98 g
Spectral Response Acceleration, S_1	0.42 g
Site Coefficient, F_a	1.01
Site Coefficient, F_v	1.38
Spectral Response Acceleration (Short Period), S_{DS}	0.66 g
Spectral Response Acceleration (1-Second Period) S_{D1}	0.39 g

6.7.1. Liquefaction Potential

Liquefaction is a phenomenon caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. The excessive buildup of pore water pressure results in the sudden loss of shear strength in a soil. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. Sand boils and flows observed at the ground surface after an earthquake are the result of excess pore pressures dissipating upwards, carrying soil particles with the draining water. In general, loose, saturated sand soil with low silt and clay contents is the most susceptible to liquefaction. Low plasticity, silty sand may be moderately susceptible to liquefaction under relatively higher levels of ground shaking.

Based on our analysis, the site soils are not prone to liquefaction during the design level earthquake. Accordingly, lateral spreading or liquefaction induced deformations are not expected.

7.0 DESIGN REVIEW AND CONSTRUCTION SERVICES

Recommendations provided in this report are based on the assumptions and design information stated herein. We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed. In addition, GeoEngineers should be retained to review the geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in this report.

Satisfactory foundation and earthwork performance depend to a large degree on quality of construction. Sufficient monitoring of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions often requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

We recommend that GeoEngineers be retained to observe construction at the site to confirm that subsurface conditions are consistent with the site explorations and to confirm that the intent of project plans and specifications relating to earthwork, pavement and foundation construction are being met.

8.0 LIMITATIONS

We have prepared this report for the exclusive use of AC + Co Architecture | Community, the owner and their authorized agents and/or regulatory agencies for the proposed Carpenter Commercial Properties – Commercial Building Project in Salem, Oregon.

This report is not intended for use by others and the information contained herein is not applicable to other sites. No other party may rely on the product of our services unless we agree in advance and in writing to such reliance.

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

9.0 REFERENCES

City of Salem. 2014. City of Salem Department of Public Works Administrative Rules Design Standards. Salem, Oregon.

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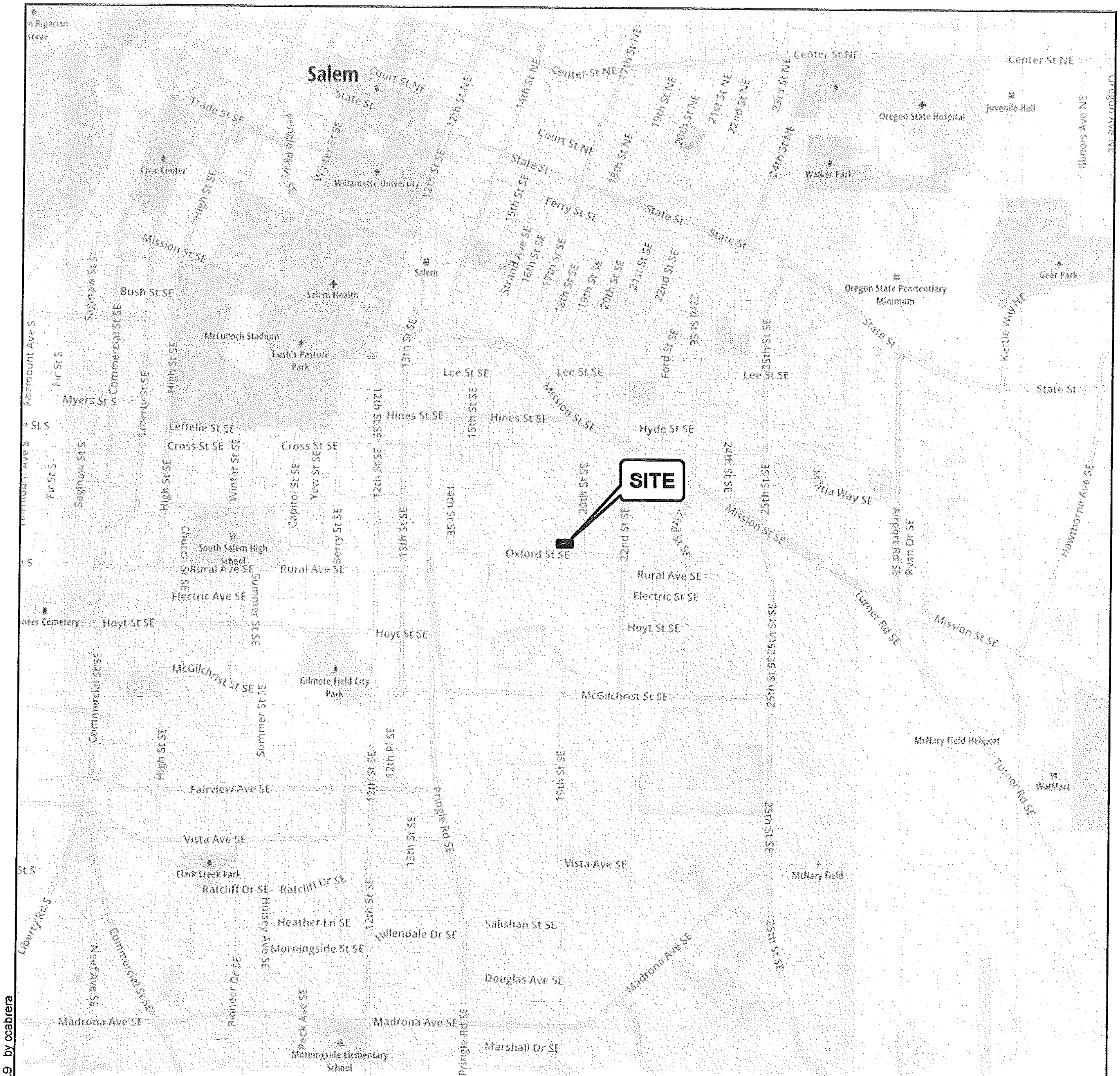
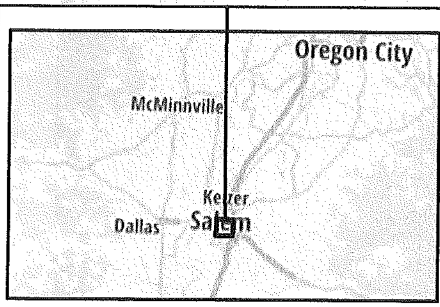
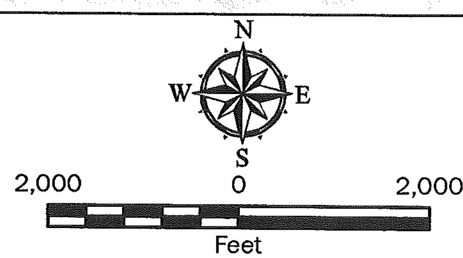
Tolan, T. L. and M. H. Beeson. 2000. Geologic Map of the Salem East 7.5 Minute Quadrangle, Marion County, Oregon: U.S. Geological Survey, Open-File Report 00-351, scale 1:24,000.

FIGURES







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, 2016

Projection: NAD 1983 UTM Zone 10N

Vicinity Map

Carpenter Commercial Properties – Commercial Building
Salem, Oregon

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Figure 1

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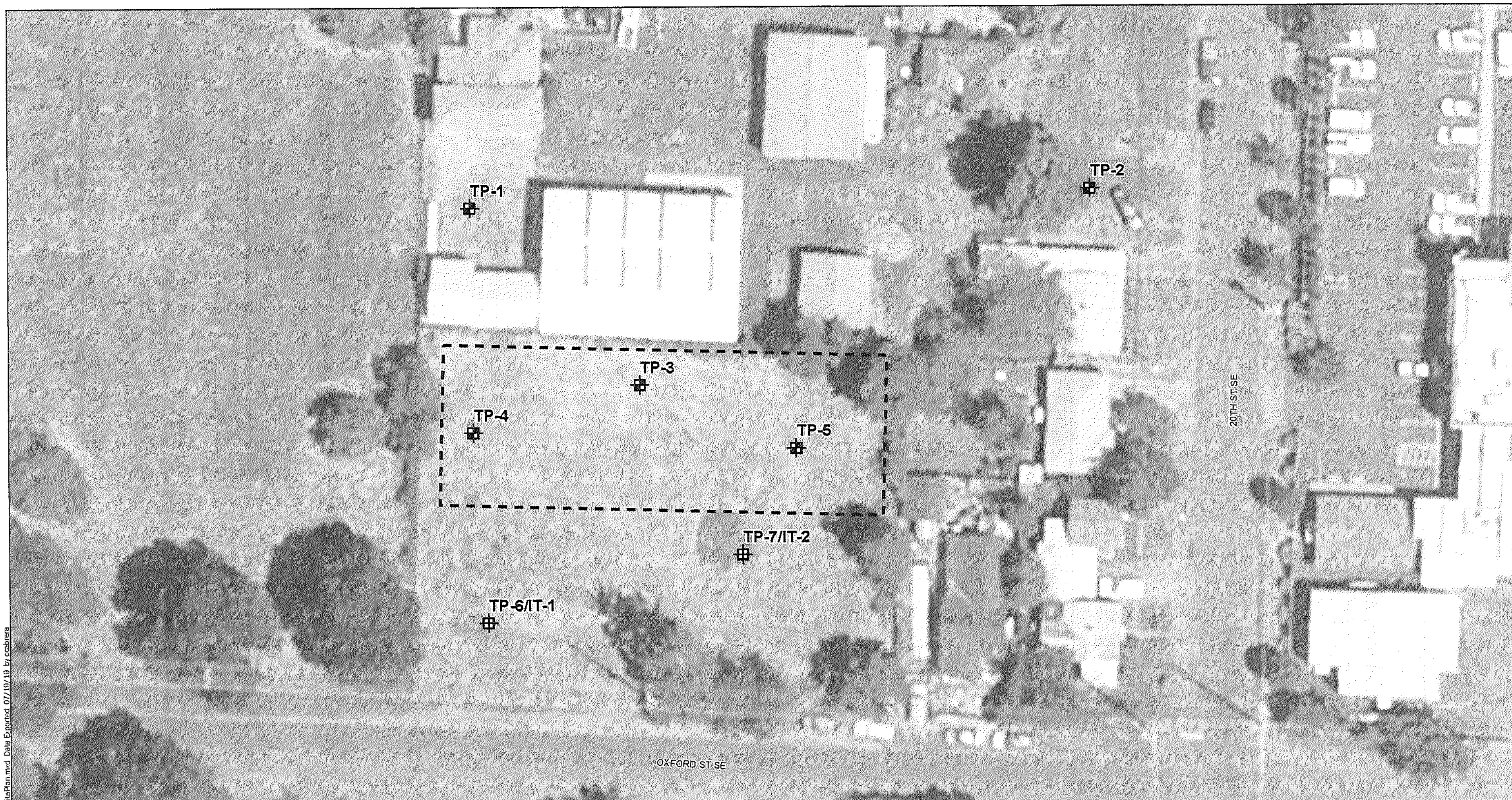

Notes:
1. The locations of all features shown are approximate.
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Data Source: Clarity ESRI.

Projection: NAD 1983 StatePlane Oregon North FIPS 3601 Feet Intl

Legend

- Test Pit Number and Approximate Location
- Test Pit & Infiltration Test Number and Approximate Location
- Proposed Building Location

Site Plan

Carpenter Commercial Properties – Commercial Building
Salem, Oregon

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Figure 2

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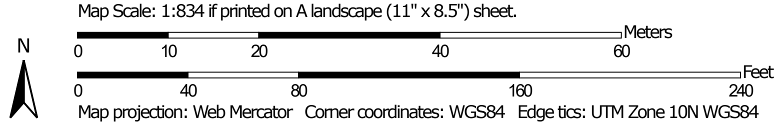
APPENDIX B

NRCS SOIL REPORT

Soil Map—Marion County Area, Oregon

Map Scale: 1:834 if printed on A landscape (11" x 8.5") sheet.



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

MAP LEGEND

Area of Interest (AOI)

Area of Interest (AOI)

Soils

Soil Map Unit Polygons

Soil Map Unit Lines

Soil Map Unit Points

Special Point Features

Blowout

Borrow Pit

Clay Spot

Closed Depression

Gravel Pit

Gravelly Spot

Landfill

Lava Flow

Marsh or swamp

Mine or Quarry

Miscellaneous Water

Perennial Water

Rock Outcrop

Saline Spot

Sandy Spot

Severely Eroded Spot

Sinkhole

Slide or Slip

Sodic Spot

Spoil Area

Stony Spot

Very Stony Spot

Wet Spot

Other

Special Line Features

Water Features

Streams and Canals

Transportation

Rails

Interstate Highways

US Routes

Major Roads

Local Roads

Background

Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:20,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Marion County Area, Oregon
Survey Area Data: Version 22, Aug 30, 2024

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

Date(s) aerial images were photographed: May 17, 2023—Jun 3, 2023

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
Ck	Clackamas gravelly loam	2.4	100.0%
Totals for Area of Interest		**2.4**	**100.0%**

Map Scale: 1:834 if printed on A landscape (11" x 8.5") sheet.

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

MAP LEGEND

Area of Interest (AOI)

- Area of Interest (AOI)

Soils

Soil Rating Polygons

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

Soil Rating Lines

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

Soil Rating Points

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

Water Features

- Streams and Canals

Transportation

- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background

- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:20,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Marion County Area, Oregon
Survey Area Data: Version 22, Aug 30, 2024

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

Date(s) aerial images were photographed: May 17, 2023—Jun 3, 2023

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Hydrologic Soil Group

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
Ck	Clackamas gravelly loam	C/D	2.4	100.0%
Totals for Area of Interest			**2.4**	**100.0%**

Description

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

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

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

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

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

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

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

Rating Options

Aggregation Method: Dominant Condition

Component Percent Cutoff: None Specified

Tie-break Rule: Higher

APPENDIX C

BASIN MAP

Developed Basin Map

APPENDIX D

HYDROCAD SUMMARIES

Summary for Subcatchment 6S: Pre-Dev Site

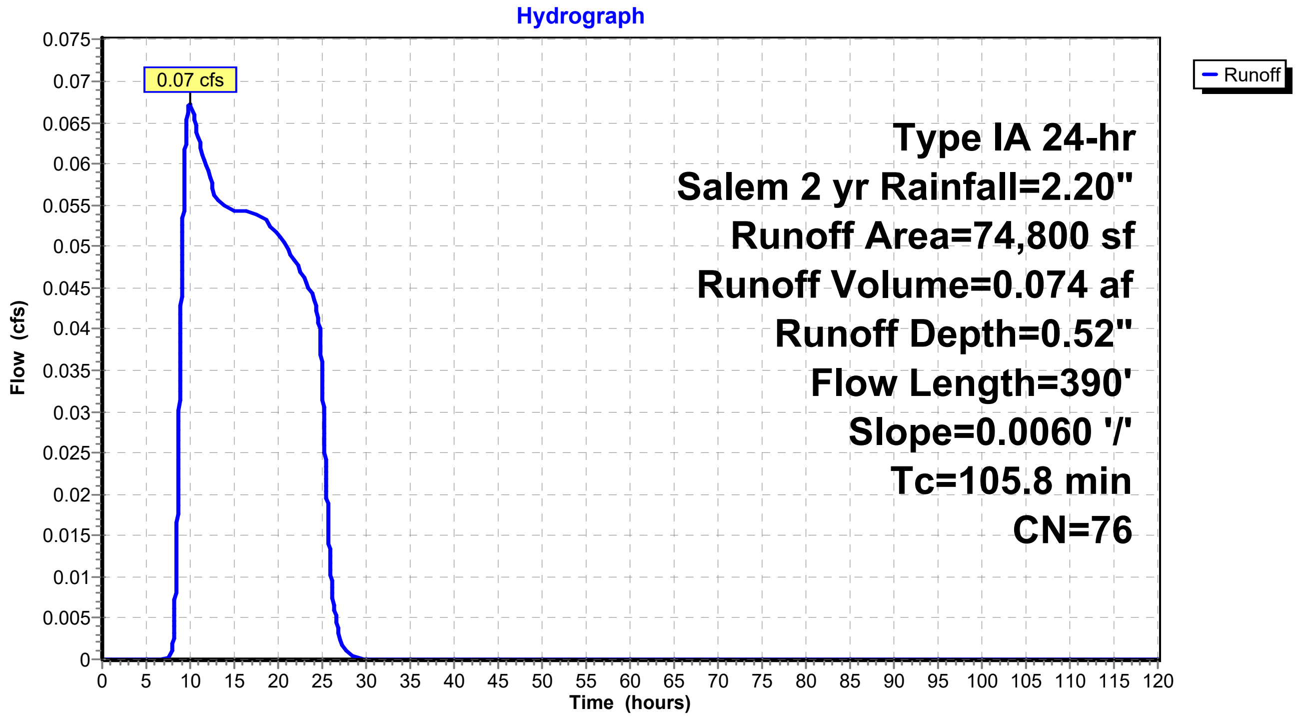
Runoff = 0.07 cfs @ 9.99 hrs, Volume= 0.074 af, Depth= 0.52"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-120.00 hrs, dt= 0.01 hrs
Type IA 24-hr Salem 2 yr Rainfall=2.20"

	Area (sf)	CN	Description
*	74,800	76	Predeveloped Soil C/D
	74,800		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
103.0	300	0.0060	0.05		**Sheet Flow, Sheet Flow** Grass: Bermuda n= 0.410 P2= 2.20"
2.8	90	0.0060	0.54		**Shallow Concentrated Flow,** Short Grass Pasture Kv= 7.0 fps
105.8	390	Total			

Subcatchment 6S: Pre-Dev Site



Summary for Subcatchment 6S: Pre-Dev Site

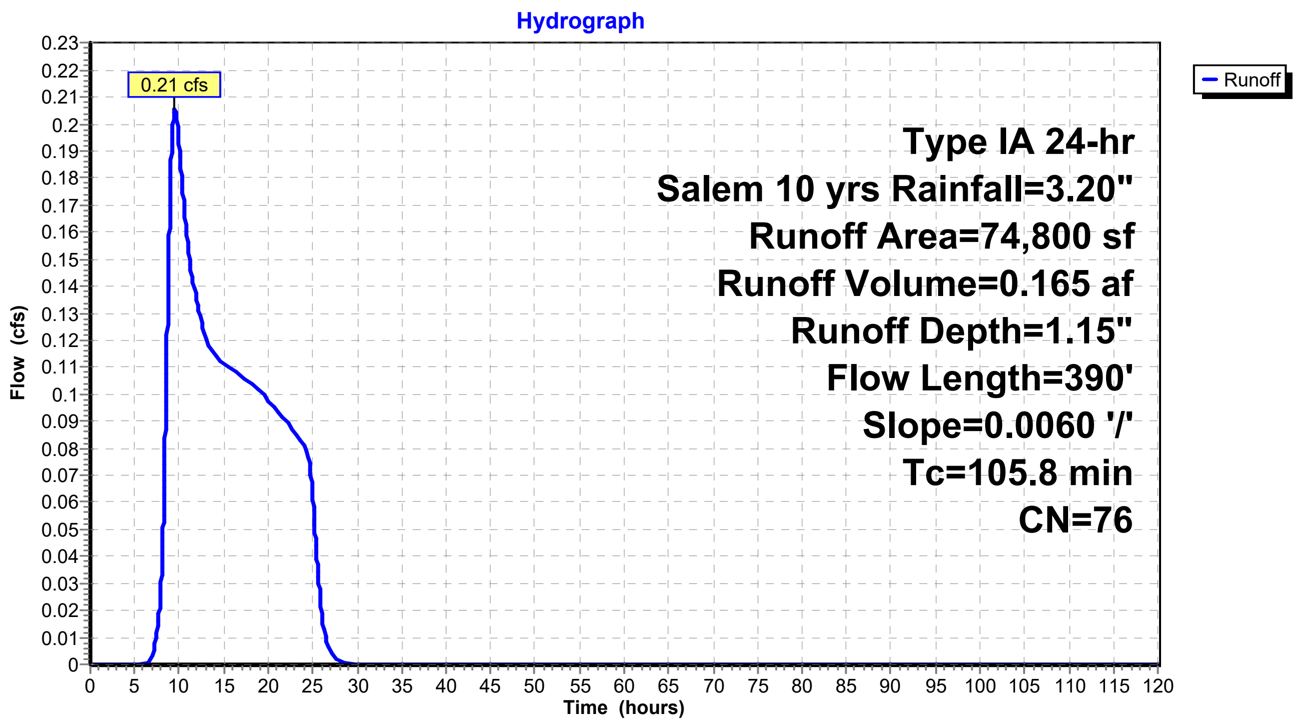
Runoff = 0.21 cfs @ 9.52 hrs, Volume= 0.165 af, Depth= 1.15"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-120.00 hrs, dt= 0.01 hrs
Type IA 24-hr Salem 10 yrs Rainfall=3.20"

	Area (sf)	CN	Description
*	74,800	76	Predeveloped Soil C/D
	74,800		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
103.0	300	0.0060	0.05		**Sheet Flow, Sheet Flow** Grass: Bermuda n= 0.410 P2= 2.20"
2.8	90	0.0060	0.54		**Shallow Concentrated Flow,** Short Grass Pasture Kv= 7.0 fps
105.8	390	Total			

Subcatchment 6S: Pre-Dev Site



Summary for Subcatchment 6S: Pre-Dev Site

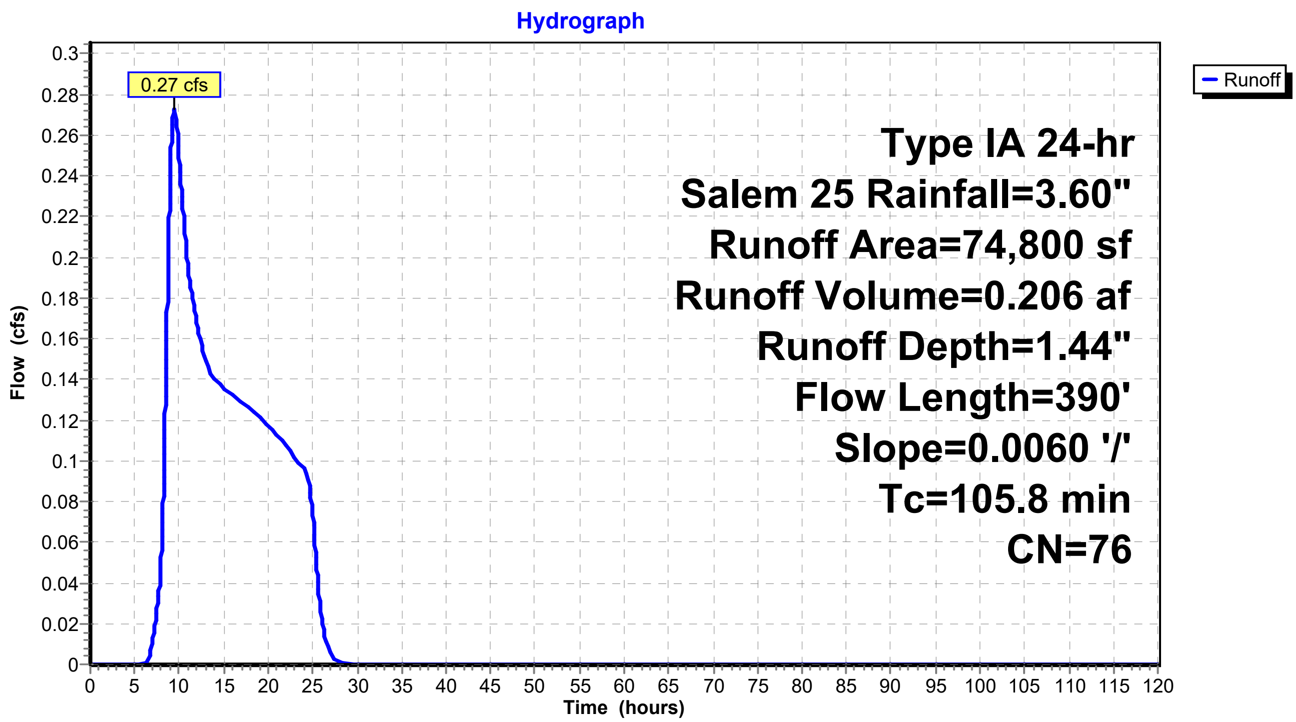
Runoff = 0.27 cfs @ 9.52 hrs, Volume= 0.206 af, Depth= 1.44"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-120.00 hrs, dt= 0.01 hrs
Type IA 24-hr Salem 25 Rainfall=3.60"

	Area (sf)	CN	Description
*	74,800	76	Predeveloped Soil C/D
	74,800		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
103.0	300	0.0060	0.05		**Sheet Flow, Sheet Flow** Grass: Bermuda n= 0.410 P2= 2.20"
2.8	90	0.0060	0.54		**Shallow Concentrated Flow,** Short Grass Pasture Kv= 7.0 fps
105.8	390	Total			

Subcatchment 6S: Pre-Dev Site



Summary for Subcatchment 6S: Pre-Dev Site

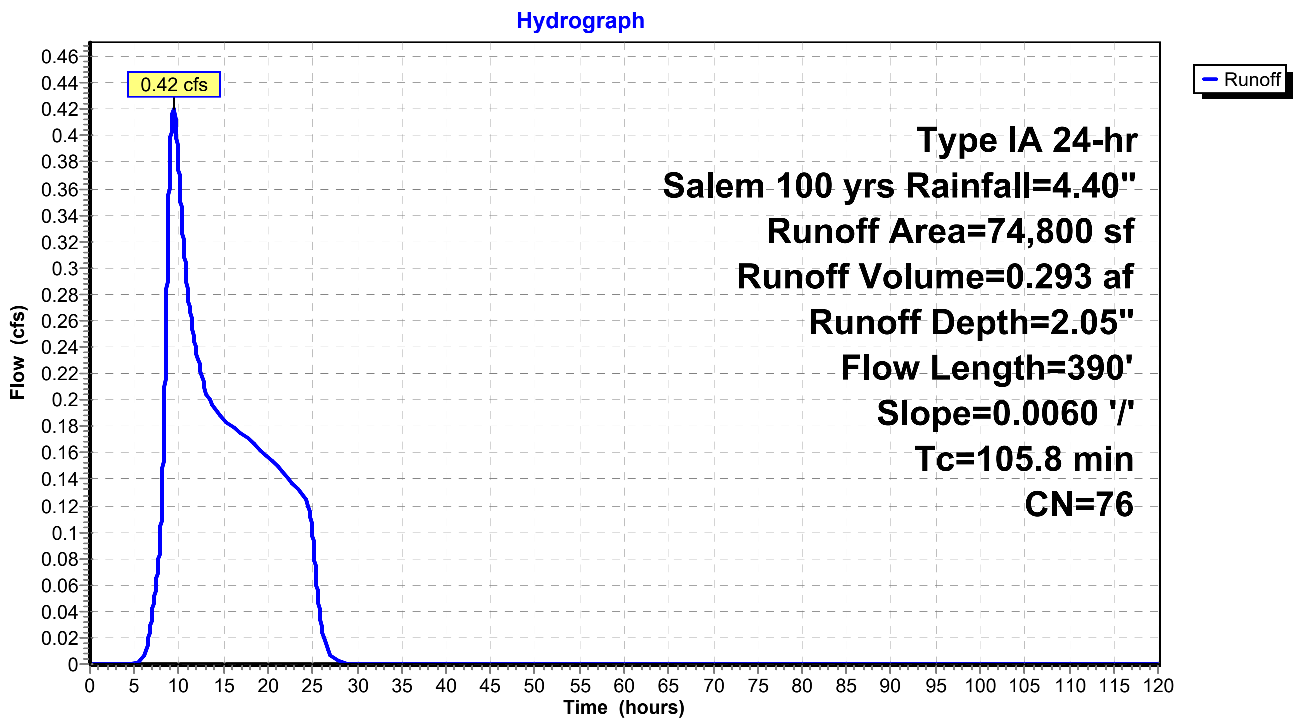
Runoff = 0.42 cfs @ 9.40 hrs, Volume= 0.293 af, Depth= 2.05"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-120.00 hrs, dt= 0.01 hrs
Type IA 24-hr Salem 100 yrs Rainfall=4.40"

	Area (sf)	CN	Description
*	74,800	76	Predeveloped Soil C/D
	74,800		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
103.0	300	0.0060	0.05		**Sheet Flow, Sheet Flow** Grass: Bermuda n= 0.410 P2= 2.20"
2.8	90	0.0060	0.54		**Shallow Concentrated Flow,** Short Grass Pasture Kv= 7.0 fps
105.8	390	Total			

Subcatchment 6S: Pre-Dev Site



Summary for Subcatchment 28S: Developed Basin

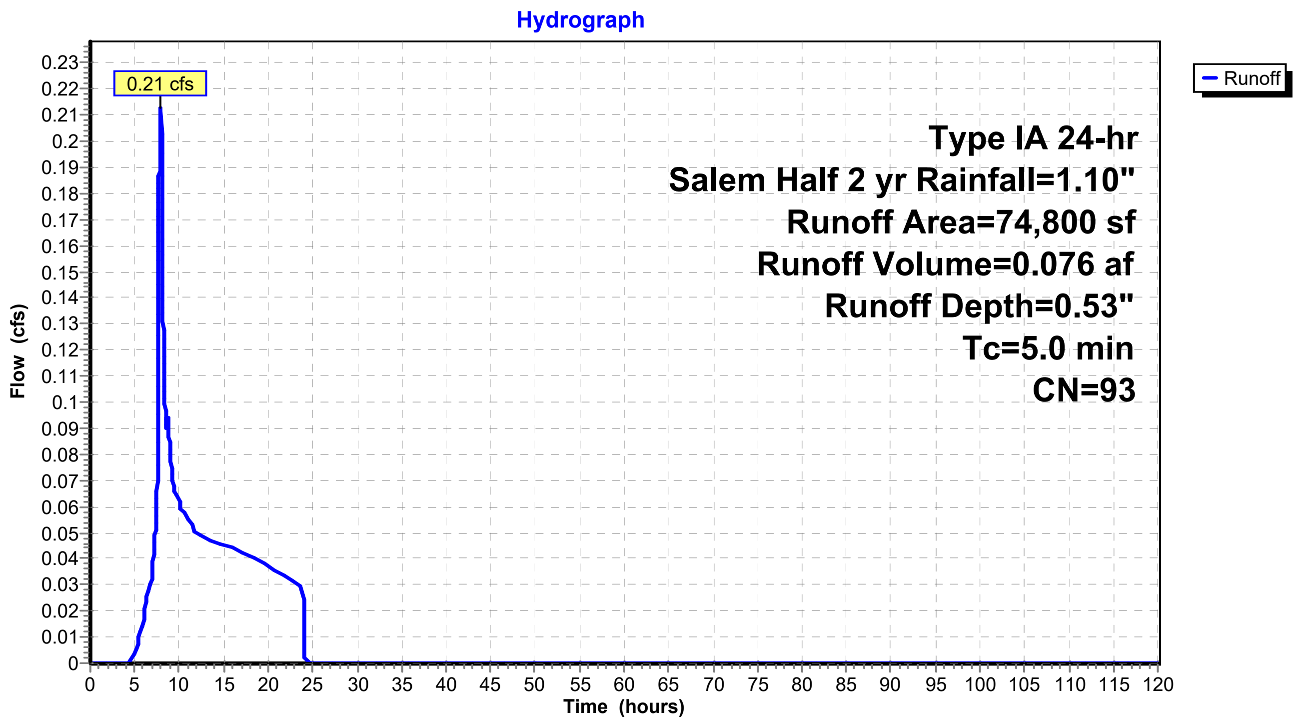
Runoff = 0.21 cfs @ 7.97 hrs, Volume= 0.076 af, Depth= 0.53"
Routed to Pond 29P : RG1

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-120.00 hrs, dt= 0.01 hrs
Type IA 24-hr Salem Half 2 yr Rainfall=1.10"

Area (sf)	CN	Description
74,800	93	Urban industrial, 72% imp, HSG D
20,944		28.00% Pervious Area
53,856		72.00% Impervious Area

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

Subcatchment 28S: Developed Basin



Summary for Subcatchment 28S: Developed Basin

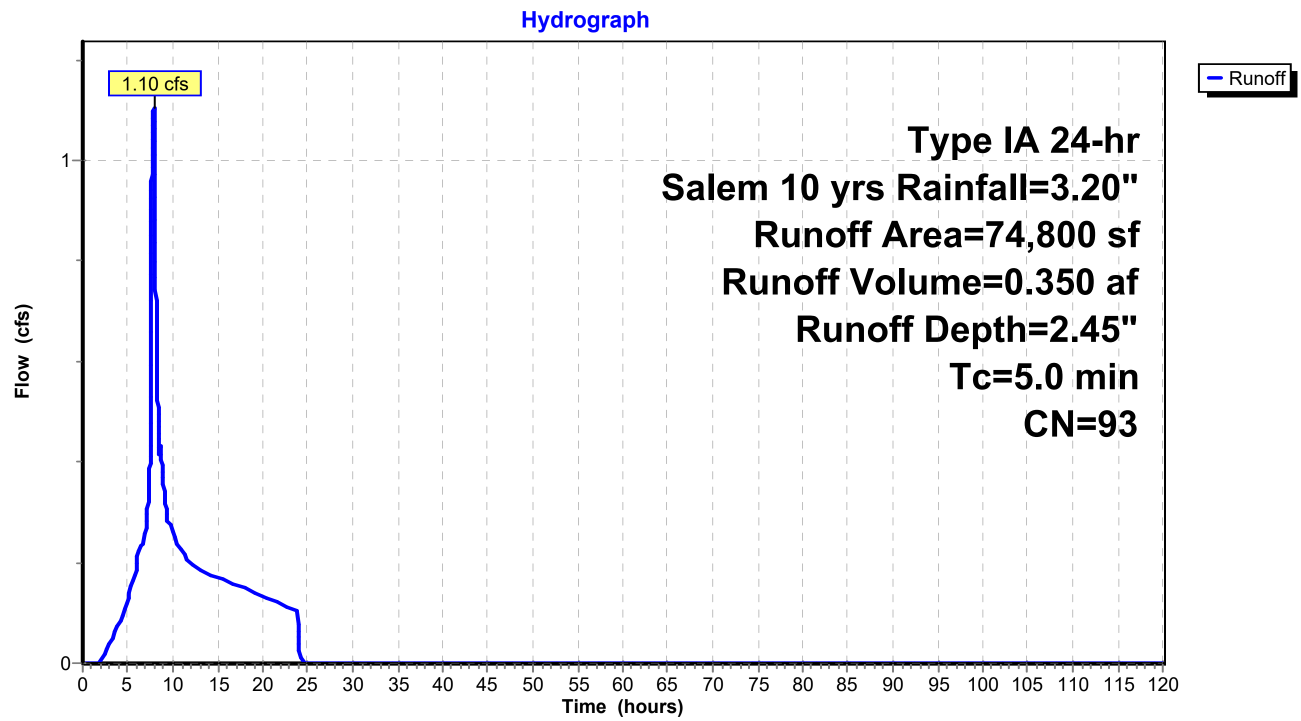
Runoff = 1.10 cfs @ 7.89 hrs, Volume= 0.350 af, Depth= 2.45"
Routed to Pond 29P : RG1

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-120.00 hrs, dt= 0.01 hrs
Type IA 24-hr Salem 10 yrs Rainfall=3.20"

Area (sf)	CN	Description
74,800	93	Urban industrial, 72% imp, HSG D
20,944		28.00% Pervious Area
53,856		72.00% Impervious Area

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

Subcatchment 28S: Developed Basin



Summary for Subcatchment 28S: Developed Basin

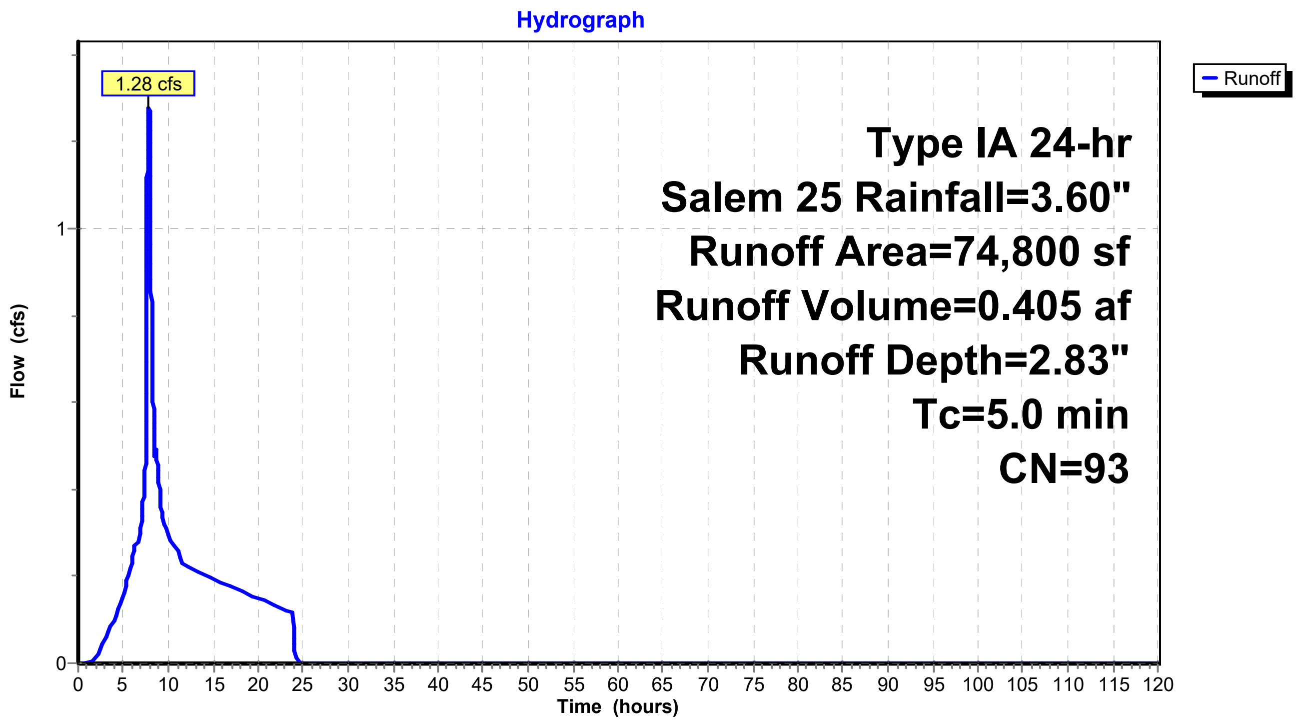
Runoff = 1.28 cfs @ 7.88 hrs, Volume= 0.405 af, Depth= 2.83"
Routed to Pond 29P : RG1

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-120.00 hrs, dt= 0.01 hrs
Type IA 24-hr Salem 25 Rainfall=3.60"

Area (sf)	CN	Description
74,800	93	Urban industrial, 72% imp, HSG D
20,944		28.00% Pervious Area
53,856		72.00% Impervious Area

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

Subcatchment 28S: Developed Basin



Summary for Subcatchment 28S: Developed Basin

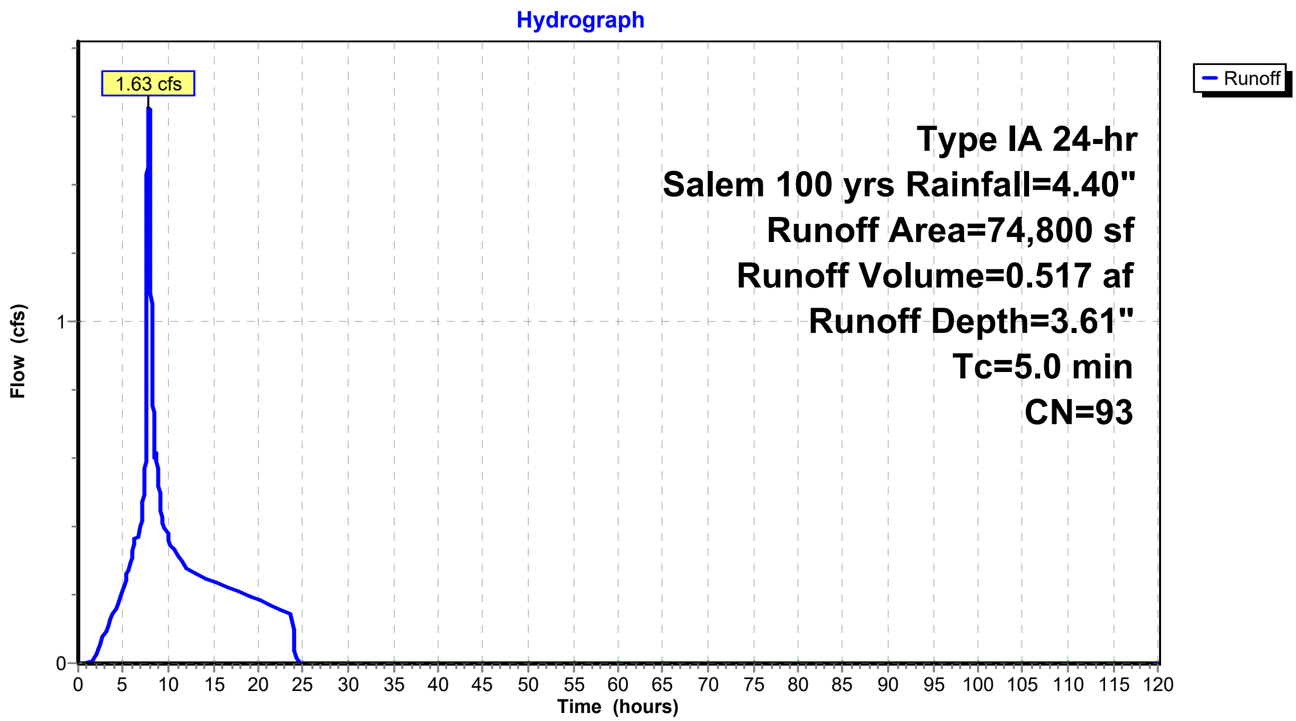
Runoff = 1.63 cfs @ 7.88 hrs, Volume= 0.517 af, Depth= 3.61"
Routed to Pond 29P : RG1

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-120.00 hrs, dt= 0.01 hrs
Type IA 24-hr Salem 100 yrs Rainfall=4.40"

Area (sf)	CN	Description
74,800	93	Urban industrial, 72% imp, HSG D
20,944		28.00% Pervious Area
53,856		72.00% Impervious Area

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

Subcatchment 28S: Developed Basin



Summary for Pond 29P: RG1

Inflow Area = 1.717 ac, 72.00% Impervious, Inflow Depth = 0.53" for Salem Half 2 yr event
Inflow = 0.21 cfs @ 7.97 hrs, Volume= 0.076 af
Outflow = 0.12 cfs @ 8.28 hrs, Volume= 0.076 af, Atten= 46%, Lag= 18.5 min
Discarded = 0.12 cfs @ 8.28 hrs, Volume= 0.076 af
Primary = 0.00 cfs @ 0.00 hrs, Volume= 0.000 af

Routing by Stor-Ind method, Time Span= 0.00-120.00 hrs, dt= 0.01 hrs
Peak Elev= 172.06' @ 8.28 hrs Surf.Area= 1,988 sf Storage= 149 cf

Plug-Flow detention time= (not calculated: outflow precedes inflow)
Center-of-Mass det. time= 3.2 min (809.1 - 805.9)

Volume	Invert	Avail.Storage	Storage Description
#1	170.49'	16,105 cf	**Custom Stage Data (Conic)** Listed below (Recalc)

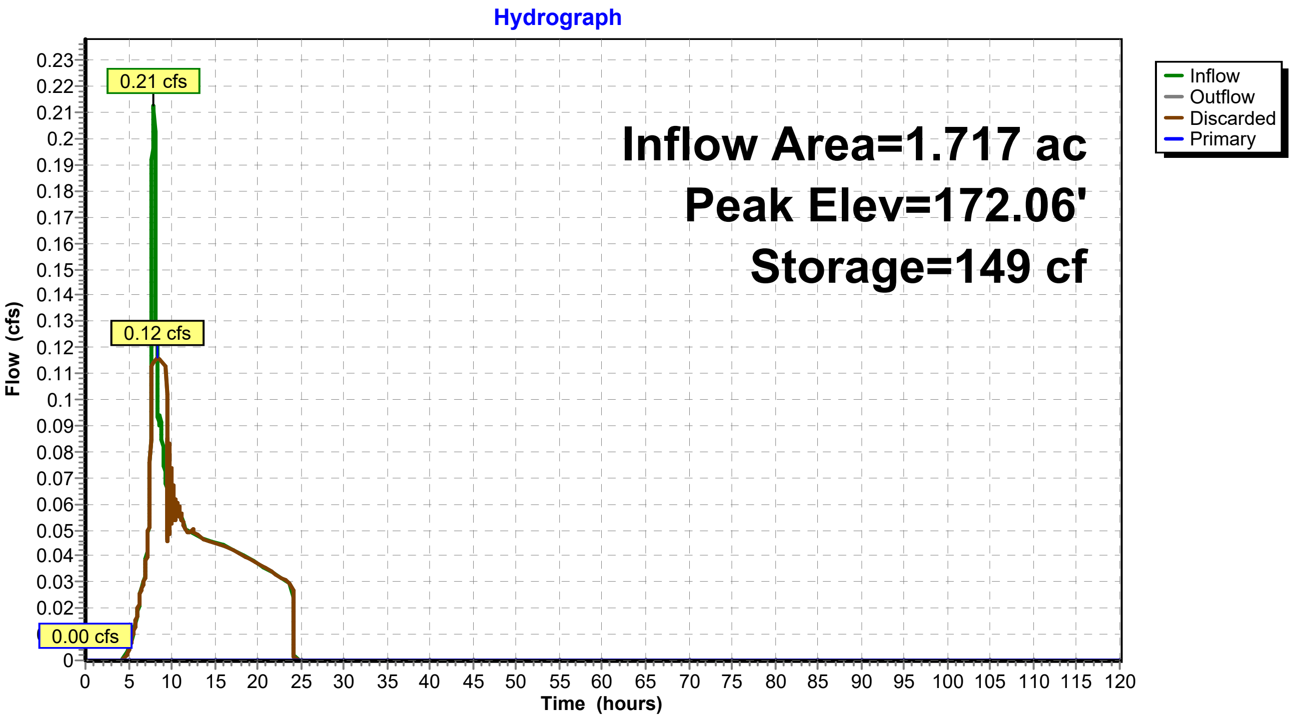
Elevation (feet)	Surf.Area (sq-ft)	Voids (%)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)	Wet.Area (sq-ft)
170.49	1,930	0.0	0	0	1,930
170.50	1,930	0.1	0	0	1,932
171.99	1,930	0.1	3	3	2,164
172.00	1,930	100.0	19	22	2,165
173.00	2,920	100.0	2,408	2,430	3,170
174.00	3,980	100.0	3,436	5,867	4,250
175.00	5,130	100.0	4,543	10,409	5,425
176.00	6,280	100.0	5,695	16,105	6,606

Device	Routing	Invert	Outlet Devices
#1	Discarded	170.49'	**2.250 in/hr Exfiltration over Wetted area**
#2	Primary	174.50'	**18.0" Vert. Orifice/Grate** C= 0.600 Limited to weir flow at low heads

Discarded OutFlow Max=0.12 cfs @ 8.28 hrs HW=172.06' (Free Discharge)
└─**1=Exfiltration** (Exfiltration Controls 0.12 cfs)

Primary OutFlow Max=0.00 cfs @ 0.00 hrs HW=170.49' (Free Discharge)
└─**2=Orifice/Grate** (Controls 0.00 cfs)

Pond 29P: RG1


Hydrograph
0.21 cfs
0.12 cfs
0.00 cfs
Inflow Area=1.717 ac
Peak Elev=172.06'
Storage=149 cf
Inflow
Outflow
Discarded
Primary
Flow (cfs)
Time (hours)

Summary for Pond 29P: RG1

Inflow Area = 1.717 ac, 72.00% Impervious, Inflow Depth = 2.45" for Salem 10 yrs event
Inflow = 1.10 cfs @ 7.89 hrs, Volume= 0.350 af
Outflow = 0.19 cfs @ 11.77 hrs, Volume= 0.350 af, Atten= 82%, Lag= 232.9 min
Discarded = 0.19 cfs @ 11.77 hrs, Volume= 0.350 af
Primary = 0.00 cfs @ 0.00 hrs, Volume= 0.000 af

Routing by Stor-Ind method, Time Span= 0.00-120.00 hrs, dt= 0.01 hrs
Peak Elev= 173.54' @ 11.77 hrs Surf.Area= 3,474 sf Storage= 4,161 cf

Plug-Flow detention time= 258.9 min calculated for 0.350 af (100% of inflow)
Center-of-Mass det. time= 259.0 min (979.2 - 720.3)

Volume	Invert	Avail.Storage	Storage Description
#1	170.49'	16,105 cf	**Custom Stage Data (Conic)** Listed below (Recalc)

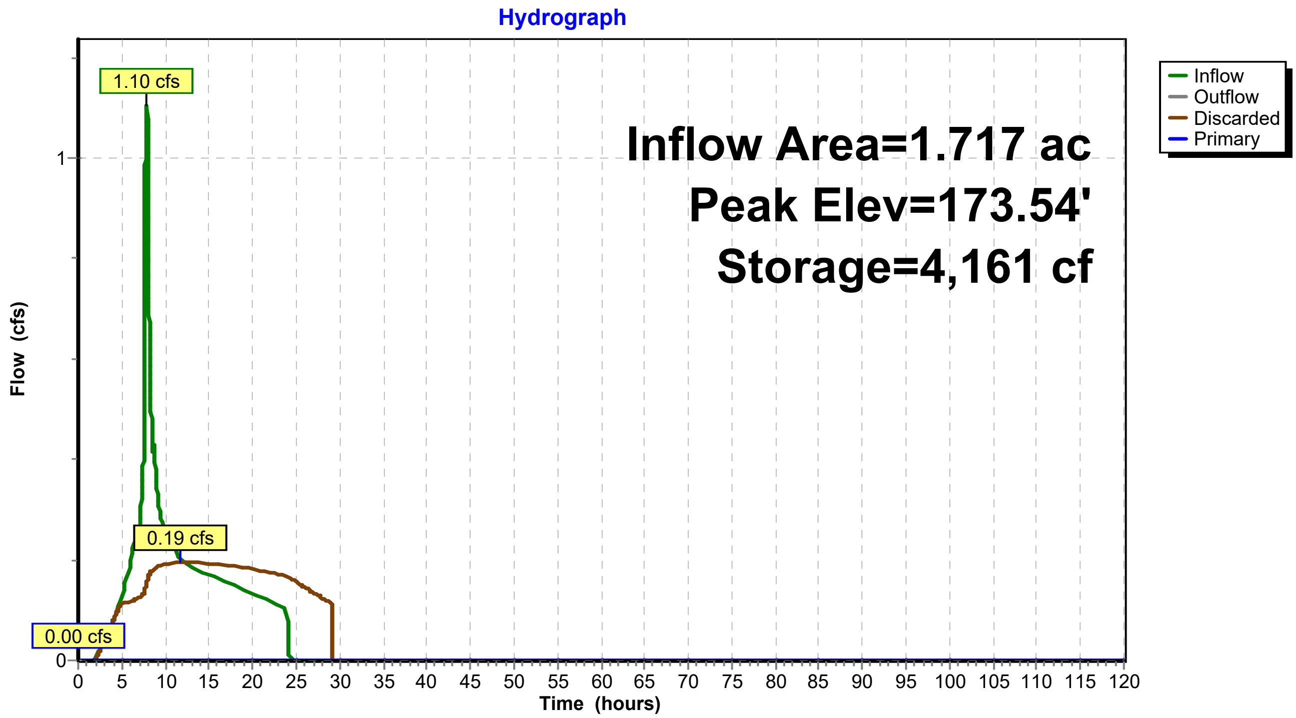
Elevation (feet)	Surf.Area (sq-ft)	Voids (%)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)	Wet.Area (sq-ft)
170.49	1,930	0.0	0	0	1,930
170.50	1,930	0.1	0	0	1,932
171.99	1,930	0.1	3	3	2,164
172.00	1,930	100.0	19	22	2,165
173.00	2,920	100.0	2,408	2,430	3,170
174.00	3,980	100.0	3,436	5,867	4,250
175.00	5,130	100.0	4,543	10,409	5,425
176.00	6,280	100.0	5,695	16,105	6,606

Device	Routing	Invert	Outlet Devices
#1	Discarded	170.49'	**2.250 in/hr Exfiltration over Wetted area**
#2	Primary	174.50'	**18.0" Vert. Orifice/Grate** C= 0.600 Limited to weir flow at low heads

Discarded OutFlow Max=0.19 cfs @ 11.77 hrs HW=173.54' (Free Discharge)
└─**1=Exfiltration** (Exfiltration Controls 0.19 cfs)

Primary OutFlow Max=0.00 cfs @ 0.00 hrs HW=170.49' (Free Discharge)
└─**2=Orifice/Grate** (Controls 0.00 cfs)

Pond 29P: RG1


Hydrograph
1.10 cfs
0.19 cfs
0.00 cfs
Inflow Area=1.717 ac
Peak Elev=173.54'
Storage=4,161 cf
Inflow
Outflow
Discarded
Primary
Flow (cfs)
Time (hours)
1
0
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
85
90
95
100
105
110
115
120

Summary for Pond 29P: RG1

Inflow Area = 1.717 ac, 72.00% Impervious, Inflow Depth = 2.83" for Salem 25 event
Inflow = 1.28 cfs @ 7.88 hrs, Volume= 0.405 af
Outflow = 0.21 cfs @ 13.30 hrs, Volume= 0.405 af, Atten= 83%, Lag= 324.8 min
Discarded = 0.21 cfs @ 13.30 hrs, Volume= 0.405 af
Primary = 0.00 cfs @ 0.00 hrs, Volume= 0.000 af

Routing by Stor-Ind method, Time Span= 0.00-120.00 hrs, dt= 0.01 hrs
Peak Elev= 173.85' @ 13.30 hrs Surf.Area= 3,807 sf Storage= 5,271 cf

Plug-Flow detention time= 305.7 min calculated for 0.405 af (100% of inflow)
Center-of-Mass det. time= 305.8 min (1,019.1 - 713.4)

Volume	Invert	Avail.Storage	Storage Description
#1	170.49'	16,105 cf	**Custom Stage Data (Conic)** Listed below (Recalc)

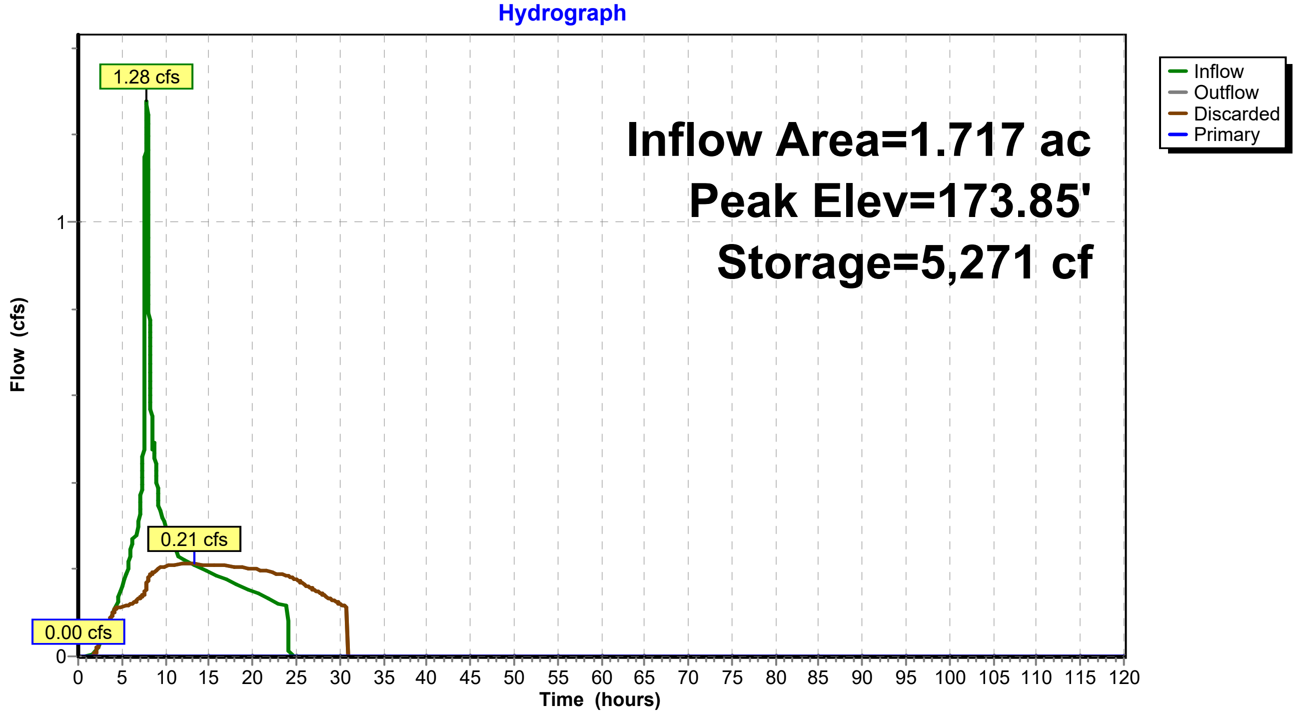
Elevation (feet)	Surf.Area (sq-ft)	Voids (%)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)	Wet.Area (sq-ft)
170.49	1,930	0.0	0	0	1,930
170.50	1,930	0.1	0	0	1,932
171.99	1,930	0.1	3	3	2,164
172.00	1,930	100.0	19	22	2,165
173.00	2,920	100.0	2,408	2,430	3,170
174.00	3,980	100.0	3,436	5,867	4,250
175.00	5,130	100.0	4,543	10,409	5,425
176.00	6,280	100.0	5,695	16,105	6,606

Device	Routing	Invert	Outlet Devices
#1	Discarded	170.49'	**2.250 in/hr Exfiltration over Wetted area**
#2	Primary	174.50'	**18.0" Vert. Orifice/Grate** C= 0.600 Limited to weir flow at low heads

Discarded OutFlow Max=0.21 cfs @ 13.30 hrs HW=173.85' (Free Discharge)
└─**1=Exfiltration** (Exfiltration Controls 0.21 cfs)

Primary OutFlow Max=0.00 cfs @ 0.00 hrs HW=170.49' (Free Discharge)
└─**2=Orifice/Grate** (Controls 0.00 cfs)

Pond 29P: RG1


Hydrograph
1.28 cfs
0.21 cfs
0.00 cfs
Inflow Area=1.717 ac
Peak Elev=173.85'
Storage=5,271 cf
Inflow
Outflow
Discarded
Primary
Flow (cfs)
Time (hours)

Summary for Pond 29P: RG1

Inflow Area = 1.717 ac, 72.00% Impervious, Inflow Depth = 3.61" for Salem 100 yrs event
Inflow = 1.63 cfs @ 7.88 hrs, Volume= 0.517 af
Outflow = 0.25 cfs @ 14.31 hrs, Volume= 0.517 af, Atten= 85%, Lag= 386.3 min
Discarded = 0.25 cfs @ 14.31 hrs, Volume= 0.517 af
Primary = 0.00 cfs @ 0.00 hrs, Volume= 0.000 af

Routing by Stor-Ind method, Time Span= 0.00-120.00 hrs, dt= 0.01 hrs
Peak Elev= 174.43' @ 14.31 hrs Surf.Area= 4,454 sf Storage= 7,670 cf

Plug-Flow detention time= 388.9 min calculated for 0.517 af (100% of inflow)
Center-of-Mass det. time= 388.9 min (1,091.5 - 702.5)

Volume	Invert	Avail.Storage	Storage Description
#1	170.49'	16,105 cf	**Custom Stage Data (Conic)** Listed below (Recalc)

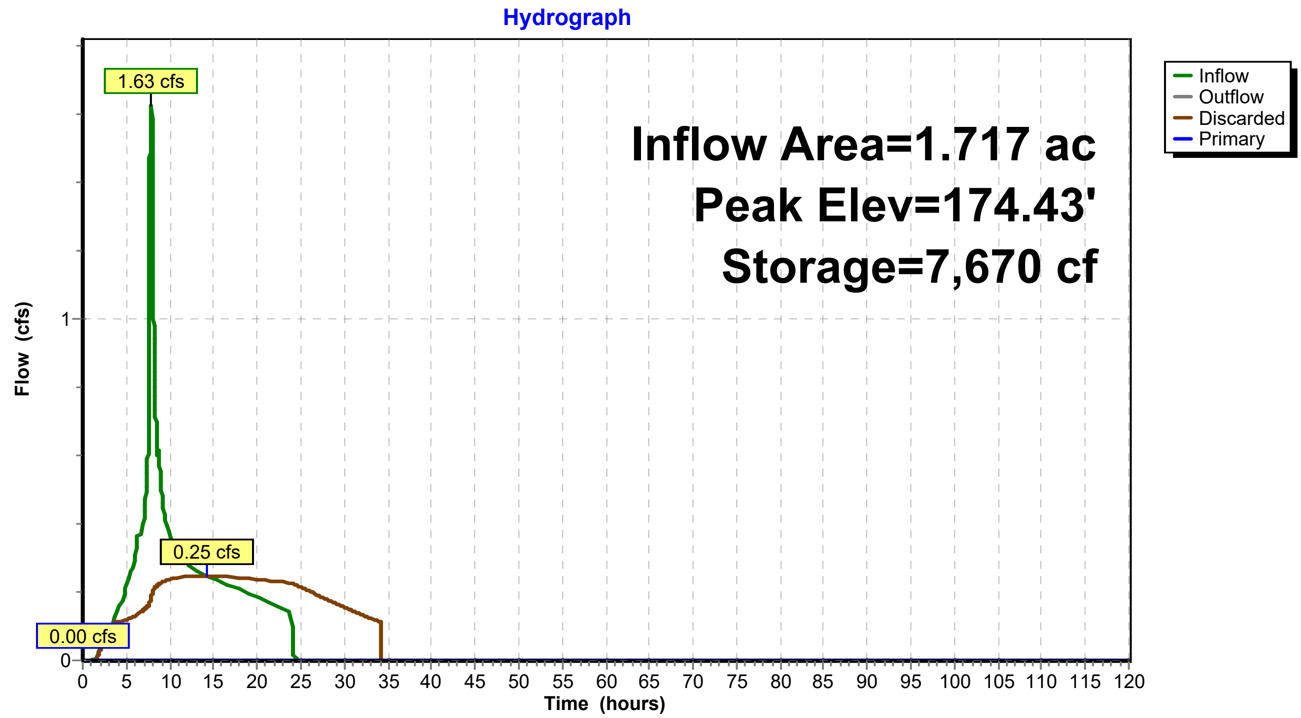
Elevation (feet)	Surf.Area (sq-ft)	Voids (%)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)	Wet.Area (sq-ft)
170.49	1,930	0.0	0	0	1,930
170.50	1,930	0.1	0	0	1,932
171.99	1,930	0.1	3	3	2,164
172.00	1,930	100.0	19	22	2,165
173.00	2,920	100.0	2,408	2,430	3,170
174.00	3,980	100.0	3,436	5,867	4,250
175.00	5,130	100.0	4,543	10,409	5,425
176.00	6,280	100.0	5,695	16,105	6,606

Device	Routing	Invert	Outlet Devices
#1	Discarded	170.49'	**2.250 in/hr Exfiltration over Wetted area**
#2	Primary	174.50'	**18.0" Vert. Orifice/Grate** C= 0.600 Limited to weir flow at low heads

Discarded OutFlow Max=0.25 cfs @ 14.31 hrs HW=174.43' (Free Discharge)
└─**1=Exfiltration** (Exfiltration Controls 0.25 cfs)

Primary OutFlow Max=0.00 cfs @ 0.00 hrs HW=170.49' (Free Discharge)
└─**2=Orifice/Grate** (Controls 0.00 cfs)

Pond 29P: RG1


Hydrograph
1.63 cfs
0.25 cfs
0.00 cfs
Inflow Area=1.717 ac
Peak Elev=174.43'
Storage=7,670 cf
Inflow
Outflow
Discarded
Primary
Flow (cfs)
Time (hours)

Summary for Pond 29P: RG1

Inflow Area =	1.717 ac,	72.00% Impervious,	Inflow Depth = 0.76"	for Salem Water Quality event
Inflow =	0.32 cfs @	7.95 hrs, Volume=	0.109 af	
Outflow =	0.12 cfs @	8.88 hrs, Volume=	0.109 af,	Atten= 62%, Lag= 56.3 min
Discarded =	0.12 cfs @	8.88 hrs, Volume=	0.109 af	
Primary =	0.00 cfs @	0.00 hrs, Volume=	0.000 af	

Routing by Stor-Ind method, Time Span= 0.00-120.00 hrs, dt= 0.01 hrs
Peak Elev= 172.19' @ 8.88 hrs Surf.Area= 2,105 sf Storage= 412 cf

Plug-Flow detention time= (not calculated: outflow precedes inflow)
Center-of-Mass det. time= 14.9 min (798.7 - 783.7)

Volume	Invert	Avail.Storage	Storage Description
#1	170.49'	16,105 cf	**Custom Stage Data (Conic)** Listed below (Recalc)

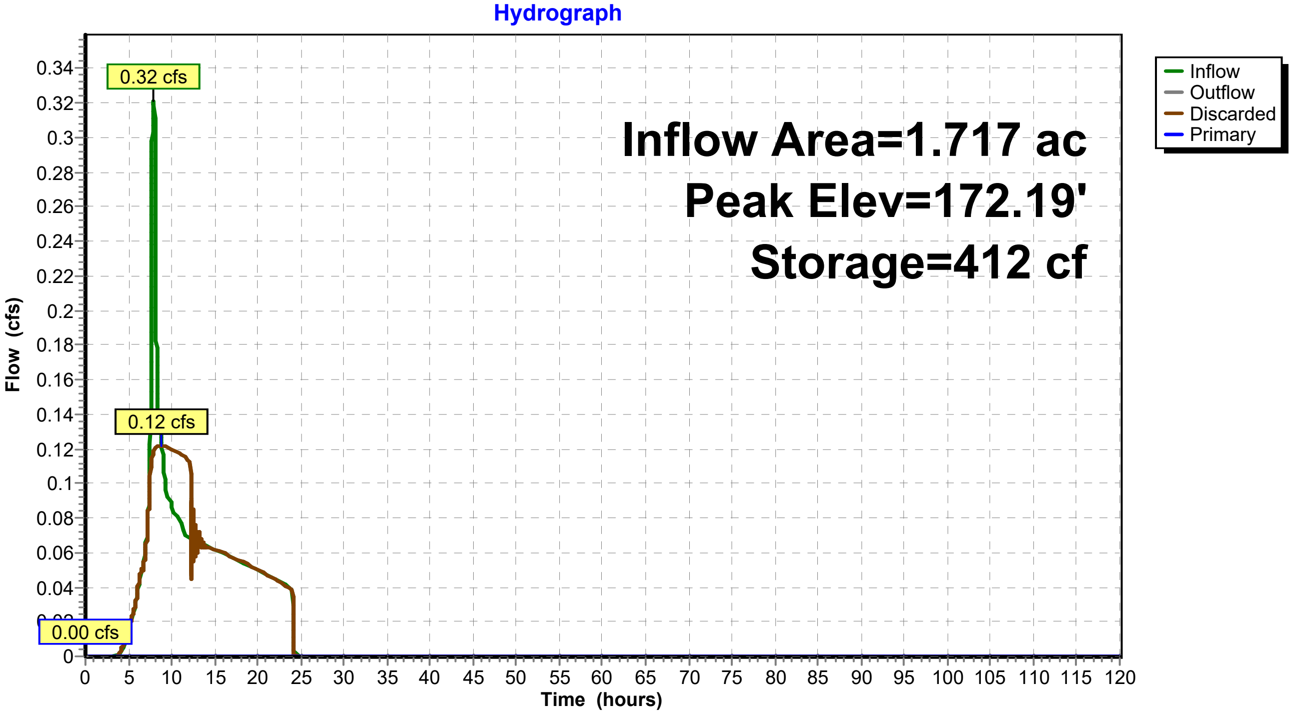
Elevation (feet)	Surf.Area (sq-ft)	Voids (%)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)	Wet.Area (sq-ft)
170.49	1,930	0.0	0	0	1,930
170.50	1,930	0.1	0	0	1,932
171.99	1,930	0.1	3	3	2,164
172.00	1,930	100.0	19	22	2,165
173.00	2,920	100.0	2,408	2,430	3,170
174.00	3,980	100.0	3,436	5,867	4,250
175.00	5,130	100.0	4,543	10,409	5,425
176.00	6,280	100.0	5,695	16,105	6,606

Device	Routing	Invert	Outlet Devices
#1	Discarded	170.49'	**2.250 in/hr Exfiltration over Wetted area**
#2	Primary	174.50'	**18.0" Vert. Orifice/Grate** C= 0.600 Limited to weir flow at low heads

Discarded OutFlow Max=0.12 cfs @ 8.88 hrs HW=172.19' (Free Discharge)
└─**1=Exfiltration** (Exfiltration Controls 0.12 cfs)

Primary OutFlow Max=0.00 cfs @ 0.00 hrs HW=170.49' (Free Discharge)
└─**2=Orifice/Grate** (Controls 0.00 cfs)

Pond 29P: RG1


Hydrograph
0.32 cfs
0.12 cfs
0.00 cfs
Inflow Area=1.717 ac
Peak Elev=172.19'
Storage=412 cf
Inflow
Outflow
Discarded
Primary
Flow (cfs)
Time (hours)