7586

PRELIMINARY DRAINAGE REPORT
FOR

Liberty Road Apartments West
Salem, Oregon

Prepared For:
Harrison Industries, LLC
10355 Liberty Road S
Salem, Oregon 97306

June 26, 2024

REGISTERED PROFESSIONAL ENGINEER
80415
Natalie Janney
OREGON
JANUARY 3, 2011
NATALIE G. JANNEY
Renew date: 6.30.2025

MULTI/TECH
ENGINEERING SERVICES, INC.

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Contents

INTRODUCTION

The Liberty Road Apartments is a proposed apartment complex with 156 units in 14 buildings. The development is located at 5871 Liberty Road S. The parcel of land to be developed is a portion of Tax Lot 600 of Marion County Assessor's Map 08 3W 16C. A vicinity map and supporting maps are in Appendix A of this report. An aerial image of the site can be seen below.

Figure 1: Project Site

The proposed apartment site will develop approximately 6.72-acres of the 15.74-acre site. The Liberty Road subdivision is currently under construction and has developed approximately 5.94-acres.

Green Stormwater Infrastructure (GSI) to the Maximum Extent Feasible (MEF) is being used for the new developed areas per City of Salem Administrative Rules, Chapter 109, Division 004, Stormwater System, Appendix 4E and Ordinance No. 8-20 (Standards). All facilities will be constructed to meet the City of Salem standards.

Existing Conditions

The entire 15.74-acre site is rectangular in the shape. Surface conditions consists of grass, brush and minimal trees. There are no identified wetlands, streams or sensitive areas located on the property. A topographical high point is located on the westerly side of the site. Drainage from this high point flows predominately easterly, with the low near the intersection of Rise Street and Big Mountain Avenue. The maximum relief is approximately 110-feet with a high point elevation of 650. The abutting properties are zoned residential with nearby public improvements that include storm water conveyance systems. Appendix A contains multiple maps of the site.

Soils

The Natural Resources Conservation Service (NRCS) Soil Resource Report for Marion County was used to determine a Hydrological Soil Group classification for runoff calculations. The report identifies the site soils to be Jory, Nekia and McAlpin soils. The predominate soils are in the hydrologic soil group C. The report is in Appendix B.

Infiltration

A geotechnical investigation of the site was performed by Redmond Geotechnical Services. Several exploratory tests were completed as part of the investigation. Groundwater was not encountered during the time of investigation to depths of at least 9 feet beneath existing surface grades. Based upon the results of the field testing at the site, recommend design infiltration rates were below 0.10 inches per hour. Per the geotechnical report, infiltration is not a viable mechanism to address storm water runoff and was not considered. A copy of the report is in Appendix B.

Water Quality Methodology

Water Quality treatment is being provided by an existing combination facility that was built with the Liberty Road Subdivision. It was previously designed with the treatment of the Western Liberty Road Apartments to be treated within the facility.

Water Quality Analysis

Water quality flow rates were calculated with HydroCAD 10.20. The SCS TR-20 Unit Hydrograph method was used to generate the hydrographs. A Type 1A storm and a 24-hour rainfall depth of 1.38 inches per hour was used to determine the water quality flow rate.

Stormwater Quantity Analysis

Stormwater quantity (Flow Control) is being handled by on-site detention. Runoff from a 6.72-acre portion is being routed to an on-site facility that ultimately controls runoff to pre-developed flow rates.

Per Subsection 4.2(p)(3)(A) of the standards, one-half of the post development peak runoff rate must be equal to or less than one-half of the peak runoff rate of the pre-developed for half of the two-year, 10, 25, and 100-year, 24-hour storm events.

The pre-developed flow rates were calculated using HydroCAD 10.20. Table 1 below lists the 24-hour rainfall depths used in the analysis of each storm event.

Table 1: 24-hour rainfall depths (in) for all design storms

Storm Event	24-hour Rainfall Depth (in)
1/2 - 2	1.1
10	3.2
25	3.6
100	4.4
WQ	1.38

For the pre-developed conditions, a time of concentration of 31.5 minutes was calculated for the entire Liberty Subdivision site. The time of concentration data is in Appendix C. The calculations are incorporated in the HydroCAD output located in Appendix D. The entire area was classified as “City of Salem Pre-Development, HSG C” with a Curve Number (CN) of 72. A pre-developed basin map is in Appendix A.

The SCS TR-20 Unit Hydrograph method was used to generate the hydrographs. A Type 1A rainfall distribution was used with the above rainfall depths. Table 2 below identifies the allowable pre-developed release rates for each storm event.

Table 2: Pre-developed release rate (cfs)

Storm Event	Pre-developed Release Rate (cfs)
1/2 - 2	0.02
10	0.85
25	1.22
100	2.04

The post-developed flow rates were calculated using HydroCAD 10.20. A time of concentration of 10 minutes was estimated for the developed basin. This time of concentration was based upon proposed cover conditions and pipe slopes. The time of concentration was incorporated in the HydroCAD output located in Appendix D. The site was classified as 60% Impervious and 40% landscaping, HSG C with a composite Curve Number (CN) of 88 that was based upon proposed cover conditions. Table 3 below lists the CN values for the developed basin areas. A developed basin map is in Appendix A.

Table 3: Developed Basin Summary

Basin	Landscaping HSG C (ac) CN = 74	Impervious HSG C (ac) CN = 98	Total (ac)	Composite CN
Site	1.888	4.030	5.918	88
Undetained	0.800	0.000	0.800	74

Detention Analysis

In the detention analysis, the 6.72-acre site was considered two basins, with 5.920 acres draining into the detention facility and 0.800 acres leaving the site undetained. A basin map is in Appendix A. The release rate for the undetained runoff leaving the site is subtracted from the pre-developed allowable release. The adjusted release rate can be found in TABLE X. This is the maximum flowrate that can leave the detention pond in order to meet the City of Salem performance requirements for pre to post detention.

Site grading and conveyance pipe will direct stormwater runoff to the system. It should be noted that the facility has a capacity to contain 26,857 cubic feet of water. This exceeds the required detention volume of 21,859 cubic feet for the developed portion.

Table 4: Allowable release rate summary table

Storm Event	Pre-developed Release Rate (cfs)	Undetained Release Rate (cfs)	Adjusted Allowable Release Rate (cfs)
1/2 - 2	0.02	0.00	0.02
10	0.85	0.16	0.69
25	1.22	0.22	1.00
100	2.04	0.34	1.70

Based on the above design parameters, runoff from developed conditions will be controlled to or below half of the 2-year, 10-year, 25-year, and 100-year pre-developed release rates. The release rates and detention requirements were generated from the HydroCAD software, which can be seen in Appendix D. Table 4 below summarizes the requirements for the storm events.

Table 5: Detention summary

Storm Event	Adjusted Allowable Release Rate (cfs)	Release Rate (cfs)	Required Detention Volume (ft^3)	Provided Detention Volume (ft^3)
1/2 - 2	0.02	0.02	6887	26857
10	0.69	0.63	16704	26857
25	1.00	0.8	19450	26857
100	1.70	1.66	21859	26857

Flow control is achieved with multiple orifices in a standard City of Salem control structure. The sizing of the orifice uses the standard orifice equation provided in the City of Salem Stormwater Management Manual. Table 5 below identifies orifice size, elevation, and the water surface elevation.

Table 6: Orifice Summary

Storm Event	Control Orifice (#)	Release Rate (cfs)	Orifice Diameter (inches)	Elevation (feet)	W.S. Elevation (feet)
1/2 - 2	1.00	0.02	5/8	599.00	604.8
10	2.00	0.63	4.00	605.00	607.21
25	2.00	0.8	4.00	605.00	607.88
100	Weir Notch	1.66	6.00	607.75	608.41
Overflow	Standpipe			608.5	

The control manhole also contains an overflow standpipe at 608.50. An additional catch basin was added to the detention pond at the same elevation as the stand pipe in the control structure. This overflow is allows for the runoff to leave the site if the outlet in the pond become blogged. In the event the control structure and the overflow catch basin experience a failure, an emergency overflow weir has been incorporated into the pond. The overflow weir that is at an elevation of 609.50.

Stormwater Quality Analysis

Water Quality treatment is being provided by the existing combination facility that was built with the Liberty Road Subdivision. It was previously designed for the Western Liberty Road Apartments to be treated within the facility. Water quality flow rates were calculated using HydroCAD 10.20. The SCS TR-20 Unit Hydrograph method was used to generate the hydrographs. A Type 1A rainfall distribution was used with a 1.38 rainfall depth. Appendix E contains the analysis. Table 6 below identifies the top of media elevation, water surface elevation and overflow elevation for the combination facility.

Table 7: Water Quality Summary

Basin	WQ Flowrate (cfs)	Media Elevation (feet)	W.S. Elevation (feet)	RIM Elevation (feet)
Lots 2-34 and Lot 35	0.75	541.75	542.46	542.50

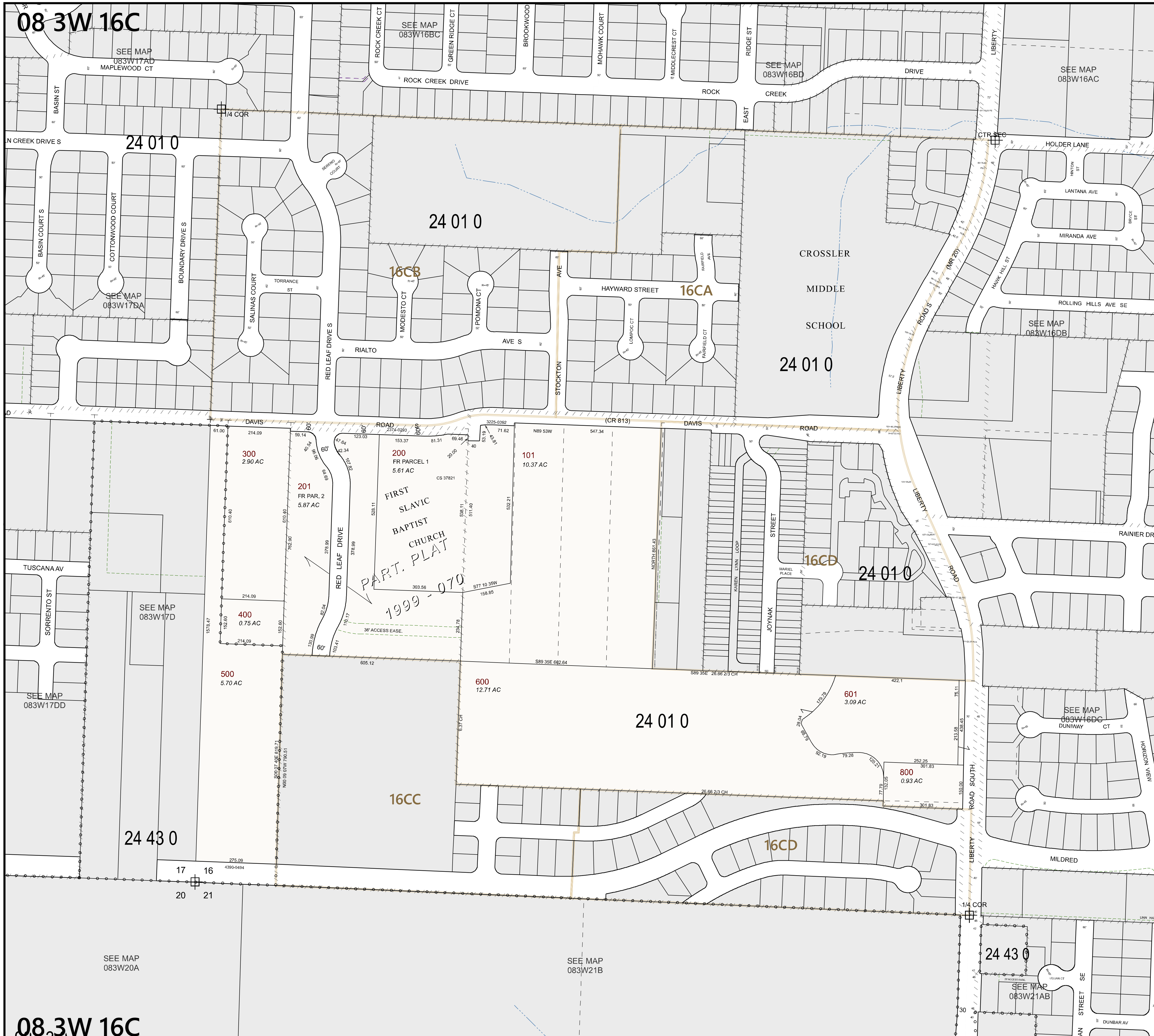
Operation and maintenance

Maintenance of the facility will be the responsibility of the HOA. Prior to permit issuance, a maintenance agreement will be created per City requirements and recorded ensuring proper maintenance.

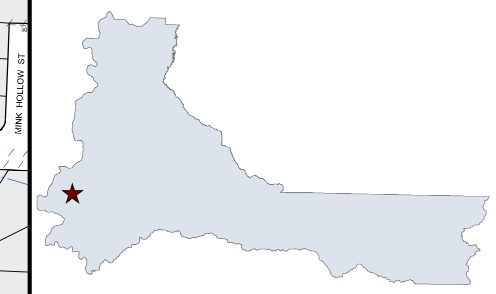
Conclusion

Based on the presented information, the proposed design will meet the City of Salem water quality and quantity standards. If there are any questions regarding this analysis or the design, please contact Natalie Janney at Multi/Tech Engineering by phone at (503) 363-9227 or via e-mail at NJanney@mtengineering.net.

Appendix A



08 3W 16C
SALEM



MARION COUNTY, OREGON
SW1/4 SEC16 T8S R3W W.M.
SCALE 1" = 200'

LEGEND

LINE TYPES

Taxlot Boundary	Historical Boundary
Road Right-of-Way	Easement
Railroad Right-of-Way	Railroad Centerline
Private Road ROW	Taxcode Line
Subdivision/Plat Bndry	Map Boundary
Waterline - Taxlot Bndry	Waterline - Non Bndry

CORNER TYPES

1/16TH Section Cor.	1/4 Section Cor.
DLC Corner	16 15 / 21 22 Section Corner

NUMBERS

Tax Code Number
00 00 0

Acreage 0.25 AC — All acres listed are Net Acres, excluding any portions of the taxlot within public ROWs

NOTES

Tick Marks: A tick mark in the road indicates that the labeled dimension extends into the public ROW

200.00
175.00

CANCELLED NUMBERS

100
700

DISCLAIMER: THIS MAP WAS PREPARED FOR ASSESSMENT PURPOSES ONLY

Marion County OREGON — Assessors Office Cartography Dept

FOR ADDITIONAL MAPS VISIT OUR WEBSITE AT www.co.marion.or.us

PLOT DATE: 5/10/2021

SALEM
08 3W 16C

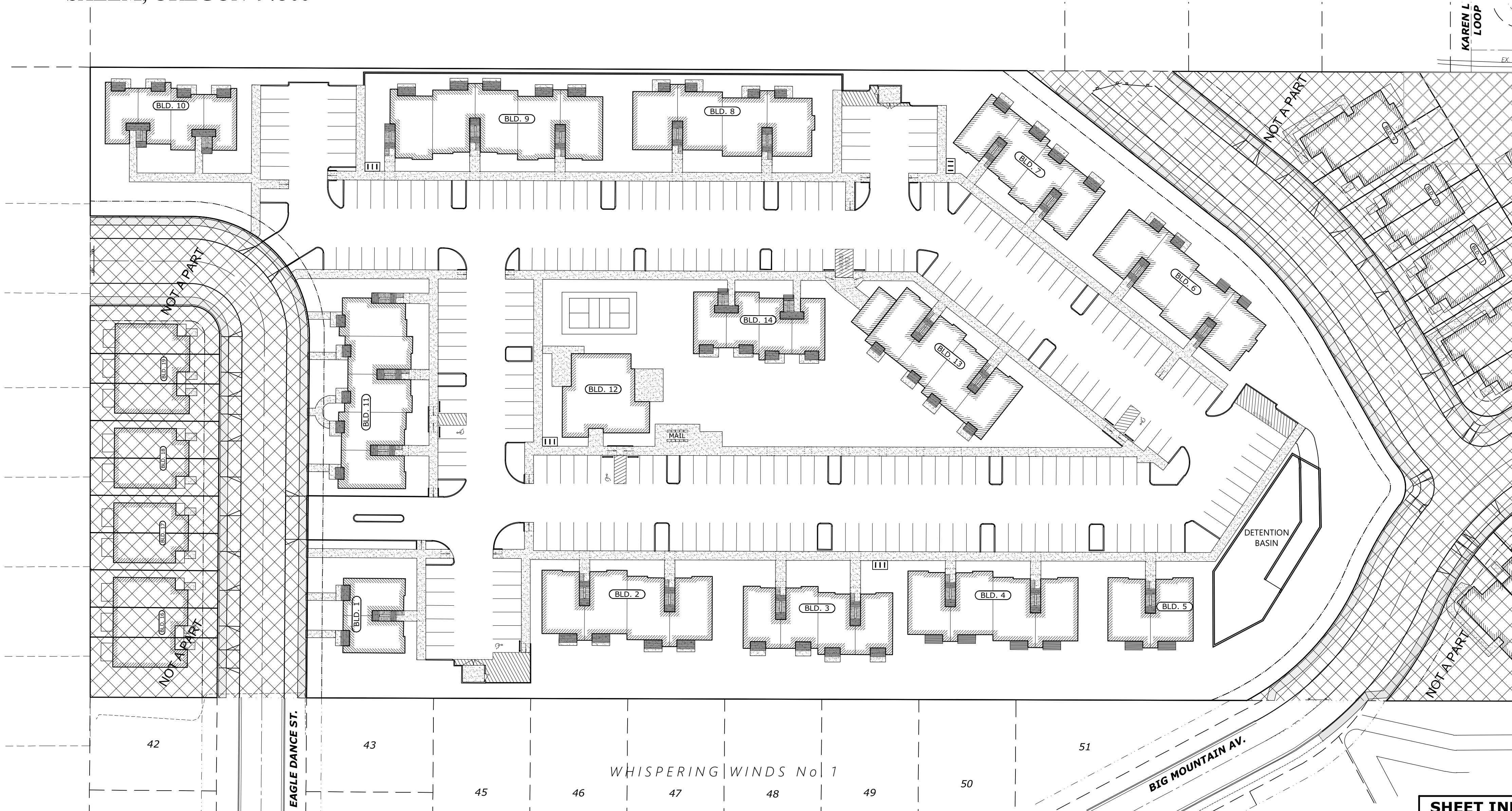
LIBERTY ROAD APARTMENTS WEST

SEC. 16, T. 8 S., R. 3 W., W.M.
CITY OF SALEM
MARION COUNTY, OREGON

Owner / Developer:

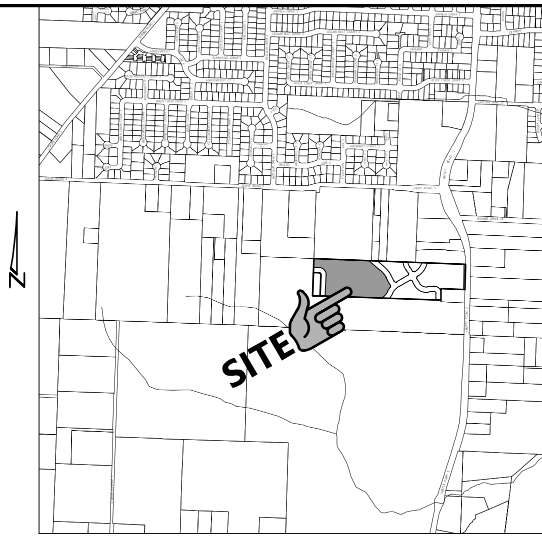
HARRISON INDUSTRIES, L.L.C.
S10355 Liberty Road S.
SALEM, OREGON 97306

N W E S
0' 40'
1 INCH
SCALE: 1" = 40'



BIG MOUNTAIN AV.

40
37



VICINITY MAP

T.B.M. 626.08
EXISTING SANITARY SEWER MANHOLE RIM IN THE INTERSECTION OF EAGLE DANCE ST. S. & BIG MOUNTAIN AV. BASED ON SURVEY CONTROL 8207 - ALUMINUM DISC AT THE NE CORNER OF LIBERTY RD SE AND KUEBLER BLVD. SE, 8.3' N OF M.H., 12.2' SW OF SIGNAL POLE. (DATUM NGVD 1929)

ABBREVIATIONS

A.C.	ASPHALTIC CONCRETE	L.P.	LIGHT POLE
ACMP	ALUMINIZED CMP	M	METER, MAIN
ASSY.	ASSEMBLY	M.H.	MANHOLE
B.O.	BLOW OFF	MTL.	METAL
B.F.V.	BUTTERFLY VALVE	O.H.	OVERHEAD
C & G	CURB & GUTTER	PC	POINT OF CURVE
CATV	CABLE TELEVISION	PCC	POINT OF CONTINUING CURVE
C.B.	CATCH BASIN	PED.	PEDESTAL
C.B.C.O.	CATCH BASIN CLEANOUT	PRC	POINT OF REVERSE CURVE
C.B.I.	CATCH BASIN INLET	PROP.	PROPOSED
C.L.	CENTERLINE	PT	POINT OF TANGENCY
CMP	CORRUGATED METAL PIPE	PUB.	PUBLIC
C.O.	CLEANOUT	PUE	PUBLIC UTILITY EASMT.
CONC.	CONCRETE	PVC	POLYVINYL CHLORIDE
CONST.	CONSTRUCT	PVT.	PRIVATE
D.I.	DUCTILE IRON	P.P.	POWER POLE
DIA.	DIAMETER	P.L.	PROPERTY LINE
DWG	DRAWING	R	RADIUS
EASMT.	EASEMENT	R-	RIM
E.G.	EXIST. GRADE / GROUND	RD	ROOF DRAIN
EOP, E.P.	EDGE OF PAVEMENT	R.O.W.	RIGHT-OF-WAY
ELEC.	ELECTRIC	SAN.S. or S.S.	SANITARY SEWER
ELEV. or EL.	ELEVATION	S	SLOPE
EX. or EXIST.	EXISTING	STA.	STATION
FT.	FEET	STD.	STANDARD
F.F.	FINISH FLOOR	STL.	STEEL
F.G.	FINISH GRADE	STM.DRN. or S.D.	STORM DRAIN
F.H.	FIRE HYDRANT	SVC.	SERVICE
F.M.	FORCE MAIN	SW	SIDEWALK
GUT. or GTR.	GUTTER	T.C.	TOP OF CURB
G.V.	GATE VALVE	TEL.	TELEPHONE
IMP.	IMPROVEMENT	TYP.	TYPICAL
INST.	INSERT	U.G.	UNDERGROUND
INV. or I-	INVERT	VLT.	VAULT
L	LENGTH, LINE	W.M.	WATER MAIN

SYMBOLS

EXIST.	PROP.		EXIST.	PROP.	
⊖	◓	BLOW OFF ASSY.	Ⓢ	Ⓢ	MANHOLE SAN. SEWER
▥	▥	CATCH BASIN	Ⓓ	Ⓓ	MANHOLE STORM DRAIN
⧈	■	CATCH BASIN CLEANOUT	②	②	2' DIA. C.O. / M.H.
⌼	▣	CATCH BASIN INLET	Ⓣ	Ⓣ	MANHOLE TELEPHONE
△	▲	CATV PED. / BOX	Ⓦ	Ⓦ	MANHOLE WATER
○	●	CLEANOUT	▷	▶	REDUCER / INCREASER
⧄	◪	ELEC. PED. / BOX			TEL. PED. / BOX
		FIRE HYDRANT			TRAFFIC PED. / BOX
		GAS LOCATION MARKER			UTILITY / POWER POLE
⋈	⧓	GAS VALVE	□	■	WATER METER
		MAIL BOX	⊗		WATER VALVE

CABLE TELEVISION	SANITARY SEWER EXIST.
CENTERLINE	SANITARY SEWER PROP.
DITCH C.L.	STORM DRAIN EXIST.
ELECTRICAL LINE	STORM DRAIN PROP.
GAS MAIN	WATER MAIN EXIST.
TELEPHONE LINE	WATER MAIN PROP.

SHEET INDEX

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COVER SHEET

LIBERTY ROAD APARTMENTS WEST

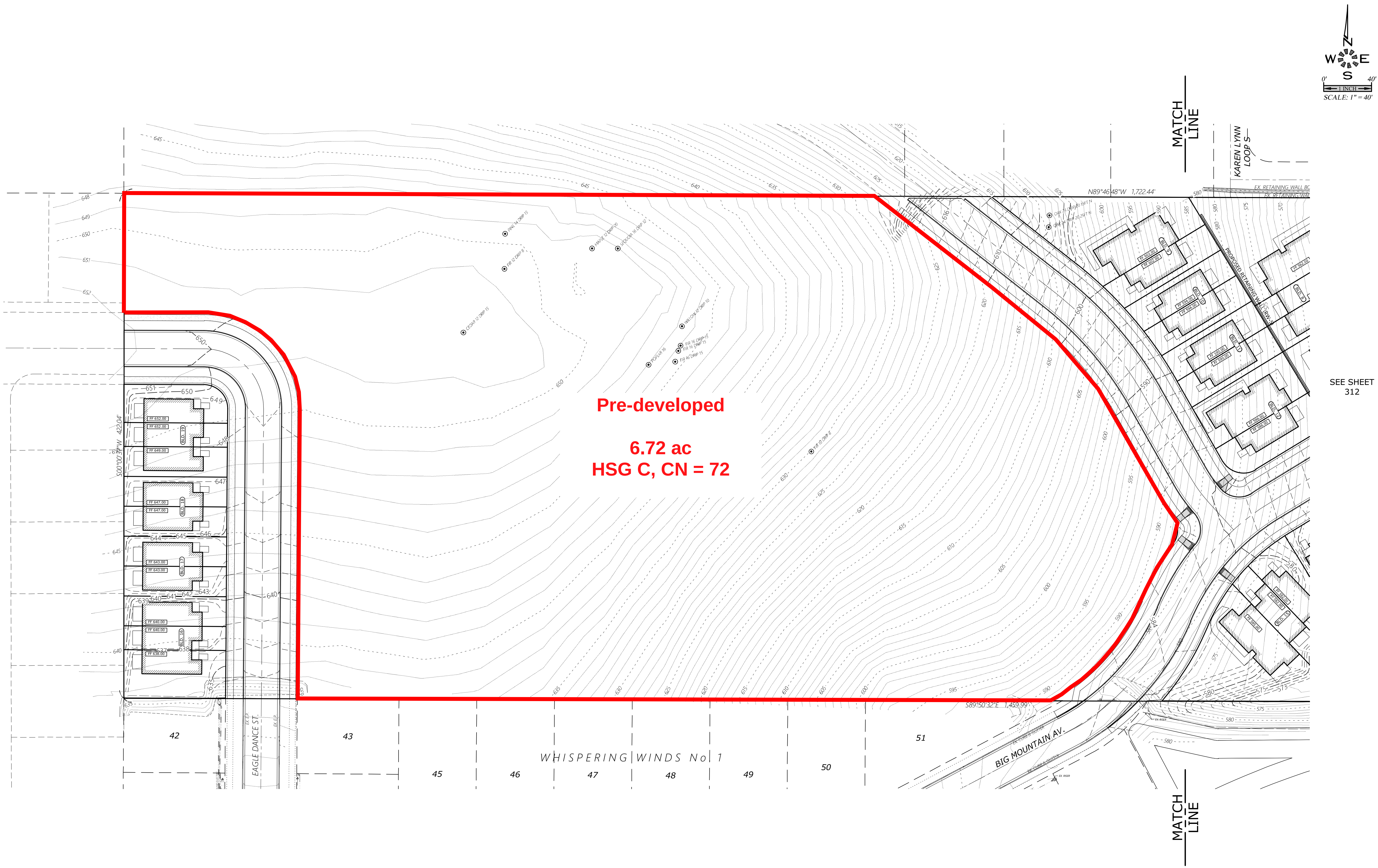
DIMENSIONS & NOTES TAKE PRECEDENCE OVER GRAPHICAL REPRESENTATION.

6727W SDR01CO
Design: M.D.G.
Drawn: P.H.S.
Checked: M.D.G.
Date: JAN. 2020
Scale: AS SHOWN

REGISTERED PROFESSIONAL ENGINEER
OREGON
JULY 14, 1978
MARK D. GRENZ
EXPIRES: 06-30-2023

JOB # 6727

SDR1



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OVERALL GRADING PLAN - WEST -

LIBERTY ROAD

NO CHANGES, MODIFICATIONS OR REPRODUCTIONS TO BE MADE TO THESE DRAWINGS WITHOUT WRITTEN AUTHORIZATION FROM THE DESIGN ENGINEER.

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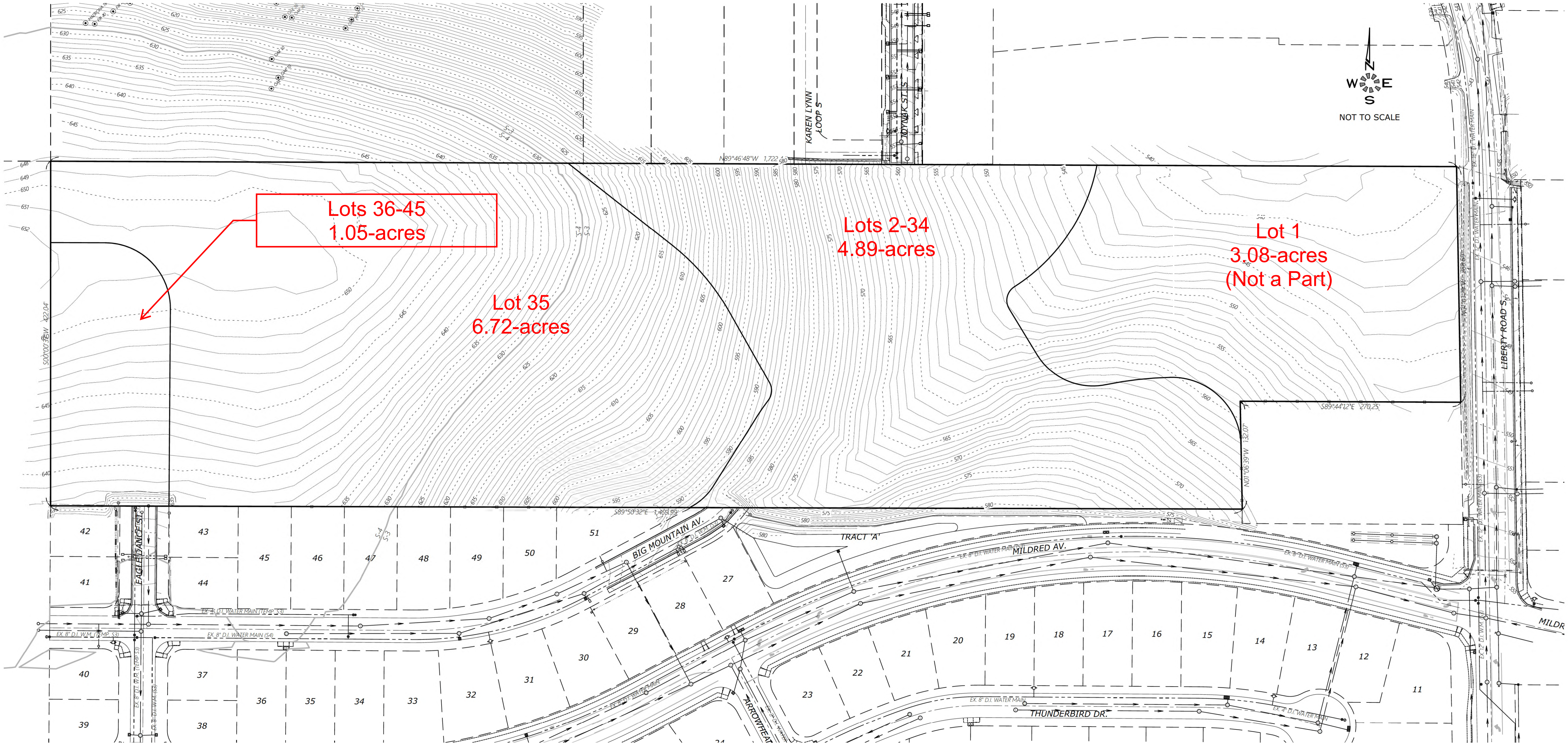
Scale: AS SHOWN
As-Built: ----

REGISTERED PROFESSIONAL ENGINEER
OREGON
JULY 14, 1978
MARK D. GRENZ
EXPIRES: 06-30-2023

JOB # 6727

320



<u>Existing Conditions</u>

City of Salem Pre-developed, HSG C
CN = 72

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EXISTING CONDITIONS PLAN

LIBERTY ROAD SUBDIVISION AND APARTMENTS

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JOB # 6727

COMMON OPEN SPACE

COMMON OPEN SPACE LANDSCAPE

COMMON OPEN SPACE DRIVEWAY & PARKING

COMMON OPEN SPACE WITHIN SETBACK AREA

SITE AREAS

BOUNDARY	**291,276 S.F.**	
TOTAL SITE	**291,276 S.F.**	
R.O.W. DEDICATION	**0 S.F.**	
NET DEVELOPABLE	**291,276 S.F.**	
SITE AREA	**291,276 S.F.**	**(100%)**
COMMON OPEN SPACE (TOTAL)	**142,862 S.F.**	**(49%)**
COMMON OPEN SPACE WITHIN SETBACKS	**41,462 S.F.**	
PRIVATE SPACE (GROUND FLOOR PATIOS)	**4,853 S.F.**	
LANDSCAPE (TOTAL)	**109,624 S.F.**	**(38%)**
PARKING LOT LANDSCAPING	**6,022 S.F.**	

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OPEN SPACE PLAN

LIBERTY ROAD APARTMENTS WEST

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6727W SDR04OS

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Date: JAN. 2020
Scale: AS SHOWN

REGISTERED PROFESSIONAL ENGINEER 9654 OREGON JULY 14, 1978 MARK D. GRENZ

EXPIRES: 06-30-2023

JOB # 6727

SDR4

Basin	Landscaping HSG C (ac) CN = 74	Impervious HSG C (ac) CN = 98	Total (ac)	Composite CN
Site	1.888	4.030	5.918	88
Undetained	0.800	0.000	0.800	74

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GRADING PLAN

LIBERTY ROAD APARTMENTS
WEST

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Design: M.D.G.
Drawn: P.H.S. & M.K.D.
Checked: M.D.G.
Date: ----
Scale: AS SHOWN

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OREGON
JULY 1, 1978
MARK D. GRENZ
EXPIRES: 06-30-2025

JOB # 7586

SDR5

Appendix B

United States Department of Agriculture

NRCS

Natural Resources Conservation Service

A product of the National Cooperative Soil Survey, a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local participants

Custom Soil Resource Report for

Marion County Area, Oregon

Liberty Road Subdivision

August 17, 2020

Hydrologic Soil Group—Marion County Area, Oregon
(Liberty Road Subdivision)

Map Scale: 1:2,910 if printed on A landscape (11" x 8.5") sheet.

Meters: 0, 40, 80, 160, 240

Feet: 0, 100, 200, 400, 600

N

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 17, Jun 11, 2020

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

Date(s) aerial images were photographed: Aug 1, 2018—Aug 31, 2018

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
JoB	Jory silty clay loam, 2 to 7 percent slopes	C	4.5	28.0%
JoC	Jory silty clay loam, 7 to 12 percent slopes	C	0.0	0.3%
JoD	Jory silty clay loam, 12 to 20 percent slopes	C	3.1	19.6%
MaA	McAlpin silty clay loam, 0 to 3 percent slopes	C	0.0	0.1%
NeB	Nekia silty clay loam, 2 to 7 percent slopes	C	8.3	52.0%
Totals for Area of Interest			**15.9**	**100.0%**

Description

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

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

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

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

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

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

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

Rating Options

Aggregation Method: Dominant Condition

Component Percent Cutoff: None Specified

Tie-break Rule: Higher

REDMOND GEOTECHNICAL SERVICES

Geotechnical Investigation

and

Geologic Hazard Assessment Services

Proposed Liberty Road Residential Subdivision Site

Tax Lot No. 600

5871 Liberty Road South

Salem (Marion County), Oregon

for

Multi/Tech Engineering Services, Inc.

Project No. 1001.072.G
November 13, 2020

REDMOND GEOTECHNICAL SERVICES

November 13, 2020

Mr. Mark D. Grenz
Multi/Tech Engineering Services, Inc.
1155 13th Street SE
Salem, Oregon 97302

Dear Mr. Grenz:

Re: Geotechnical Investigation and Geologic Hazard Assessment, Proposed Liberty Road Residential Subdivision Site, Tax Lot No. 600, 5871 Liberty Road South, Salem (Marion County), Oregon

Submitted herewith is our report entitled "Geotechnical Investigation and Geologic Hazard Assessment, Proposed Liberty Road Residential Subdivision Site, Tax Lot No. 600, 5871 Liberty Road South, Salem (Marion County), Oregon". The scope of our services was outlined in our formal discussions with Mr. Mark D. Grenz of Multi/Tech Engineering Services, Inc. on September 23, 2020. Verbal authorization of our services was provided by Mr. Mark D. Grenz of Multi/Tech Engineering Services, Inc. on September 23, 2020.

During the course of our investigation, we have kept you and/or others advised of our schedule and preliminary findings. We appreciate the opportunity to assist you with this phase of the project. Should you have any questions regarding this report, please do not hesitate to call.

Sincerely,

Daniel M. Redmond, P.E., G.E.
President/Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
12,309
OREGON
July 15, 1983
DANIEL M. REDMOND
Exp. 12-31-20

PO Box 20547 • Portland, Oregon 97294 • Fax 503/286-7176 • Phone 503/285-0598

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GEOTECHNICAL INVESTIGATION AND GEOLOGIC HAZARD ASSESSMENT PROPOSED LIBERTY ROAD RESIDENTIAL DEVELOPMENT SITE TAX LOT NO. 600 5871 LIBERTY ROAD SOUTH SALEM (MARION COUNTY) OREGON

INTRODUCTION

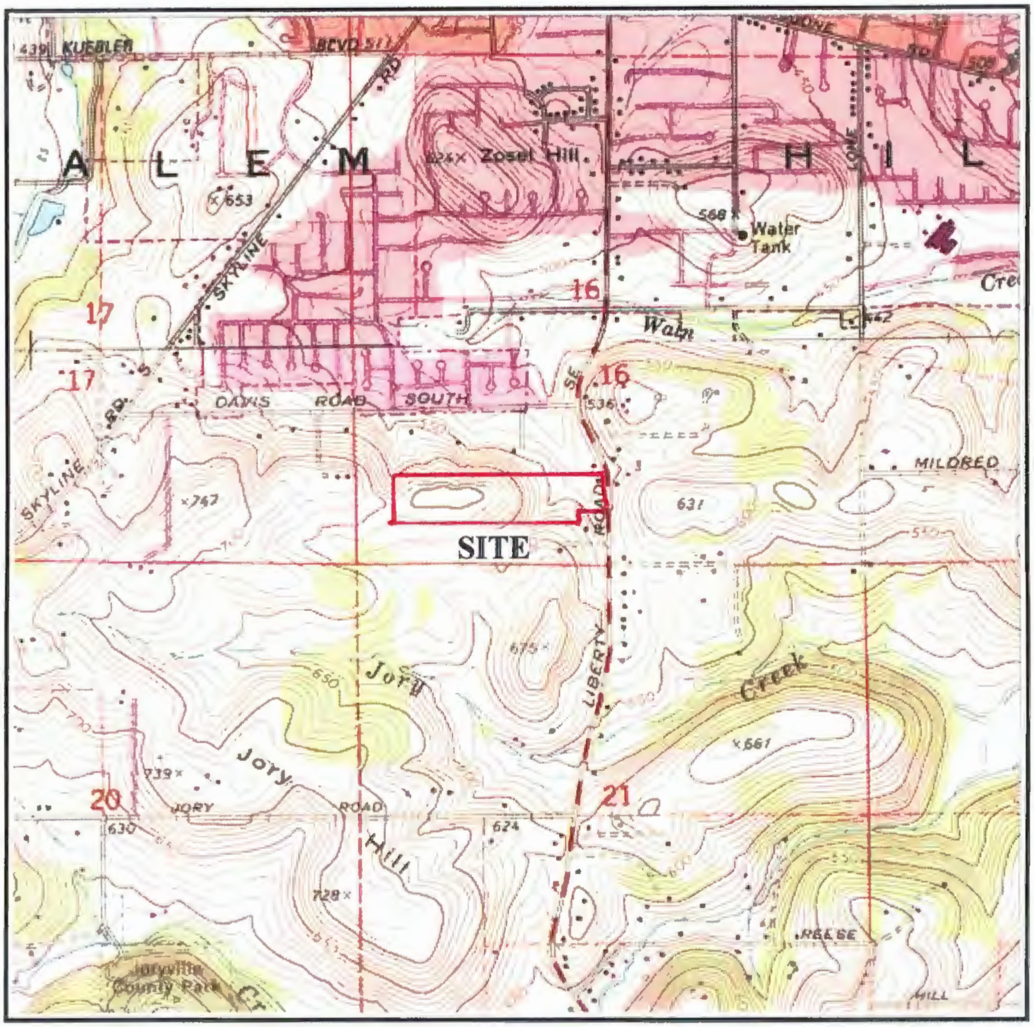
Redmond Geotechnical Services, LLC is please to submit to you the results of our Geotechnical Investigation and Geologic Hazard Assessment at the site of the proposed new residential development located to the west of Liberty Road NW and to the north of the intersection with Mildred Lane SE in Salem (Marion County), Oregon. The general location of the subject site is shown on the Site Vicinity Map, Figure No. 1. The purpose of our geotechnical investigation and geologic hazard study services at this time was to explore the existing subsurface soils and/or groundwater conditions across the subject site and to evaluate any potential concerns with regard to potential slope failure at the site as well as to develop and/or provide appropriate geotechnical design and construction recommendations for the proposed new residential development project.

PROJECT DESCRIPTION

We understand that present plans are to develop the subject property into new single-family residential lots. Based on a review of the proposed site development plan(s) prepared by Multi/Tech Engineering Services, Inc, we understand that the proposed new residential development will consist of the construction of approximately one hundred and twenty-five (125) to one hundred and fifty (150) new single-family residential lots and/or home sites ranging in size from about 6,000 to 10,000 square feet (see Site Exploration Plan, Figure No. 2). The new residential homes are anticipated to be of two- and/or three-story structures constructed with wood framing and raised wooden post and beam floors. However, due to the existing and/or finish grade sloping site conditions, some of the proposed new single-family residential structures and/or lots may also include the construction of a partial below grade floor(s) and/or retaining walls.

Support of the new residential structures is anticipated to consist primarily of conventional shallow strip (continuous) footings although some individual (column) footings may also be required. Structural loading information, although unavailable at this time, is anticipated to be fairly typical and light for this type of wood-frame single-family residential structure and is expected to result in maximum dead plus live continuous (strip) and individual (column) footing loads on the order of about 1.5 to 3.0 kips per lineal foot (klf) and 10 to 25 kips, respectively.

Other associated site improvements for the project will include construction of new public street improvements along the west side of Liberty Road South as well as new local residential streets.



SIDNEY QUADRANGLE
OREGON
7.5-MINUTE SERIES

SCALE 1:24 000

1 0.5 0 KILOMETERS 1 2
1000 500 0 METERS 1000 2000
1 0.5 0 1
MILES
1000 0 1000 2000 3000 4000 5000 6000 7000 8000 9000 10000
FEET

CONTOUR INTERVAL 10 FEET
NORTH AMERICAN VERTICAL DATUM OF 1988

SITE VICINITY MAP		
Project No. 1001.072.G	LIBERTY ROAD SUBDIVISION TL 600, 5871 LIBERTY ROAD SOUTH	Figure No. 1

Additionally, the project will include the construction of new underground utility services as well as new concrete curbs and sidewalks. Further, we anticipate that storm water from hard and/or impervious surfaces (i.e., roofs and pavements) will be collected for on-site treatment and possible disposal.

Although a site grading plan is not available at this time, we understand that both cuts and fills are presently planned for the residential project. In general, cuts and/or fills of at least five (5) feet are generally anticipated across the proposed residential lots as well as the proposed new public streets.

SCOPE OF WORK

The purpose of our geotechnical and/or geologic studies was to evaluate the overall subsurface soil and/or groundwater conditions underlying the subject site with regard to the proposed new residential development and construction at the site and any associated impacts or concerns with respect to potential slope failure at the site as well as provide appropriate geotechnical design and construction recommendations for the project. Specifically, our geotechnical investigation and landslide hazard study performed as a collaboration with Northwest Geological Services, Inc. (NWGS, Inc.) included the following scope of work items:

1. Review of available and relevant geologic and/or geotechnical investigation reports for the subject site and/or area.

2. A detailed field reconnaissance and subsurface exploration program of the soil and ground water conditions underlying the site by means of eleven (11) exploratory test pit excavations. The exploratory test pits were excavated to depths ranging from about six (6) to nine (9) feet beneath existing site grades at the approximate locations as shown on the Site Exploration Plan, Figure No. 2. Additionally, field infiltration testing was also performed within three (3) of the exploratory test holes (TH-#1, TH-#2 and TH-#8) at the time of our field work.

3. Laboratory testing to evaluate and identify pertinent physical and engineering properties of the subsurface soils encountered relative to the planned site development and construction at the site. The laboratory testing program included tests to help evaluate the natural (field) moisture content and dry density, maximum dry density and optimum moisture content, Atterberg Limits and gradational characteristics as well as (remolded) direct shear strength tests as well as "R"-value tests.

4. A literature review and engineering evaluation and assessment of the regional seismicity to evaluate the potential ground motion hazard(s) at the subject site. The evaluation and assessment included a review of the regional earthquake history and sources such as potential seismic sources, maximum credible earthquakes, and reoccurrence intervals as well as a discussion of the possible ground response to the selected design earthquake(s), fault rupture, landsliding, liquefaction, and tsunami and seiche flooding.

5. Engineering analyses utilizing the field and laboratory data as a basis for furnishing recommendations for foundation support of the proposed new residential structures. Recommendations include maximum design allowable contact bearing pressure(s), depth of footing embedment, estimates of foundation settlement, lateral soil resistance, and foundation subgrade preparation. Additionally, construction and/or permanent subsurface water drainage considerations have also been prepared. Further, our report includes recommendations regarding site preparation, placement and compaction of structural fill materials, suitability of the on-site soils for use as structural fill, criteria for import fill materials, and preparation of foundation, pavement and/or floor slab subgrades.

6. Flexible pavement design and construction recommendations for the proposed new public street improvements.

SITE CONDITIONS

Site Geology

The subject site and/or area is underlain by highly weathered Basalt bedrock deposits and/or residual soils of the Columbia River Basalt formation. A more detailed description of the site geology across and/or beneath the site is presented in the Geologic Hazard Study in Appendix B.

Surface Conditions

The subject proposed new residential development property consists of one (1) irregular shaped tax lot (TL 600) which encompass a plan area of approximately 14.8 acres. The proposed residential development property is roughly located to the west of Liberty Road South and to the north of the intersection with Mildred Lane SE. The subject property is generally unimproved and generally void of existing structures and/or site improvements. However, the subject property contains a single-family residential home located across the upper northwesterly portion of the site as well as a detached barn and/or outbuilding across the lower northeasterly portion of the site.

Surface vegetation across the site generally consists of a light to moderate growth of grass, weeds and brush (groundcover) as well as occasional small to large size trees.

Topographically, the site is characterized as gently to moderately sloping terrain (10 to 25 percent) descending downward towards the east with overall topographic relief estimated at about one hundred and fourteen (114) feet and ranges from a low about Elevation 538 feet near the easterly portion of the subject site and/or adjacent to Liberty Road South to a high of about Elevation 652 near the northwesterly site boundary.

Subsurface Soil Conditions

Our understanding of the subsurface soil conditions underlying the site was developed by means of eleven (11) exploratory test pits excavated to depths ranging from about six (6) to nine (9) feet beneath existing site grades on October 15, 2020 with a John Deere 200C track-mounted excavator. The location of the exploratory test pits were located in the field by marking off distances from existing and/or known site features and are shown in relation to the proposed new residential structures and/or site improvements on the Site Exploration Plan, Figure No's. 2-A and 2-B. Detailed logs of the test pit explorations, presenting conditions encountered at each location explored, are presented in the Appendix, Figure No's. A-4 through A-9.

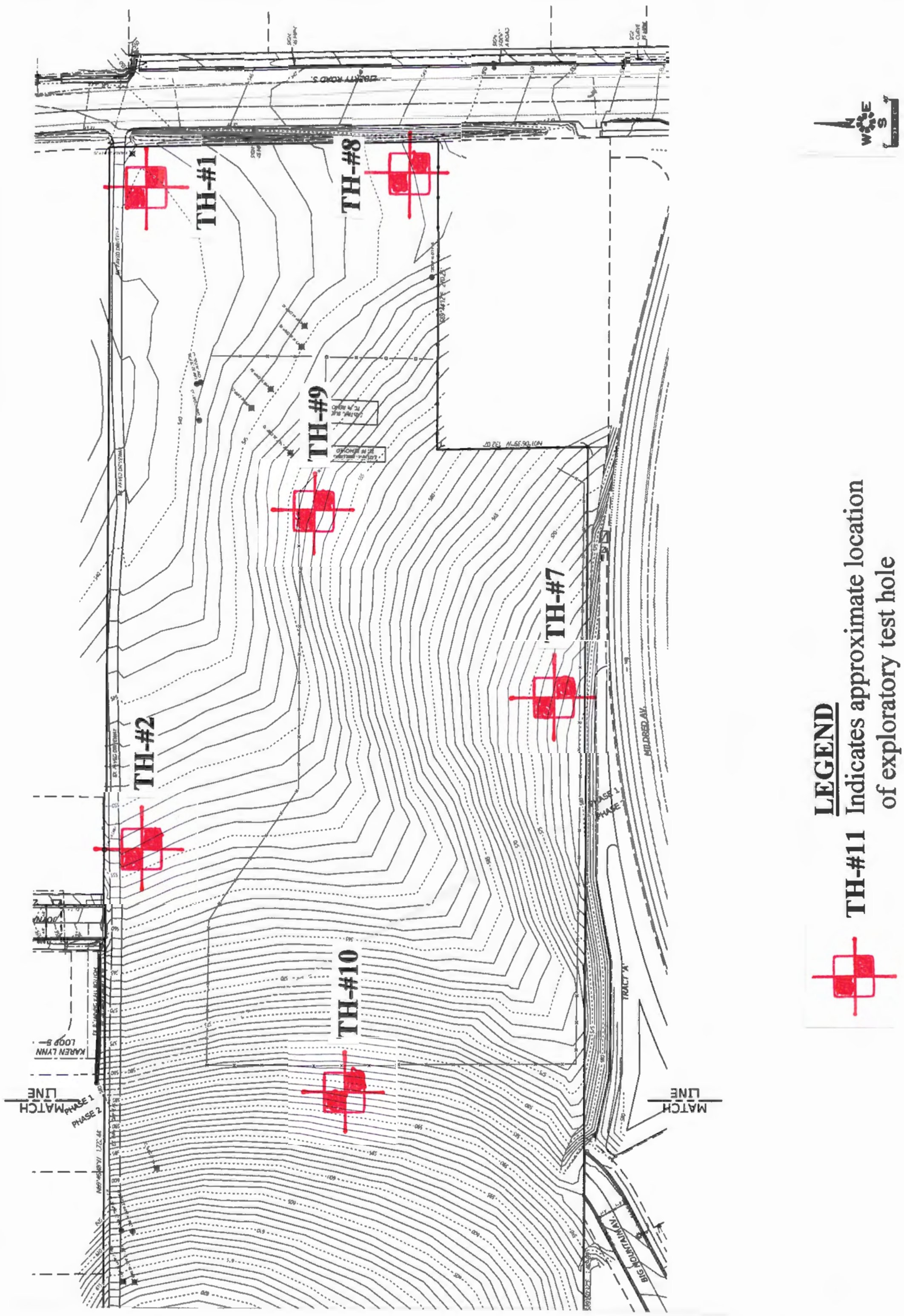
The exploratory test pit excavations were observed by staff from Redmond Geotechnical Services, LLC who logged each of the test pit explorations and obtained representative samples of the subsurface soils encountered across the site. Additionally, the elevation of the exploratory test pit excavations were referenced from the proposed Site Development Plan prepared by Multi/Tech Engineering, Inc. and should be considered as approximate. All subsurface soils encountered at the site and/or within the exploratory test pit excavations were logged and classified in general conformance with the Unified Soil Classification System (USCS) which is outlined on Figure No. A-3.

The test pit explorations revealed that the subject site is underlain by native soil deposits comprised of residual soils composed of a surficial layer of dark brown, very moist to wet, soft, organic, sandy, clayey silt topsoil materials to depths of about 10 to 16 inches. These surficial topsoil materials were inturn underlain by medium to reddish-brown, moist to very moist, medium stiff to stiff, sandy, clayey silt and/or residual soils to the maximum depth explored of about nine (9) feet beneath the existing site and/or surface grades. These sandy, clayey silt and/or residual subgrade soils are best characterized by relatively moderate strength and low to moderate compressibility.

Groundwater

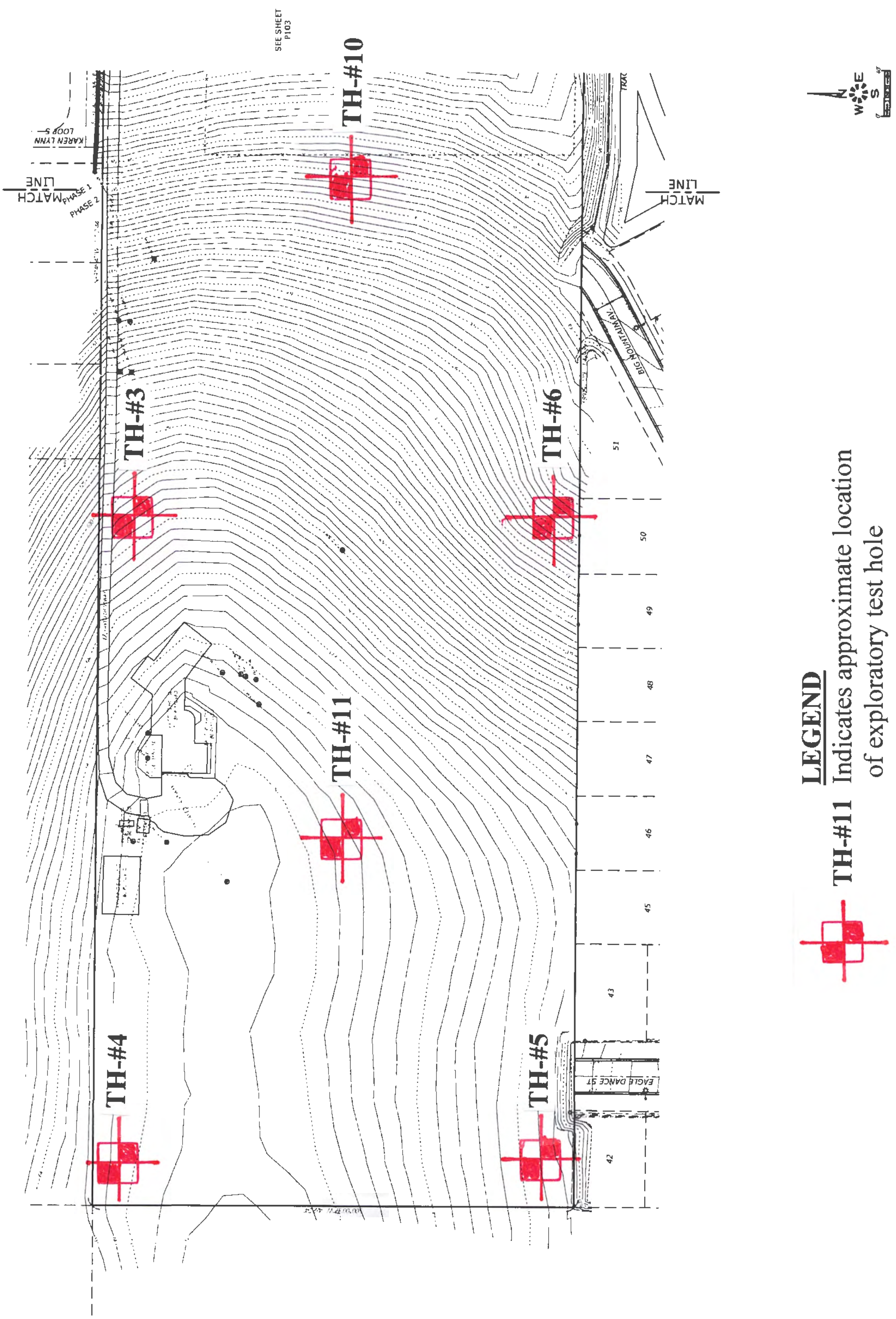
Groundwater was generally not encountered within any of the exploratory test pit explorations (TH-#1 through TH-#11) at the time of excavation to depths of at least nine (9) feet beneath existing surface grades. However, the central and/or easterly portion of the subject property contains an existing seasonal drainage basin.

In this regard, groundwater elevations at the site will likely fluctuate seasonally in accordance with rainfall conditions and/or associated with runoff of the easterly and/or southeasterly drainage basins as well as with changes in site utilization. As such, we are generally of the opinion that the static water levels may approach near surface elevations and may temporarily perch at the ground surface during periods of heavy and/or peak rainfall.



SITE EXPLORATION PLAN

Project No. 1001.072.G	LIBERTY ROAD SUBDIVISION TL 600, 5871 LIBERTY ROAD S	Figure No. 2-A



SITE EXPLORATION PLAN

Project No. 1001.072.G	LIBERTY ROAD SUBDIVISION TL 600, 5871 LIBERTY ROAD S	Figure No. 2-B

INFILTRATION TESTING

We performed three (3) field infiltration tests at the site on October 15, 2020. The infiltration tests were performed in test holes TH-#1, TH-#2 and TH-#8 at depths of about four (4) feet beneath the existing site and/or surface grades. The subgrade soils encountered in the infiltration test hole consisted of sandy, clayey silt and/or residual soils. The infiltration testing was performed in general conformance with current EPA and/or the City of Salem Encased Falling Head test method which consisted of advancing a 6-inch diameter PVC pipe approximately 6 inches into the exposed soil horizon at each test location. Using a steady water flow, water was discharged into the pipe and allowed to penetrate and saturate the subgrade soils. The water level was adjusted over a two (2) hour period and allowed to achieve a saturated subgrade soil condition consistent with the bottom elevation of the surrounding test pit excavation. Following the required saturating period, water was again added into the PVC pipe and the time and/or rate at which the water level dropped was monitored and recorded. Each measurable drop in the water level was recorded until a consistent infiltration rate was observed and/or repeated.

Based on the results of the field infiltration testing at the site (see Figures A-15 through A-17), we have found that the native sandy, clayey silt subgrade soil deposits posses an ultimate infiltration rate on the order of less than 0.2 inches per hour (in/hr).

LABORATORY TESTING

Representative samples of the on-site subsurface soils were collected at selected depths and intervals from various test pit excavations and returned to our laboratory for further examination and testing and/or to aid in the classification of the subsurface soils as well as to help evaluate and identify their engineering strength and compressibility characteristics. The laboratory testing consisted of visual and textural sample inspection, moisture content and dry density determinations, maximum dry density and optimum moisture content, Atterberg Limits and gradation analyses as well as (remolded) direct shear strength and "R"-value tests. Results of the various laboratory tests are presented in the Appendix, Figure No's. A-10 through A-14.

SEISMICITY AND EARTHQUAKE SOURCES

The seismicity of the southwest Washington and northwest Oregon area, and hence the potential for ground shaking, is controlled by three separate fault mechanisms. These include the Cascadia Subduction Zone (CSZ), the mid-depth intraplate zone, and the relatively shallow crustal zone. Descriptions of these potential earthquake sources are presented below.

The CSZ is located offshore and extends from northern California to British Columbia. Within this zone, the oceanic Juan de Fuca Plate is being subducted beneath the continental North American Plate to the east. The interface between these two plates is located at a depth of approximately 15 to 20 kilometers (km).

The seismicity of the CSZ is subject to several uncertainties, including the maximum earthquake magnitude and the recurrence intervals associated with various magnitude earthquakes. Anecdotal evidence of previous CSZ earthquakes has been observed within coastal marshes along the Washington and Oregon coastlines. Sequences of interlayered peat and sands have been interpreted to be the result of large Subduction zone earthquakes occurring at intervals on the order of 300 to 500 years, with the most recent event taking place approximately 300 years ago. A study by Geomatrix (1995) and/or USGS (2008) suggests that the maximum earthquake associated with the CSZ is moment magnitude (Mw) 8 to 9. This is based on an empirical expression relating moment magnitude to the area of fault rupture derived from earthquakes that have occurred within Subduction zones in other parts of the world. An Mw 9 earthquake would involve a rupture of the entire CSZ. As discussed by Geomatrix (1995) this has not occurred in other subduction zones that have exhibited much higher levels of historical seismicity than the CSZ. However, the 2008 USGS report has assigned a probability of 0.67 for a Mw 9 earthquake and a probability of 0.33 for a Mw 8.3 earthquake. For the purpose of this study an earthquake of Mw 9.0 was assumed to occur within the CSZ.

The intraplate zone encompasses the portion of the subducting Juan de Fuca Plate located at a depth of approximately 30 to 50 km below western Washington and western Oregon. Very low levels of seismicity have been observed within the intraplate zone in western Oregon and western Washington. However, much higher levels of seismicity within this zone have been recorded in Washington and California. Several reasons for this seismic quiescence were suggested in the Geomatrix (1995) study and include changes in the direction of Subduction between Oregon, Washington, and British Columbia as well as the effects of volcanic activity along the Cascade Range. Historical activity associated with the intraplate zone includes the 1949 Olympia magnitude 7.1 and the 1965 Puget Sound magnitude 6.5 earthquakes. Based on the data presented within the Geomatrix (1995) report, an earthquake of magnitude 7.25 has been chosen to represent the seismic potential of the intraplate zone.

The third source of seismicity that can result in ground shaking within the Vancouver and southwest Washington area is near-surface crustal earthquakes occurring within the North American Plate. The historical seismicity of crustal earthquakes in this area is higher than the seismicity associated with the CSZ and the intraplate zone. The 1993 Scotts Mills (magnitude 5.6) and Klamath Falls (magnitude 6.0), Oregon earthquakes were crustal earthquakes.

Liquefaction

Seismic induced soil liquefaction is a phenomenon in which lose, granular soils and some silty soils, located below the water table, develop high pore water pressures and lose strength due to ground vibrations induced by earthquakes. Soil liquefaction can result in lateral flow of material into river channels, ground settlements and increased lateral and uplift pressures on underground structures. Buildings supported on soils that have liquefied often settle and tilt and may displace laterally. Soils located above the ground water table cannot liquefy, but granular soils located above the water table may settle during the earthquake shaking.

Our review of the subsurface soil test pit logs from our exploratory field explorations (TH-#1 through TH-#11) and laboratory test results indicate that the site is generally underlain by medium stiff to stiff, sandy, clayey silt and/or residual soils to depths of at least 9.0 feet beneath existing site grades. Additionally, groundwater was generally not encountered within any of the exploratory test pit excavations (TH-#1 through TH-#11) at the site during our field exploration work to depths of at least 9.0 feet. As such, due to the medium stiff to stiff and/or cohesive nature of the sandy, clayey silt subgrade soils beneath the site, it is our opinion that the native sandy, clayey silt subgrade soil deposits located beneath the subject site have a very low potential for liquefaction during the design earthquake motions previously described.

Landslides

No ancient and/or active landslides were observed or are known to be present on the subject site. Additionally, development of the subject site into the planned residential homes sites does not appear to present a potential geologic and/or landslide hazard provided that the site grading and development activities conform with the recommendations presented within this report. A more detailed assessment of the potential landslide hazard of the subject site is presented in the Geologic Hazard Study in Appendix B.

Surface Rupture

Although the site is generally located within a region of the country known for seismic activity, no known faults exist on and/or immediately adjacent to the subject site. As such, the risk of surface rupture due to faulting is considered negligible.

Tsunami and Seiche

A tsunami, or seismic sea wave, is produced when a major fault under the ocean floor moves vertically and shifts the water column above it. A seiche is a periodic oscillation of a body of water resulting in changing water levels, sometimes caused by an earthquake. Tsunami and seiche are not considered a potential hazard at this site because the site is not near to the coast and/or there are no adjacent significant bodies of water.

Flooding and Erosion

Stream flooding is a potential hazard that should be considered in lowland areas of Marion County and Salem. The FEMA (Federal Emergency Management Agency) flood maps should be reviewed as part of the design for the proposed new residential structures and site improvements. Elevations of structures on the site should be designed based upon consultants reports, FEMA (Federal Emergency Management Agency), and Marion County requirements for the 100-year flood levels of any nearby creeks, streams and/or drainage basins.

CONCLUSIONS AND RECOMMENDATIONS

General

Based on the results of our field explorations, laboratory testing, and engineering analyses, it is our opinion that the site is presently stable and suitable for the proposed new Liberty Road Subdivision single-family residential development and its associated site improvements provided that the recommendations contained within this report are properly incorporated into the design and construction of the project.

The primary features of concern at the site are 1) the presence of highly moisture sensitive sandy, clayey silt subgrade soils across the site, 2) the presence of moderately sloping site conditions across the proposed new residential lots and/or home sites, and 3) the relatively low infiltration rates anticipated within the near surface sandy, clayey silt subgrade soils.

With regard to the moisture sensitive sandy, clayey silt subgrade soils, we are generally of the opinion that all site grading and earthwork activities be scheduled for the drier summer months which is typically June through September. In regards to the moderately sloping site conditions across the proposed new residential home sites and/or lots, we are of the opinion that site grading and/or structural fill placement should be minimized where possible and should generally limit cuts and/or fills to about five (5) feet or less unless approved by the Geotechnical Engineer. Additionally, where existing site slopes and/or surface grades exceed about 20 percent (1V:5H) and in order to construct the proposed improvements to Liberty Road South and/or a new local residential streets, benching and keying of all fills into the natural site slopes may be required. With regard to the relatively low infiltration rates anticipated within the sandy, clayey silt subgrade soils beneath the site, we generally do not recommend any storm water infiltration within structural and/or embankment fills. However, some limited storm water infiltration may be feasible within the lower easterly portion of the site and/or along Liberty Road South where the existing and/or finish slope gradients are no steeper than about 20 percent (1V:5H). In this regard, we recommend that all proposed storm water detention and/or infiltration systems for the project be reviewed and approved by Redmond Geotechnical Services, LLC.

The following sections of this report provide specific recommendations regarding subgrade preparation and grading as well as foundation and floor slab design and construction for the new Liberty Road Subdivision residential development project.

Site Preparation

As an initial step in site preparation, we recommend that the proposed new residential building sites and/or lots as well as their associated structural and/or site improvement area(s) be stripped and cleared of all existing improvements, any existing unsuitable fill materials, surface debris, existing vegetation, topsoil materials, and/or any other deleterious materials present at the time of construction. In general, we envision that the site stripping to remove existing vegetation and topsoil materials will generally be about 10 to 16 inches.

However, localized areas requiring deeper removals, such as any existing undocumented and/or unsuitable fill materials as well as old foundation remnants, will likely be encountered and should be evaluated at the time of construction by the Geotechnical Engineer. The stripped and cleared materials should be properly disposed of as they are generally considered unsuitable for use/reuse as fill materials.

Following the completion of the site stripping and clearing work and prior to the placement of any required structural fill materials and/or structural improvements, the exposed subgrade soils within the planned structural improvement area(s) should be inspected and approved by the Geotechnical Engineer and possibly proof-rolled with a half and/or fully loaded dump truck. Areas found to be soft or otherwise unsuitable should be over-excavated and removed or scarified and recompacted as structural fill. During wet and/or inclement weather conditions, proof rolling and/or scarification and recompaction as noted above may not be appropriate.

The on-site native clayey, sandy silt subgrade soil materials are generally considered suitable for use/reuse as structural fill materials provided that they are free of organic materials, debris, and rock fragments in excess of about 6 inches in dimension. However, if site grading is performed during wet or inclement weather conditions, the use of some of the on-site native soil materials which contain significant silt and clay sized particles will be difficult at best. In this regard, during wet or inclement weather conditions, we recommend that an import structural fill material be utilized which should consist of a free-draining (clean) granular fill (sand & gravel) containing no more than about 5 percent fines. Representative samples of the materials which are to be used as structural fill materials should be submitted to the Geotechnical Engineer and/or laboratory for approval and determination of the maximum dry density and optimum moisture content for compaction.

In general, all site earthwork and grading activities should be scheduled for the drier summer months (June through September) if possible. However, if wet weather site preparation and grading is required, it is generally recommended that the stripping of topsoil materials be accomplished with a tracked excavator utilizing a large smooth-toothed bucket working from areas yet to be excavated. Additionally, the loading of strippings into trucks and/or protection of moisture sensitive subgrade soils will also be required during wet weather grading and construction. In this regard, we recommend that areas in which construction equipment will be traveling be protected by covering the exposed subgrade soils with a woven geotextile fabric such as Mirafi FW404 followed by at least 12 inches or more of crushed aggregate base rock. Further, the geotextile fabric should have a minimum Mullen burst strength of at least 250 pounds per square inch for puncture resistance and an apparent opening size (AOS) between the U.S. Standard No. 70 and No. 100 sieves.

All structural fill materials placed within the new building and/or pavement areas should be moistened or dried as necessary to near (within 3 percent) optimum moisture conditions and compacted by mechanical means to a minimum of 92 percent of the maximum dry density as determined by the ASTM D-1557 (AASHTO T-180) test procedures. Structural fill materials should be placed in lifts (layers) such that when compacted do not exceed about 8 inches.

Additionally, all fill materials placed within five (5) lineal feet of the perimeter (limits) of the proposed residential structures and/or pavements should be considered structural fill. Additionally, due to the sloping site conditions, we recommend that all structural fill materials planned in areas where existing surface and/or slope gradients exceed about 20 percent (1V:5H) be properly benched and/or keyed into the native (natural) slope subgrade soils. In general, a bench width of at least eight (8) feet and a keyway depth of at least one (1) foot is recommended. A typical key and bench fill slope detail is presented on Figure No. 3. However, the actual bench width and keyway depth should be determined at the time of construction by the Geotechnical Engineer. Further, all fill slopes should be constructed with a finish slope surface gradient no steeper than about 2H:1V. Estimated post construction settlements within and/or beneath the proposed public road embankment fills for the new public streets are expected to be between 1/4-inch and 1/2-inch and should occur within four (4) to five (5) weeks following construction of the embankment fills.

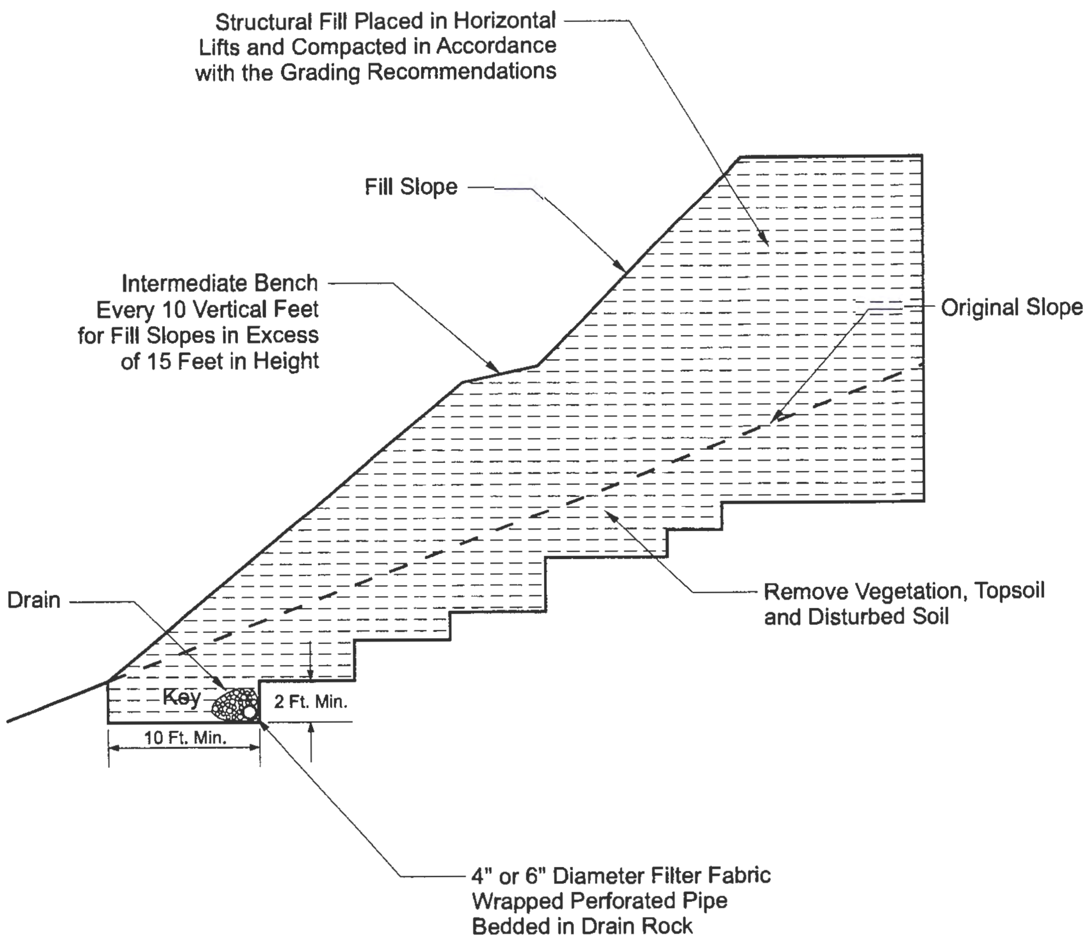
As such, settlement sensitive site and/or surface improvements (i.e., concrete curbs and sidewalks) should not be constructed until after primary consolidation and/or settlement has been completed. All aspects of the site grading, including a review of the proposed site grading plan(s), should be approved and/or monitored by a representative of Redmond Geotechnical Services, LLC.

Foundation Support

Based on the results of our investigation, it is our opinion that the site of the proposed new Liberty Road residential development is suitable for support of the two- and/or three-story wood-frame structures provided that the following foundation design recommendations are followed. The following sections of this report present specific foundation design and construction recommendations for the planned new residential structures.

Shallow Foundations

In general, conventional shallow continuous (strip) footings and individual (spread) column footings may be supported by approved and firm native (untreated) subgrade soil materials and/or silty structural fill soils based on an allowable contact bearing pressure of about 2,000 pounds per square foot (psf). However, where higher allowable contact bearing pressures are required and/or desired, an allowable contact bearing pressure of 2,500 psf may be used for design where larger (i.e., 3-feet or more) retaining wall footings are supported by approved native subgrade soils or at least 6 inches or more of granular (crushed rock) structural fill. These recommended allowable contact bearing pressures are intended for dead loads and sustained live loads and may be increased by one-third for the total of all loads including short-term wind or seismic loads. In general, continuous strip footings should have a minimum width of at least 16 inches and be embedded at least 18 inches below the lowest adjacent finish grade (includes frost protection). Individual column footings (where required) should be embedded at least 18 inches below grade and have a minimum width of at least 24 inches. Additionally, if foundation excavation and construction work is planned to be performed during wet and/or inclement weather conditions, we recommend that a 3 to 4 inch layer of compacted crushed rock be used to help protect the exposed foundation bearing surfaces until the placement of concrete.



TYPICAL KEY AND BENCH FILL SLOPE DETAIL

Project No. 1001.072.G	LIBERTY ROAD SUBDIVISION TL 600, 5871 LIBERTY ROAD S	Figure No. 3

Total and differential settlements of foundations constructed as recommended above and supported by approved native subgrade soils or by properly compacted structural fill materials are expected to be well within the tolerable limits for this type of lightly loaded wood-frame structure and should generally be less than about 1-inch and 1/2-inch, respectively.

Allowable lateral frictional resistance between the base of the footing element and the supporting subgrade bearing soil can be expressed as the applied vertical load multiplied by a coefficient of friction of 0.30 and 0.45 for native silty subgrade soils and/or import gravel fill materials, respectively. In addition, lateral loads may be resisted by passive earth pressures on footings poured "neat" against in-situ (native) subgrade soils or properly backfilled with structural fill materials based on an equivalent fluid density of 350 pounds per cubic foot (pcf). This recommended value includes a factor of safety of approximately 1.5 which is appropriate due to the amount of movement required to develop full passive resistance.

Floor Slab Support

In order to provide uniform subgrade reaction beneath concrete slab-on-grade floors, we recommend that the floor slab area be underlain by a minimum of 6 inches of free-draining (less than 5 percent passing the No. 200 sieve), well-graded, crushed rock. The crushed rock should help provide a capillary break to prevent migration of moisture through the slab. However, additional moisture protection can be provided by using a 10-mil polyolefin geo-membrane sheet such as StegoWrap.

The base course materials should be compacted to at least 95 percent of the maximum dry density as determined by the ASTM D-1557 (AASHTO T-180) test procedures. Where floor slab subgrade materials are undisturbed, firm and stable and where the underslab aggregate base rock section has been prepared and compacted as recommended above, we recommend that a modulus of subgrade reaction of 150 pci be used for design.

Retaining/Below Grade Walls

Retaining and/or below grade walls should be designed to resist lateral earth pressures imposed by native soils or granular backfill materials as well as any adjacent surcharge loads. For walls which are unrestrained at the top and free to rotate about their base, we recommend that active earth pressures be computed on the basis of the following equivalent fluid densities:

Non-Restrained Retaining Wall Pressure Design Recommendations

Slope Backfill (Horizontal/Vertical)	Equivalent Fluid Density/Silt (pcf)	Equivalent Fluid Density/Gravel (pcf)
Level	35	30
3H:1V	60	50
2H:1V	90	80

For walls which are fully restrained at the top and prevented from rotation about their base, we recommend that at-rest earth pressures be computed on the basis of the following equivalent fluid densities:

Restrained Retaining Wall Pressure Design Recommendations

Slope Backfill (Horizontal/Vertical)	Equivalent Fluid Density/Silt (pcf)	Equivalent Fluid Density/Gravel (pcf)
Level	45	35
3H:1V	65	60
2H:1V	95	90

The above recommended values assume that the walls will be adequately drained to prevent the buildup of hydrostatic pressures. Where wall drainage will not be present and/or if adjacent surcharge loading is present, the above recommended values will be significantly higher. For seismic loading, we recommend an additional uniform pressure of about 6H where H is the height of the wall in feet.

Backfill materials behind walls should be compacted to 90 percent of the maximum dry density as determined by the ASTM D-1557 (AASHTO T-180) test procedures. Special care should be taken to avoid over-compaction near the walls which could result in higher lateral earth pressures than those indicated herein. In areas within three (3) to five (5) feet behind walls, we recommend the use of hand-operated compaction equipment.

Pavements

Flexible pavement design for the proposed street improvements along the east side of Liberty Road South as well as the proposed new street improvements for the residential development project was determined in accordance with the City of Salem Department of Public Works Administrative Rules Chapter 109-006 (Street Design Standards) Section 6 dated January 1, 2014.

Specifically, on October 15, 2020, samples of the subgrade soils from the existing and/or proposed public streets were collected by means of test hole excavations. The subgrade soils encountered in the test holes located across the proposed residential subdivision site as well as along the west side of Liberty Road South generally consisted of native and/or residual soils comprised of medium to reddish-brown, medium stiff to stiff, sandy, clayey silt (ML).

The subgrade soil samples collected at the site were tested in the laboratory in accordance with the ASTM Vol. 4.08 Part D-2844-69 (AASHTO T-190-93) test method for the determination of the subgrade soil "R"-value and expansion pressure.

The results of the "R"-value testing was then converted to an equivalent Resilient Modulus (M_{RSG}) in accordance with current AASHTO methodology. The results of the laboratory "R"-value tests revealed that the subgrade soils have an apparent "R"-value of between 24 and 26 with an average "R"-value of 25 (see Figure No. A-14). Using the current AASHTO methodology for converting "R"-value to Resilient Modulus (M_{RSG}), the subgrade soils have a Resilient Modulus (M_{RSG}) of about 5,112 psi which is classified a "Fair" (M_{RSG} = 5,000 psi to 10,000 psi).

In addition to the above, Dynamic Cone Penetration (DCP) tests were performed along the proposed new interior public street alignment at approximate 100-feet intervals. The results of the DCP tests found that the underlying native clayey, sandy silt subgrade soils have a DCP value of between 2 to 3 blows per 2-inches which correlates to a California Bearing Ratio (CBR) of between 5 and 12. Using current AASHTO methodology for converting CBR to Resilient Modulus (M_{RSG}), the subgrade soils have a Resilient Modulus (M_{RSG}) of between 5,842 and 10,637 psi with an average M_{RSG} of 7,150 psi which is classified as "Fair" (M_{RSG} = 5,000 psi to 10,000 psi).

Liberty Road South

The following documents and/or design input parameters were used to help determine the flexible pavement section design for improvements to Liberty Road South:

- **Street Classification:** Major Arterial
- **Design Life:** 20 years
- **Serviceability:** 4.2 initial, 2.5 terminal
- **Traffic Loading Data:** 10,000,000 18-kip EAL's
- **Reliability Level:** 90%
- **Drainage Coefficient:** 1.0 (asphalt), 0.8 (aggregate)
- **Asphalt Structural Coefficient:** 0.41
- **Aggregate Structural Coefficient:** 0.10

Based on the above design input parameters and using the design procedures contained within the AASHTO 1993 Design of Pavement Structures Manual, a Structural Number (SN) of 4.4 was determined.

In this regard, we recommend the following flexible pavement section for the new improvements to Liberty Road South:

Material Type	Pavement Section (inches)
Asphaltic Concrete	7.0
Aggregate Base Rock	16.0

Local Residential Streets

The following documents and/or design input parameters were used to help determine the flexible pavement section design for new local residential streets:

- **Street Classification:** Local Residential Street
- **Design Life:** 25 years
- **Serviceability:** 4.2 initial, 2.5 terminal
- **Traffic Loading Data:** 100,000 18-kip EAL's
- **Reliability Level:** 90%
- **Drainage Coefficient:** 1.0 (asphalt), 0.8 (aggregate)
- **Asphalt Structural Coefficient:** 0.41
- **Aggregate Structural Coefficient:** 0.10

Based on the above design input parameters and using the design procedures contained within the AASHTO 1993 Design of Pavement Structures Manual, a Structural Number (SN) of 2.6 was determined.

In this regard, we recommend the following flexible pavement section for the construction of new Local Residential Streets:

Material Type	Pavement Section (inches)
Asphaltic Concrete	4.0
Aggregate Base Rock	10.0

Pavement Subgrade, Base Course & Asphalt Materials

The above recommended pavement section(s) were based on the design assumptions listed herein and on the assumption that construction of the pavement section(s) will be completed during an extended period of reasonably dry weather. All thicknesses given are intended to be the minimum acceptable. Increased base rock sections and the use of a woven geotextile fabric may be required during wet and/or inclement weather conditions and/or in order to adequately support construction traffic and protect the subgrade during construction. Additionally, the above recommended pavement section(s) assume that the subgrade will be prepared as recommended herein, that the exposed subgrade soils will be properly protected from rain and construction traffic, and that the subgrade is firm and unyielding at the time of paving. Further, it assumes that the subgrade is graded to prevent any ponding of water which may tend to accumulate in the base course.

Pavement base course materials should consist of well-graded 1-1/2 inch and/or 3/4-inch minus crushed base rock having less than 5 percent fine materials passing the No. 200 sieve. The base course and asphaltic concrete materials should conform to the requirements set forth in the latest edition of the Oregon Department of Transportation, Standard Specifications for Highway Construction. The base course materials should be compacted to at least 95 percent of the maximum dry density as determined by the ASTM D-1557 (AASHTO T-180) test procedures. The asphaltic concrete paving materials should be compacted to at least 92 percent of the theoretical maximum density as determined by the ASTM D-2041 (Rice Gravity) test method.

Wet Weather Grading and Soft Spot Mitigation

Construction of the proposed new public street improvements is generally recommended during dry weather. However, during wet weather grading and construction, excavation to subgrade can proceed during periods of light to moderate rainfall provided that the subgrade remains covered with aggregate. A total aggregate thickness of 8-inches may be necessary to protect the subgrade soils from heavy construction traffic. Construction traffic should not be allowed directly on the exposed subgrade but only atop a sufficient compacted base rock thickness to help mitigate subgrade pumping. If the subgrade becomes wet and pumps, no construction traffic shall be allowed on the road alignment. Positive site drainage away from the street shall be maintained if site paving will not occur before the on-set of the wet season.

Depending on the timing for the project, any soft subgrade found during proof-rolling or by visual observations can either be removed and replaced with properly dried and compacted fill soils or removed and replaced with compacted crushed aggregate. However, and where approved by the Geotechnical Engineer, the soft area may be covered with a bi-axial geogrid and covered with compacted crushed aggregate.

Soil Shrink-Swell and Frost Heave

The results of the laboratory "R"-value tests indicate that the native subgrade soils possess a low to moderate expansion potential. As such, the exposed subgrade soils should not be allowed to completely dry and should be moistened to near optimum moisture content (plus or minus 3 percent) at the time of the placement of the crushed aggregate base rock materials. Additionally, exposure of the subgrade soils to freezing weather may result in frost heave and softening of the subgrade. As such, all subgrade soils exposed to freezing weather should be evaluated and approved by the Geotechnical Engineer prior to the placement of the crushed aggregate base rock materials.

Excavation/Slopes

Temporary excavations of up to about four (4) feet in depth may be constructed with near vertical inclinations. Temporary excavations greater than about four (4) feet but less than eight (8) feet should be excavated with inclinations of at least 1 to 1 (horizontal to vertical) or properly braced/shored. Where excavations are planned to exceed about eight (8) feet, this office should be consulted. Permanent slopes should be constructed no steeper than 2H to 1V.

All shoring systems and/or temporary excavation bracing for the project should be the responsibility of the excavation contractor. Permanent slopes should be constructed no steeper than about 2H to 1V unless approved by the Geotechnical Engineer.

Depending on the time of year in which trench excavations occur, trench dewatering may be required in order to maintain dry working conditions if the invert elevations of the proposed utilities are located at and/or below the groundwater level. If groundwater is encountered during utility excavation work, we recommend placing trench stabilization materials along the base of the excavation.

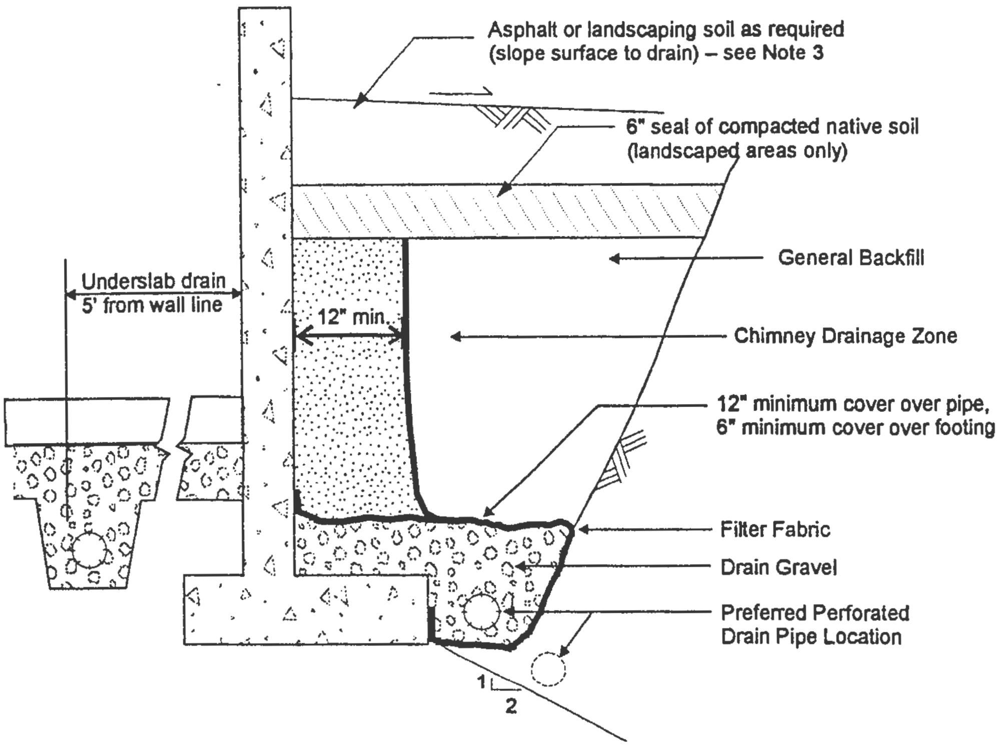
Trench stabilization materials should consist of 1-foot of well-graded gravel, crushed gravel, or crushed rock with a maximum particle size of 4 inches and less than 5 percent fines passing the No. 200 sieve. The material should be free of organic matter and other deleterious material and placed in a single lift and compacted until well keyed.

Surface Drainage/Groundwater

We recommend that positive measures be taken to properly finish grade the site so that drainage waters from the residential structures and landscaping areas as well as adjacent properties or buildings are directed away from the new residential structures foundations and/or floor slabs. All roof drainage should be directed into conduits that carry runoff water away from the residential structures to a suitable outfall. Roof downspouts should not be connected to foundation drains. A minimum ground slope of about 2 percent is generally recommended in unpaved areas around the proposed new residential structures.

Groundwater was not encountered at the site in any of the exploratory test pits (TH-#1 through TH-#11) at the time of excavation to depths of at least 9 feet beneath existing site grades. Additionally, surface water ponding was not observed at the site during our field exploration work. However, the central and/or easterly portion of the site contains an existing seasonal drainage basin feature. Further, groundwater elevations in the area and/or across the subject property may fluctuate seasonally and may temporarily pond/perch near the ground surface during periods of prolonged rainfall.

As such, based on our current understand of the possible site grading required to bring the subject site and/or residential lots to finish design grade(s), we are of the opinion that an underslab drainage system is not required for the proposed single-family residential structures. However, a perimeter foundation drain is recommended for any perimeter footings and/or below grade retaining walls. A typical recommended perimeter footing/retaining wall drain detail is shown on Figure No. 4. Further, due to our understanding that various surface infiltration ditches and/or swales may be utilized for the project as well as the relatively low infiltration rates of the near surface sandy, clayey silt and/or residual soils anticipated within and/or near to the foundation bearing level of the proposed residential structures, we are generally of the opinion that storm water detention and/or disposal systems should not be utilized within the residential lots and/or around the proposed residential structures unless approved by the Geotechnical Engineer.



SCHEMATIC - NOT TO SCALE

NOTES:

1. Filter Fabric to be non-woven geotextile (Amoco 4545, Mirafi 140N, or equivalent)
2. Lay perforated drain pipe on minimum 0.5% gradient, widening excavation as required. Maintain pipe above 2:1 slope, as shown.
3. All-granular backfill is recommended for support of slabs, pavements, etc. (see text for structural fill).
4. Drain gravel to be clean, washed ¾" to 1½" gravel.
5. General backfill to be on-site gravels, or ¾""-0 or 1½"-0 crushed rock compacted to 92% Modified Proctor (AASHTO T-180).
6. Chimney drainage zone to be 12" wide (minimum) zone of clean washed, medium to coarse sand or drain gravel if protected with filter fabric. Alternatively, prefabricated drainage structures (Miradrain 6000 or similar) may be used.

TYPICAL PERIMETER FOOTING/RETAINING WALL DRAIN DETAIL		
Project No. 1001.072.G	LIBERTY ROAD SUBDIVISION TL 600, 5871 LIBERTY ROAD S	Figure No. 4

Design Infiltration Rates

Based on the results of our field infiltration testing, we recommend using the following infiltration rate to design any on-site near surface storm water infiltration and/or disposal systems for the project:

Subgrade Soil Type	Recommended Infiltration Rate
sandy, clayey SILT (ML)	less than 0.1 inches per hour (in/hr)

Note: A safety factor of two (2) was used to calculate the above recommended design infiltration rate. Additionally, given the gradational variability of the on-site sandy, clayey silt subgrade soils beneath the site as well as the anticipation of some site grading for the project, it is generally recommended that field testing be performed during and/or following construction of any on-site storm water infiltration system(s) in order to confirm that the above recommended design infiltration rates are appropriate.

Seismic Design Considerations

Structures at the site should be designed to resist earthquake loading in accordance with the methodology described in the 2019 and/or latest edition of the State of Oregon Structural Specialty Code (OSSC), ASCE 7-16 and/or Amendments to the 2018 International Building Code (IBC). The maximum considered earthquake ground motion for short period and 1.0 period spectral response may be determined from the Oregon Structural Specialty Code (OSSC)and/or from the 2015 National Earthquake Hazard Reduction Program (NEHRP) "Recommended Provisions for Seismic Regulations for New Buildings and Other Structures" published by the Building Seismic Safety Council. We recommend Site Class "D" be used for design. Using this information, the structural engineer can select the appropriate site coefficient values (F_a and F_v) from ASCE 7-16 and/or the 2018 IBC to determine the maximum considered earthquake spectral response acceleration for the project. However, we have assumed the following response spectrum for the project:

Table 1. ASCE 7-16 Recommended Seismic Design Parameters

Site Class	S_S	S_1	F_a	F_v	S_{MS}	S_{M1}	S_{DS}	S_{D1}
D	0.830	0.421	1.200	1.879	0.996	0.791	0.664	0.527

Notes: 1. S_S and S_1 were established based on the USGS 2015 mapped maximum considered earthquake spectral acceleration maps for 2% probability of exceedence in 50 years.

2. F_a and F_v were established based on ASCE 7-16 using the selected S_S and S_1 values.

CONSTRUCTION MONITORING AND TESTING

We recommend that **Redmond Geotechnical Services, LLC** be retained to provide construction monitoring and testing services during all earthwork operations for the proposed new Liberty Road subdivision residential development. The purpose of our monitoring services would be to confirm that the site conditions reported herein are as anticipated, provide field recommendations as required based on the actual conditions encountered, document the activities of the grading contractor and assess his/her compliance with the project specifications and recommendations. It is important that our representative meet with the contractor prior to any site grading to help establish a plan that will minimize costly over-excavation and site preparation work. Of primary importance will be observations made during site preparation and stripping, structural fill placement, footing excavations and construction as well as retaining wall backfill.

CLOSURE AND LIMITATIONS

This report is intended for the exclusive use of the addressee and/or their representative(s) to use to design and construct the proposed new single-family residential structures and their associated site improvements described herein as well as to prepare any related construction documents. The conclusions and recommendations contained in this report are based on site conditions as they presently exist and assume that the explorations are representative of the subsurface conditions between the explorations and/or at other locations across the study area. The data, analyses, and recommendations herein may not be appropriate for other structures and/or purposes. We recommend that parties contemplating other structures and/or purposes contact our office. In the absence of our written approval, we make no representation and assume no responsibility to other parties regarding this report. Additionally, the above recommendations are contingent on Redmond Geotechnical Services, LLC being retained to provide all site inspections and constriction monitoring services for this project. Redmond Geotechnical Services, LLC will not assume any responsibility and/or liability for any engineering judgment, inspection and/or testing services performed by others.

It is the owners/developers responsibility for insuring that the project designers and/or contractors involved with this project implement our recommendations into the final design plans, specifications and/or construction activities for the project. Further, in order to avoid delays during construction, we recommend that the final design plans and specifications for the project be reviewed by our office to evaluate as to whether our recommendations have been properly interpreted and incorporated into the project.

If during any future site grading and construction, subsurface conditions different from those encountered in the explorations are observed or appear to be present beneath excavations, we should be advised immediately so that we may review these conditions and evaluate whether modifications of the design criteria are required. We also should be advised if significant modifications of the proposed site development are anticipated so that we may review our conclusions and recommendations.

LEVEL OF CARE

The services performed by the Geotechnical Engineer for this project have been conducted with that level of care and skill ordinarily exercised by members of the profession currently practicing in the area under similar budget and time restraints. No warranty or other conditions, either expressed or implied, is made.

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Appendix "A"

Test Pit Logs and Laboratory Test Data

APPENDIX

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATION

Subsurface conditions at the site were explored by excavating eleven (11) exploratory test pits (TH-#1 through TH-#11) on October 15, 2020. The approximate location of the test pit explorations are shown in relation to the proposed new residential lots and the associated site improvements on the Site Exploration Plan, Figure No's. 2=A and 2-B.

The test pits were excavated using track-mounted excavating equipment in general conformance with ASTM Methods in Vol. 4.08, D-1586-94 and D-1587-83. The test pits were excavated to depths ranging from about 6.0 to 9.0 feet beneath existing site grades. Detailed logs of the test pits are presented on the Log of Test Pits, Figure No's. A-4 through A-9. The soils were classified in accordance with the Unified Soil Classification System (USCS), which is outlined on Figure No. A-3.

The exploration program was coordinated by a field engineer who monitored the excavating and exploration activity, obtained representative samples of the subsurface soils encountered, classified the soils by visual and textural examination, and maintained continuous logs of the subsurface conditions. Disturbed and/or undisturbed samples of the subsurface soils were obtained at appropriate depths and/or intervals and placed in plastic bags and/or with a thin walled ring sample.

Groundwater was not encountered in any of the exploratory test pits (TH-#1 through TH-#11 at the time of excavating to depths of at least 9.0 feet beneath existing surface grades.

LABORATORY TESTING

Pertinent physical and engineering characteristics of the soils encountered during our subsurface investigation were evaluated by a laboratory testing program to be used as a basis for selection of soil design parameters and for correlation purposes. Selected tests were conducted on representative soil samples. The program consisted of tests to evaluate the existing (in-situ) moisture-density, maximum dry density and optimum moisture content, Atterberg Limits and gradational characteristics as well as direct shear strength and "R"-value tests.

Dry Density and Moisture Content Determinations

Density and moisture content determinations were performed on both disturbed and relatively undisturbed samples from the test pit explorations in general conformance with ASTM Vol. 4.08 Part D-216. The results of these tests were used to calculate existing overburden pressures and to correlate strength and compressibility characteristics of the soils. Test results are shown on the test pit logs at the appropriate sample depths.

Maximum Dry Density

One (1) Maximum Dry Density and Optimum Moisture Content test was performed on a representative sample of the on-site clayey, sandy silt subgrade soils in accordance with ASTM Vol. 4.08 Part D-1557. This test was conducted to help establish various engineering properties for use as structural fill. The test results are presented on Figure No. A-10.

Atterberg Limits

One (1) Liquid Limit (LL) and Plastic Limit (PL) test was performed on a representative sample of the sandy, clayey silt subgrade soils in accordance with ASTM Vol. 4.08 Part D-4318-85. These tests were conducted to facilitate classification of the soils and for correlation purposes. The test results appear on Figure No. A-11.

Gradation Analysis

One (1) Gradation analyses was performed on a representative sample of the subsurface soils in accordance with ASTM Vol. 4.08 Part D-422. The test results were used to classify the soil in accordance with the Unified Soil Classification System (USCS). The test results are shown graphically on Figure No. A-12.

Direct Shear Strength Test

One (1) Direct Shear Strength test was performed on a remolded sample at a continuous rate of shearing deflection (0.02 inches per minute) in accordance with ASTM Vol. 4.08 Part D-3080-79. The test results were used to determine engineering strength properties and are shown graphically on Figure No. A-13.

"R"-Value Tests

Two (2) "R"-value tests were performed on a remolded subgrade soil sample in accordance with ASTM Vol. 4.08 Part D-2844. The test results were used to help evaluate the subgrade soils supporting and performance capabilities when subjected to traffic loading. The test results are shown on Figure No. A-14.

The following figures are attached and complete the Appendix:

Figure No. A-3	Key To Exploratory Test Pit Logs
Figure No's. A-4 through A-9	Log of Test Pits
Figure No. A-10	Maximum Dry Density
Figure No. A-11	Atterberg Limits Test Results
Figure No. A-12	Gradation Test Results
Figure No. A-13	Direct Shear Strength Test Results
Figure No. A-14	Results of "R"-Value Tests
Figure No's. A-15 through A-17	Infiltration Test Results

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50%		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

DEFINITION OF TERMS

	U.S. STANDARD SERIES SIEVE: 200	40	10	4	CLEAR SQUARE SIEVE OPENINGS: 3/4"	3"	12"
SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

GRAIN SIZES

SANDS, GRAVELS AND NON-PLASTIC SILTS	BLOWS/FOOT †
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

RELATIVE DENSITY

CLAYS AND PLASTIC SILTS	STRENGTH ‡	BLOWS/FOOT †
VERY SOFT	0 - 1/4	0 - 2
SOFT	1/4 - 1/2	2 - 4
FIRM	1/2 - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	OVER 4	OVER 32

CONSISTENCY

† Number of blows of 140 pound hammer falling 30 inches to drive a 2 inch O.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586).
‡ Unconfined compressive strength in tons/sq. ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

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KEY TO EXPLORATORY TEST PIT LOGS
Unified Soil Classification System (ASTM D-2487)

LIBERTY ROAD SUBDIVISION
TL 600, 5871 Liberty Road South

PROJECT NO.	DATE	Figure
1001.072.G	11/13/20	A-3

BACKHOE COMPANY: Gene S. McMurrin BUCKET SIZE: 24 inches DATE: 10/15/20

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION TEST PIT NO. TH-#1 ELEVATION 540'±
0					ML	Dark brown, wet, soft, oreganic, sandy, clayey SILT (Topsoil)
	X			33.1	ML	Medium to reddish-brown, very moist, medium stiff to stiff, sandy, clayey SILT
5						
						Total Deoth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

TEST PIT NO. TH-#2 ELEVATION 554'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
					ML	Medium to reddish-brown, very moist, medium stiff to stiff, sandy, clayey SILT
5						
						Total Depth = 7.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1001.072.G LIBERTY ROAD SUBDIVISION FIGURE NO. A-4

BACKHOE COMPANY: Gene S. McMurrin BUCKET SIZE: 24 inches DATE: 10/15/20

SOIL DESCRIPTION

TEST PIT NO. TH-#3 ELEVATION 630'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
	X			32.2	ML	Medium to reddish-brown, very moist, medium stiff to stiff, sandy, clayey SILT
5						
	X			36.6		
10						Total Depth = 9.0 feet No groundwater encountered at time of exploration
15						

TEST PIT NO. TH-#4 ELEVATION 649'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
	X			32.9	ML	Medium to reddish-brown, very moist, medium stiff to stiff, sandy, clayey SILT
5						
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1001.072.G LIBERTY ROAD SUBDIVISION FIGURE NO. A-5

BACKHOE COMPANY: Gene S. McMurrin BUCKET SIZE: 24 inches DATE: 10/15/20

SOIL DESCRIPTION

TEST PIT NO. TH-#5 ELEVATION 639'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, very moist to wet, soft, organic, sandy, clayey SILT (Topsoil)
	X			34.6	ML	Medium to reddish-brown, very moist, medium stiff, sandy, clayey SILT
5	X			37.7		
						Total Depth = 7.0 feet No groundwater encountered at time of exploration
10						
15						

TEST PIT NO. TH-#6 ELEVATION 602'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
					ML	Medium to reddish-brown, very moist, medium stiff to stiff, sandy, clayey SILT
5						
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1001.072.G LIBERTY ROAD SUBDIVISION FIGURE NO. A-6

BACKHOE COMPANY: Gene S. McMurrin **BUCKET SIZE:** 24 inches **DATE:** 10/15/20

TEST PIT NO. TH-#7 **ELEVATION** 578'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
	X			35.0	ML	Medium to reddish-brown, very moist, medium stiff to stiff, sandy, clayey SILT
5						
	X			38.8		
						Total Depth = 7.0 feet No groundwater encountered at time of exploration
10						
15						

TEST PIT NO. TH-#8 **ELEVATION** 546'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
	X			34.1	ML	Medium to reddish-brown, very moist, medium stiff to stiff, sandy, clayey SILT
5						
	X			39.2		
						Total Depth = 7.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1001.072.G | LIBERTY ROAD SUBDIVISION | **FIGURE NO.** A-7

BACKHOE COMPANY: Gene S. McMurrin BUCKET SIZE: 24 inches DATE: 10/15/20

SOIL DESCRIPTION

TEST PIT NO. TH_#9 ELEVATION 553'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
					ML	Medium to reddish-brown, very moist, medium stiff to stiff, sandy, clayey SILT
5						
						Total Depth = 8.0 feet No groundwater encountered at time of exploration
10						
15						

TEST PIT NO. TH-#10 ELEVATION 591'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
	X			36.6	ML	Medium to reddish-brown, very moist, medium stiff to stiff, sandy, clayey SILT
5						
	X			38.9		
						Total Depth = 7.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1001.072.G | LIBERTY ROAD SUBDIVISION | FIGURE NO. A-8

BACKHOE COMPANY: Gene S. McMurrin BUCKET SIZE: 24 inches DATE: 10/15/20

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION TEST PIT NO. TH-#11 ELEVATION 648'±
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
					ML	Medium to reddish-brown, very moist, medium stiff to stiff, sandy, clayey SILT
5						
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	TEST PIT NO. ELEVATION
0						
5						
10						
15						

LOG OF TEST PITS

PROJECT NO. 1001.072.G | LIBERTY ROAD SUBDIVISION | FIGURE NO. A-9

MAXIMUM DENSITY TEST RESULTS

SAMPLE LOCATION	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)
TH-#10 @ 3.0'	Medium to reddish-brown, sandy, clayey SILT (ML)	104.0	30.0

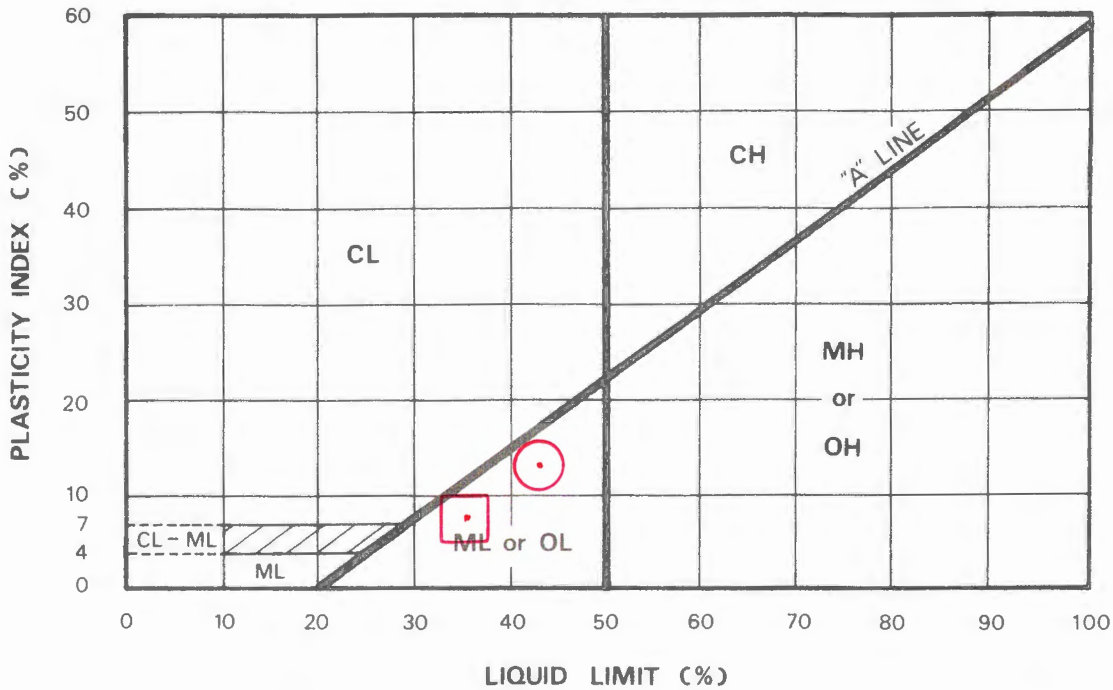
EXPANSION INDEX TEST RESULTS

SAMPLE LOCATION	INITIAL MOISTURE (%)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (%)	VOLUMETRIC SWELL (%)	EXPANSION INDEX	EXPANSIVE CLASS.

MAXIMUM DENSITY & EXPANSION INDEX TEST RESULTS

PROJECT NO.: 1001.072.G	LIBERTY ROAD SUBDIVISION	FIGURE NO.: A-10

REDMOND GEOTECHNICAL SERVICES



KEY SYMBOL	BORING NO.	SAMPLE DEPTH (feet)	NATURAL WATER CONTENT %	LIQUID LIMIT %	PLASTICITY INDEX %	PASSING NO. 200 SIEVE %	LIQUIDITY INDEX	UNIFIED SOIL CLASSIFICATION SYMBOL
	TH-#10	3.0	36.6	36.3	8.2	87.7		ML
	TH-#10	6.0	38.9	42.9	12.4	93.7		ML

REDMOND GEOTECHNICAL SERVICES
PO Box 20547 • Portland, Oregon 97294

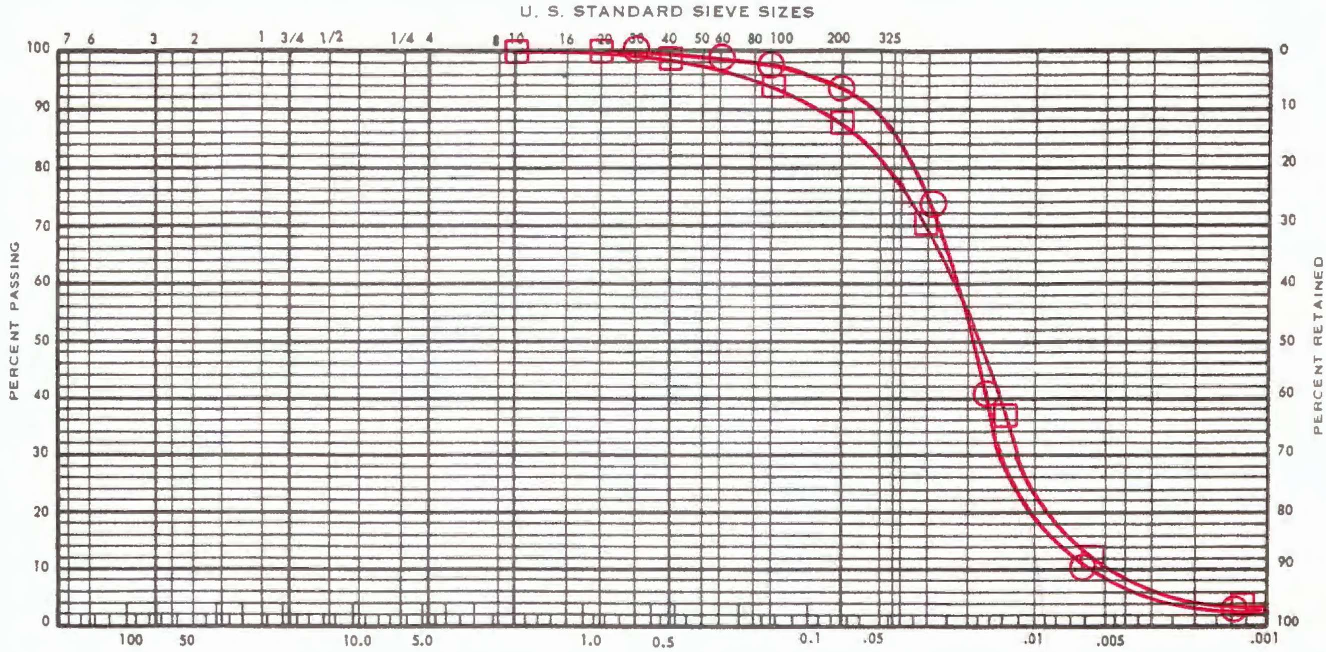
PLASTICITY CHART AND DATA

LIBERTY ROAD SUBDIVISION
TL 600, 5871 LIBERTY ROAD SOUTH

PROJECT NO.	DATE	Figure
1001.072.G	11/13/20	A-11

UNIFIED SOIL CLASSIFICATION SYSTEM

(ASTM D 422-72)

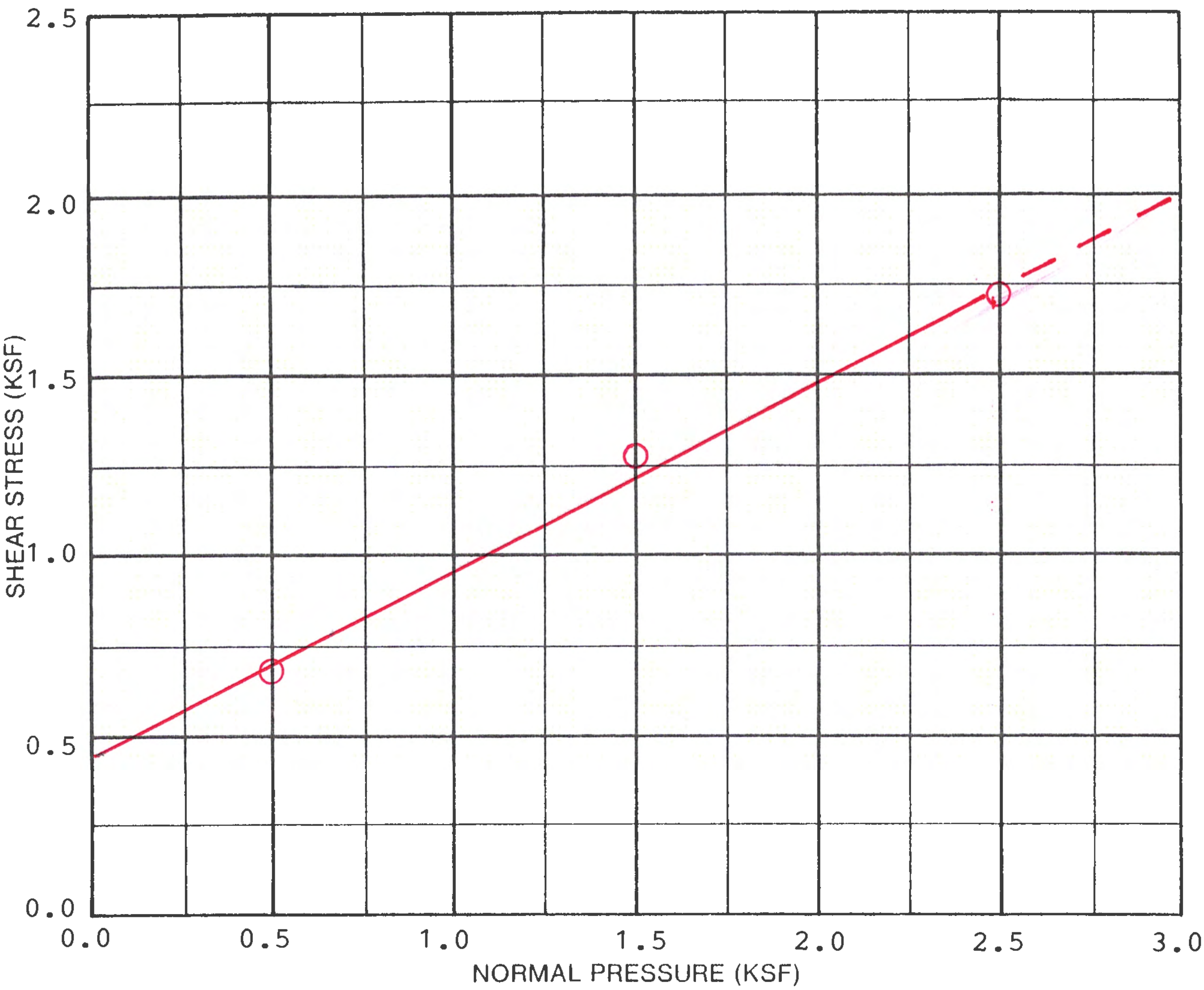


COBBLES	GRAVEL		SAND			SILT AND CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

KEY SYMBOL	BORING NO.	SAMPLE DEPTH (feet)	ELEV. (feet)	UNIFIED SOIL CLASSIFICATION SYMBOL	SAMPLE DESCRIPTION
—□—	TH-#10	3.0		ML	Medium to reddish-brown, sandy, clayey SILT
—○—	TH-#10	6.0		ML	Medium to reddish-brown, sandy, clayey SILT

REDMOND GEOTECHNICAL SERVICES
PO Box 20547 • Portland, Oregon 97294

GRADATION TEST DATA		
LIBERTY ROAD SUBDIVISION TL 600, 5871 LIBERTY ROAD SOUTH		
PROJECT NO.	DATE	FIGURE A-12
1001.072.G	11/13/20	



SAMPLE DATA	
DESCRIPTION: Medium to reddish-brown sandy, clayey SILT (ML) (Remolded)	
BORING NO.: TH-#10	
DEPTH (ft.): 3.0'	ELEVATION (ft):
TEST RESULTS	
APPARENT COHESION (C): 450 psf	
APPARENT ANGLE OF INTERNAL FRICTION (ϕ): 26°	

TEST DATA				
TEST NUMBER	1	2	3	4
NORMAL PRESSURE (KSF)	0.5	1.5	2.5	
SHEAR STRENGTH (KSF)	0.7	1.3	1.7	
INITIAL H_2O CONTENT (%)	30.0	30.0	30.0	
FINAL H_2O CONTENT (%)	31.1	25.5	19.3	
INITIAL DRY DENSITY (PCF)	91.0	91.0	91.0	
FINAL DRY DENSITY (PCF)	91.8	95.5	102.2	
STRAIN RATE: 0.02 inches per minute				

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PO Box 20547 • Portland, Oregon 97294

DIRECT SHEAR TEST DATA		
LIBERTY ROAD SUBDIVISION TL 600, 5871 Liberty Road South		
PROJECT NO.	DATE	Figure A-13
1001.072.G	11/13/20	

RESULTS OF R (RESISTANCE) VALUE TESTS

SAMPLE LOCATION: TH-#1

SAMPLE DEPTH: 2.5 feet bgs

Specimen	A	B	C
Exudation Pressure (psi)	214	325	424
Expansion Dial (o.ooo1")	0	1	2
Expansion Pressure (psf)	0	3	8
Moisture Content (%)	32.6	26.4	20.1
Dry Density (pcf)	93.4	97.2	101.6
Resistance Value, "R"	17	28	36
"R"-Value at 300 psi Exudation Pressure = 27			

SAMPLE LOCATION: TH-#10

SAMPLE DEPTH: 3.0 feet bgs

Specimen	A	B	C
Exudation Pressure (psi)	209	321	418
Expansion Dial (0.0001")	0	1	2
Expansion Pressure (psf)	0	3	8
Moisture Content (%)	33.3	27.1	21.7
Dry Density (pcf)	92.9	96.1	100.7
Resistance Value "R"	16	26	33
"R"-Value at 300 psi Exudation Pressure = 25			

Field Infiltration Test Results

Location: TL 600, 5871 Liberty Road South	**Date:** October 15, 2020	**Test Hole:** TH-#1
Depth to Bottom of Hole: 4.0 feet	**Hole Diameter:** 6 inches	**Test Method:** Encased Falling Head
Tester's Name: Daniel M. Redmond, P.E., G.E. **Tester's Company:** Redmond Geotechnical Services, LLC		**Tester's Contact Number:** 503-285-0598

Depth (feet)	**Soil Characteristics**
0-1.0	Dark brown Topsoil
1.0-4.0	Medium to reddish-brown, sandy, clayey SILT (ML)

Time	Time Interval (Minutes)	Measurement (inches)	Drop in Water (inches)	Infiltration Rate (inches/hour)	Remarks
11:00	0	36.00	----		Filled w/12" water
11:20	20	36.10	0.10	0.30	
11:40	20	36.19	0.09	0.27	
12:00	20	36.27	0.08	0.24	
12:20	20	36.34	0.07	0.21	
12:40	20	36.41	0.07	0.21	
1:00	20	36.47	0.06	0.18	
1:20	20	36.53	0.06	0.18	
1:40	20	36.59	0.06	0.18	

Infiltration Test Data Table

Field Infiltration Test Results

Location: TL 600, 5871 Liberty Road South	**Date:** October 15, 2020	**Test Hole:** TH-#2
Depth to Bottom of Hole: 4.0 feet	**Hole Diameter:** 6 inches	**Test Method:** Encased Falling Head
Tester's Name: Daniel M. Redmond, P.E., G.E. **Tester's Company:** Redmond Geotechnical Services, LLC		**Tester's Contact Number:** 503-285-0598

Depth (feet)	Soil Characteristics
0-1.0	Dark brown Topsoil
1.0-4.0	Medium to reddish-brown, sandy, clayey SILT (ML)

Time	Time Interval (Minutes)	Measurement (inches)	Drop in Water (inches)	Infiltration Rate (inches/hour)	Remarks
11:10	0	36.00	----		Filled w/12" water
11:30	20	36.09	0.09	0.27	
11:50	20	36.16	0.07	0.21	
12:10	20	36.22	0.06	0.18	
12:30	20	36.28	0.06	0.18	
12:50	20	36.33	0.05	0.15	
1:10	20	36.38	0.05	0.15	
1:30	20	36.42	0.05	0.15	
1:50	20	36.47	0.05	0.15	

Infiltration Test Data Table

Field Infiltration Test Results

Location: TL 600, 5871 Liberty Road South	**Date:** October 15, 2020	**Test Hole:** TH-#8
Depth to Bottom of Hole: 4.0 feet	**Hole Diameter:** 6 inches	**Test Method:** Encased Falling Head
Tester's Name: Daniel M. Redmond, P.E., G.E. **Tester's Company:** Redmond Geotechnical Services, LLC		**Tester's Contact Number:** 503-285-0598

Depth (feet)	Soil Characteristics
0-1.0	Dark brown Topsoil
1.0-4.0	Medium to reddish-brown, sandy, clayey SILT (ML)

Time	Time Interval (Minutes)	Measurement (inches)	Drop in Water (inches)	Infiltration Rate (inches/hour)	Remarks
11:20	0	36.00	----		Filled w/12" water
11:40	20	36.09	0.09	0.27	
12:00	20	36.17	0.08	0.24	
12:20	20	36.24	0.07	0.21	
12:40	20	36.31	0.07	0.21	
1:00	20	36.37	0.06	0.18	
1:20	20	36.43	0.06	0.18	
1:40	20	36.49	0.06	0.18	
2:00	20	36.55	0.06	0.18	

Infiltration Test Data Table

Figure No. A-17

Appendix "B"

Geologic Hazard Assessment

NORTHWEST GEOLOGICAL SERVICES, INC.
Consulting Geologists and Hydrogeologists
2505 N.E. 42nd Avenue, Portland, Oregon 97213-1201
503-249-1093 ngs@spiritone.com

Redmond Geotechnical Services **10 November 2020**
P. O. Box 20547
Portland, Or 97294

Attention: Dan Redmond

Geologic Hazard Assessment
5871 Liberty Rd S
8S/3W – 16C TL 600
Salem, Oregon

Dear Dan:

The purpose of this letter is to present Northwest Geological Services, Inc. (NGS) Geologic Hazard Assessment for the above referenced property as per your verbal authorization of 2 October 2020. We understand that our services are in support of your client's effort to subdivide and develop the site for residential use (Figures 1, 2 and 3).

1. Purpose and Scope of Study

The City landslide slope hazard database indicates that the site has a hazard score of from 2 to 3 points. City of Salem Planning rules add 3 more points for a new subdivision, thus, indicating that the site requires a geologic hazard assessment. The purpose of this letter is to meet that requirement.

For the study we conducted the following tasks:

- Reviewed DOGAMI hazard studies and maps of the area;
- Reviewed our files for several nearby sites;
- Reviewed geologic and topographic maps for the site area;[1]
- Reviewed aerial photographs and imagery (1936-2018);
- Observed samples from and logs of your test pits; and
- Prepared this letter.

2. Site Setting and Slopes

The subject property consists of the entirety of TL 600, located on the west side of Liberty Rd S at 5871 (Figures 1 and 2). It is the base of the north slope of the south Salem Hills on the divide north of the Jory Creek drainage (Figure 1). The site is currently open pasture with an existing residence near the NW corner (Figure 3) and auxiliary farm structures near the house and east end of the site (Figure 2). New residential developments border the site on the northwest, north and south as well as across Liberty Rd S to the east (Figures 1 and 2).

Topographic relief on the site is modest (Figure 3) with elevations ranging just from over 650 ft near the house down to below 540 near the entrance at the NE corner. Natural slopes

[1] We were unable to obtain City of Salem detailed Hazard or LIDAR topographic maps due to lack of response from the City Recorder to our Public Records request.

range from <5% at the east and west ends up to 18% in a band across the center of the site. A swale crosses the center part of the site from SW-NE. Only a small area in the north central part of the site, west of the swale, slopes more than 20% (Figure 3). A cut with retaining structure along the driveway exceeds 30%. It is an engineered slope and was made around 2008 for construction of Karen Lynn Loop, that abuts the site on the north (Figures 1 and 2, lower).

Figure 4 shows 1936 and 1954 aerial photos of the site and adjacent area. They show the site has been used for agricultural purposes (orchard and grains or pasture) since before 1936. By 1954 the west 1/3 was cleared (Figure 4, lower), However, by 1967 the west half was again planted as orchard.

The current residence was built in the 1986 along with the driveway along the north property line and the water well (Mari 11987). The SE corner with the old house was split off into TL 800. By 1990 the orchard and pasture appeared well maintained (Figure 5, upper). The orchard was uprooted and cleared by February 1990 (Figure 5, lower). The site has changed little since then (Figures 2 and 3).

3. Site Engineering Geology

According to published mapping (Foxworthy, 1970; Bella, 1981; Hoffman, 1981) and our geologic mapping for Marion County (NGS, 1997), the site area is underlain by the Sentinel Bluffs flows of the Columbia River Basalt. The basalt is mantled by up to several feet of weathered to decomposed basalt (10 ft in the site well, Mari 11987). The decomposed basalt is weathered to a hard-to-very hard red-brown clayey silt (laterite).

Eleven test pits[2] were excavated to confirm the depth to suitable foundation material and the nature of the overlying soils. The test pit and road cuts adjacent to the site exposed medium to red-brown, moist to very moist, medium stiff to stiff, sandy, clayey SILT. Up to a foot of topsoil mantles the SILT. It is dark brown, wet, soft, sandy organic clayey SILT. The topsoil is typical of the loessal soils of the Salem Hills.

The red-brown SILT grades downwards from medium stiff to stiff. Practical refusal in the test pits with the small excavator was reached at 6 to 9 feet depth. The SILT retains the texture of the parent basalt with ghost outlines of crystals and vesicles. The permeability appears to be quite low. Experience in the area (NGS, 1997) is that most recharge moves downward through desiccation cracks, fissures and relict root casts.

4. Geologic Hazards

Available geologic mapping shows no potential geologic hazards at the site or to the proposed development (previous section). Our mapping and the test pits show the site is underlain by stiff-to-hard soils with decomposed basalt at shallow depths. That is about as good a geologic setting one could expect to find in the Salem City limits. The only potential hazards we could identify were those estimated several years ago for earthquakes by DOGAMI (IMS-17) or more recently by their online tool SLIDO. Figure 6 shows those estimates.

[2] Test pits were done by Redmond Geotechnical Services for their Geotechnical Report for the site. Please refer to that report (attached) for test pit logs, locations and soil properties.

4.1 IMS-17 Estimated Earthquake Hazards

IMS-17 authors (Hofmeister & others, 2000) estimated a Very Low, Low or Moderate relative risk rating for the site (Figure 6, upper).[3] The hazards evaluated were failure of "…steep rock slope, soil slide and lateral spread…" IMS-17 divided the Salem area into thousands of small blocks (10-meter square cells) and used computer algorithms to estimate hazards based on the estimated geologic and slope properties in each cell.

IMS-17 used digitized versions of available topographic[4] and geologic maps and estimated cell properties from slope and generalized physical properties of various geologic units. Neither method is capable of estimating actual risk in a cell. Consequently, such maps are referred to as maps of relative risk because they are only useful for that purpose. A crucial shortcoming of the method is how slope is assigned to a particular cell. If there is no slope there cannot be landslide risk. Lateral spreading also requires topographic relief – there must be a nearby low area to spread into. Thus, hazards assigned to flat or low slope sites are glitches in the GIS.

4.2 SLIDO estimates of Landslide Hazard

SLIDO also used a 10-meter DEM but that DEM is derived from LIDAR and thus has much finer resolution IMS-17. SLIDO shows no mapped or historic landslides near the site. However, based on slope (Figure 6, middle) and assumed average soil conditions the SLIDO GIS model estimates a moderate to high landslide susceptibility for the site (Figure 6, bottom). The SLIDO source data for the area includes regional scale geologic mapping and the GIS topography layer. The "High" areas have slopes in the GIS from 20% to 30%. However, as noted, the entire site is underlain by decomposed basalt and has measured slopes of 20% or less except for a small area west of the swale.

4.3 Actual Potential for Geologic Hazards

Note that the most authoritative sources for water induced landslides (IMS-6) and seismic induced slope failures (GMS -105 do not extend to the site or into other flat areas of the City. However, both show geologically similar flat areas just north and west of the site as having no or very low landslide risk.

The site is low- to moderately-sloped. There are no natural slopes at the site steep enough to fail during an earthquake. There is no nearby declivity for site soils to spread towards, thus the potential for lateral spreading seems limited to the sides of the swale and the band of east-facing slope mid-site. Possibly, IMS-17 and SLIDO overestimated the risk because the GIS indicated a "steep slope" somewhere in the estimation cell that included the site. However, the lack of elevated risk for slope failure does not imply a lack of seismic risk. The site is subject to the same strong ground motions from local or distant earthquakes as are similar shallow bedrock sites throughout the area.

5. Conclusions and Recommendations

Areas shown as "low", or "moderate" or: high" hazard by IMS-17 or SLIDO are in fact low to moderately sloping, or have very localized topographic features of a few feet in ampli-

[3] Based on a scale of very low, low, moderate, and high hazard.

[4] IMS-17 estimated its Digital Elevation Model (DEM) on a 10-meter grid from 10-foot contour interval USGS topographic maps.

tude. The entire site is underlain by hard decomposed basalt. Thus, on-the-ground geologic reconnaissance and test pits indicate that there are no geologic or topographic features that create seismic induced slope failure risk beyond that of adjacent areas shown by IMS-17 as "no risk" or “low risk” areas. In our opinion, the site has a very low to nonexistent susceptibility to landsliding under any foreseeable natural geologic circumstance.

In our experience, the weathered basalt is not susceptible to slope spreading or liquefaction during strong ground motions from earthquakes. The basalt bedrock is at shallow depth and is should not be susceptible to failure of any sort during earthquakes. The site does not appear to be at significant risk from the forms of slope instability evaluated by IMS-17 or SLIDO. In our opinion, development of this site should not create new or exacerbate existing geologic hazards. However, we caution that any existing fills at the site may be subject to local failure or settlement during strong ground motions. We recommend that any man-made fills found during development be evaluated or replaced by structural fill beneath any new structures or pavements.

The topsoil and upper few feet of the decomposed basalt may be locally susceptible to soil creep on the moderate slope in the center of the site. The project geotechnical engineer should provide recommendations for lateral soil pressures. Additionally, because of the long site use for agriculture we recommend that excavation be inspected by a geotechnical engineer for fills or other unsuitable foundation conditions.

6. LIMITATIONS AND LIABILITY

We call your attention to the paragraphs on Warranty and Liability in the General Conditions (dated 1/2019) that you previously approved. Interpretations and recommendations presented herein are based on limited data and observations. Actual subsurface conditions may vary from those inferred from the limited information available to us. If site excavations for development find conditions to differ significantly from those inferred herein, you should contact us and provide an opportunity for us to review our recommendations for the site.

We thank you for the opportunity to assist you with your project. Please contact me if you have questions about the report.

Yours very truly,
Northwest Geological Services, Inc.

Expires 31 Oct 21

Clive F. (Rick) Kienle, Jr.
Principal Engineering Geologist
and Vice President

NGS Reference 235.118-1

7. References Cited

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Harvey, A. F. and G. L. Peterson, 1998, Water-induced landslide hazards, Western Portion of the Salem Hills, Marion County, Oregon, Oregon Dept. Geology & Mineral Industries, IMS-6.

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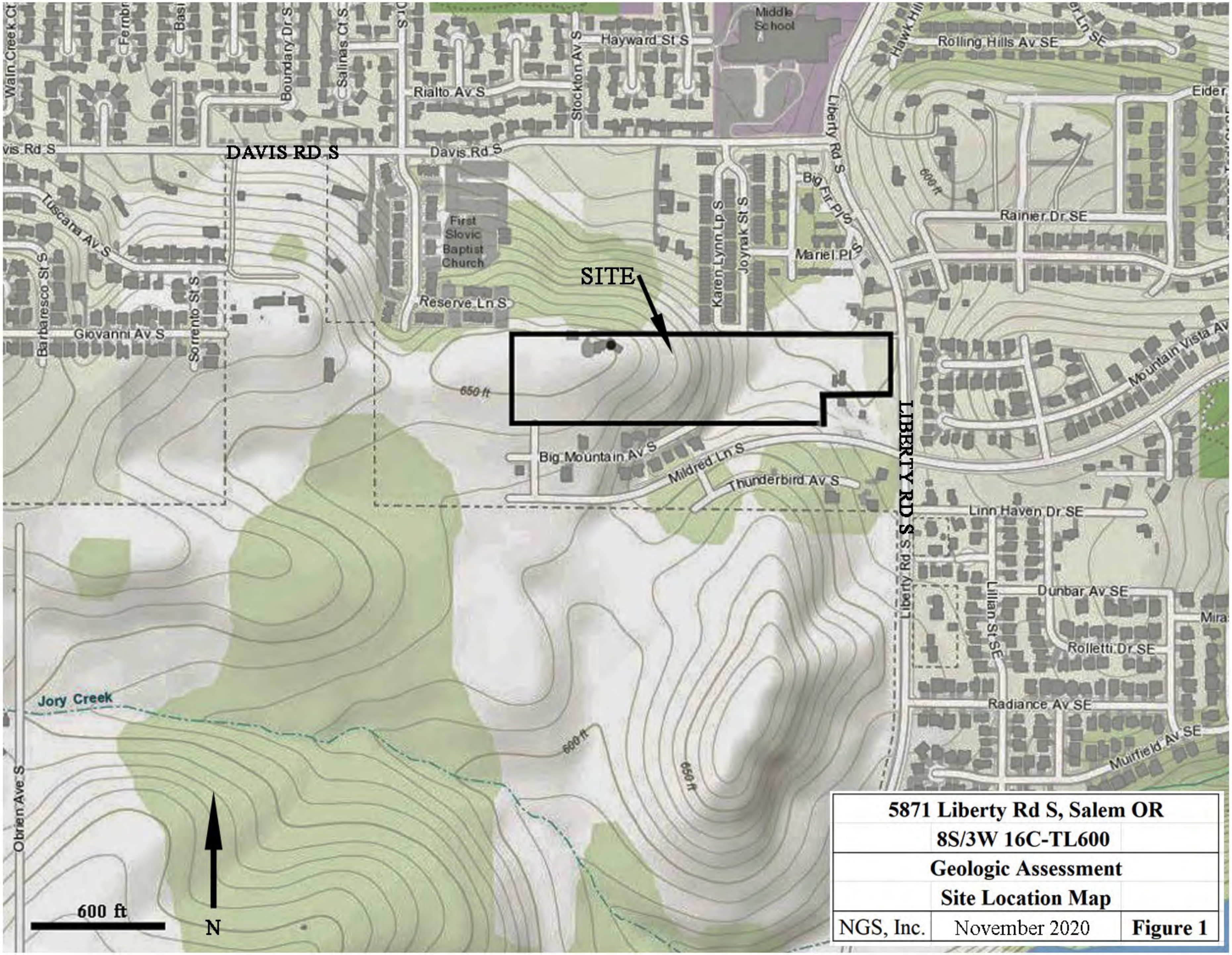
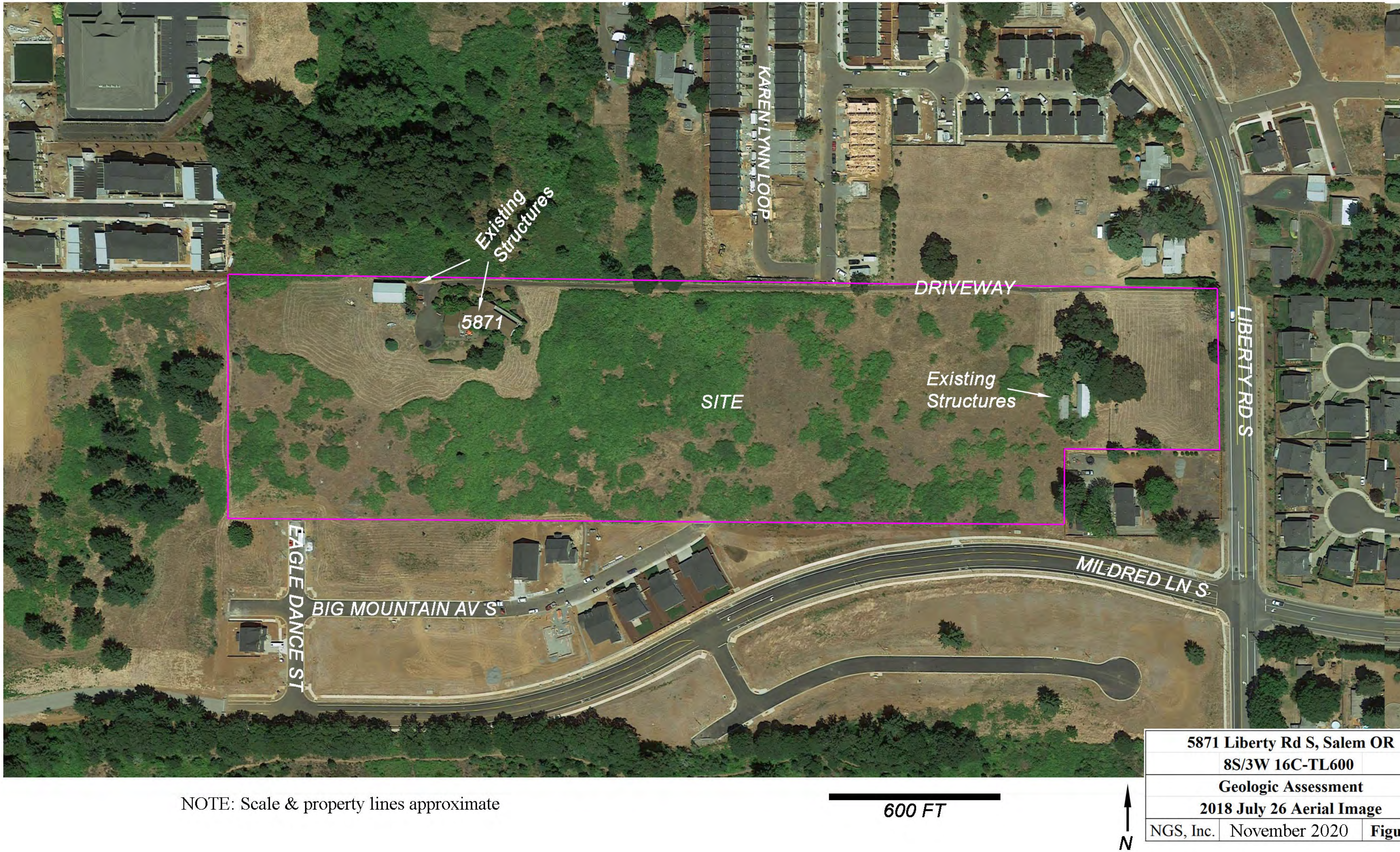
SLIDO, https://gis.dogami.oregon.gov/maps/slido/ for 5871 Liberty Rd S, Salem OR

Wang, Y. and W.J. Leonard, 1996, Relative earthquake hazard maps of the Salem E. and Salem W. Quadrangles, Marion and Polk Counties, Oregon, Oregon Dept. Geology & Mineral Industries, Geologic Map Series, GMS-105.

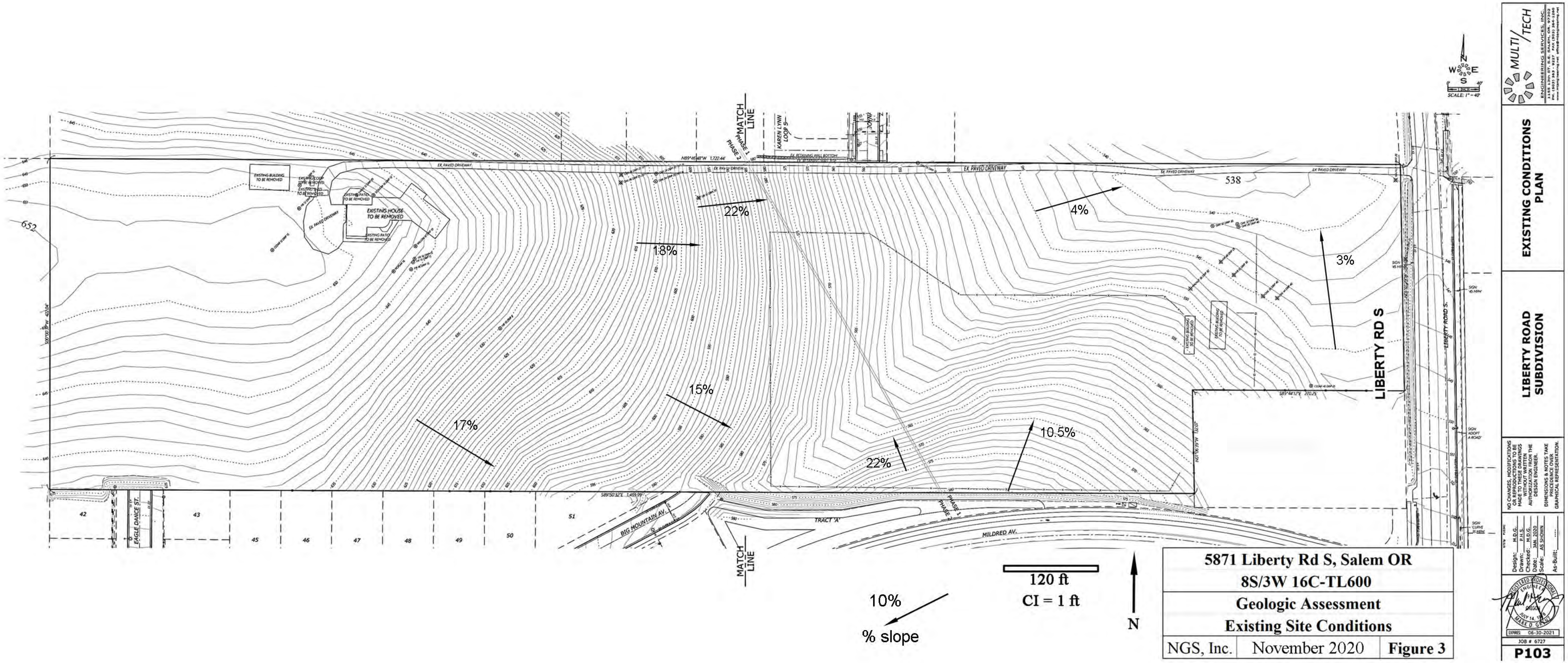


5871 Liberty Rd S, Salem OR		
8S/3W 16C-TL600		
Geologic Assessment		
Site Location Map		
NGS, Inc.	November 2020	**Figure 1**



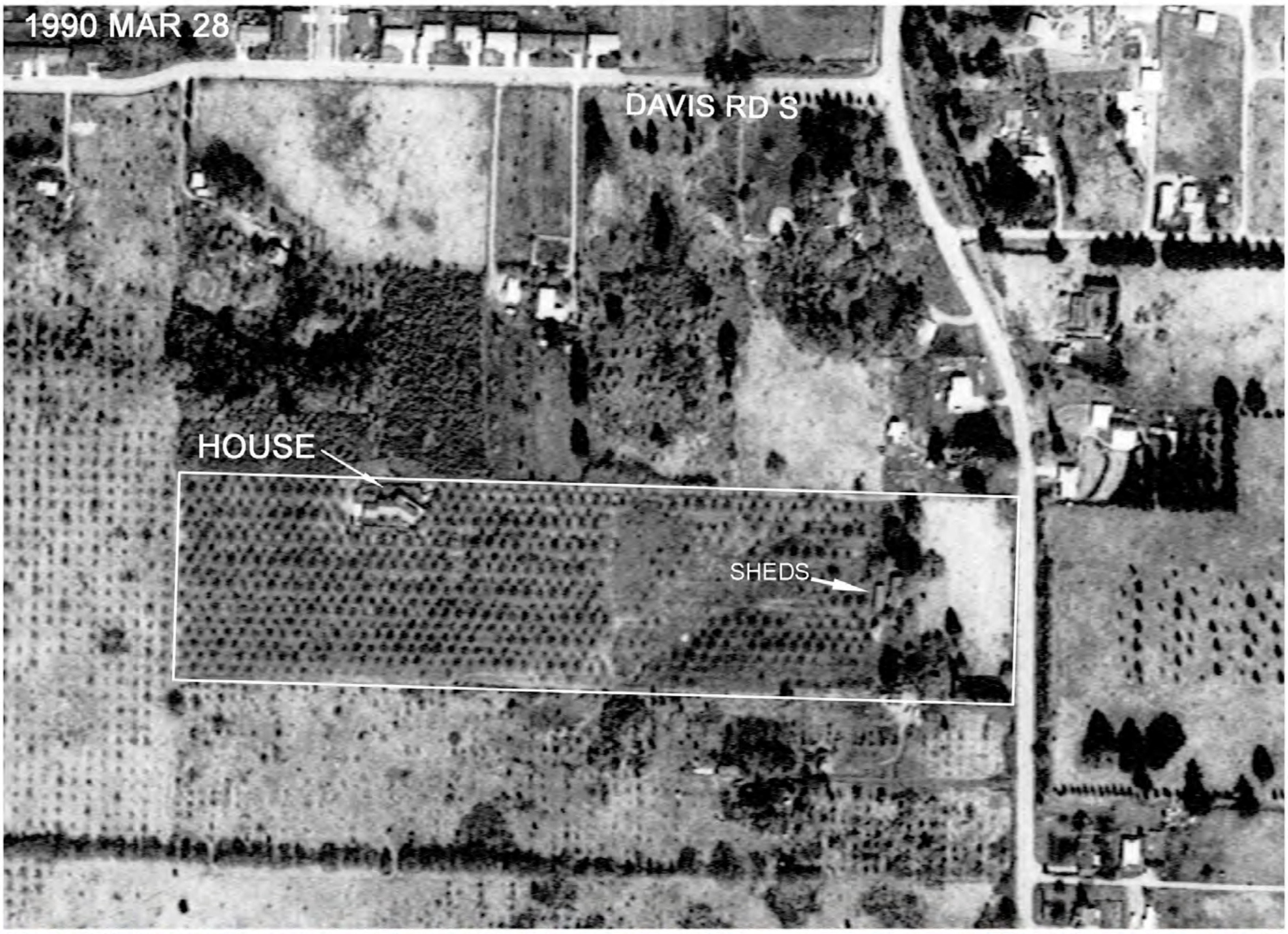
NOTE: Scale & property lines approximate

5871 Liberty Rd S, Salem OR		
8S/3W 16C-TL600		
Geologic Assessment		
2018 July 26 Aerial Image		
NGS, Inc.	November 2020	**Figure 2**



5871 Liberty Rd S, Salem OR		
8S/3W 16C-TL600		
Geologic Assessment		
Existing Site Conditions		
NGS, Inc.	November 2020	Figure 3

5871 Liberty Rd S, Salem OR		
8S/3W 16C-TL600		
Geologic Assessment		
1936 and 1954 Aerial Photos		
NGS, Inc.	October 2020	**Figure 4**

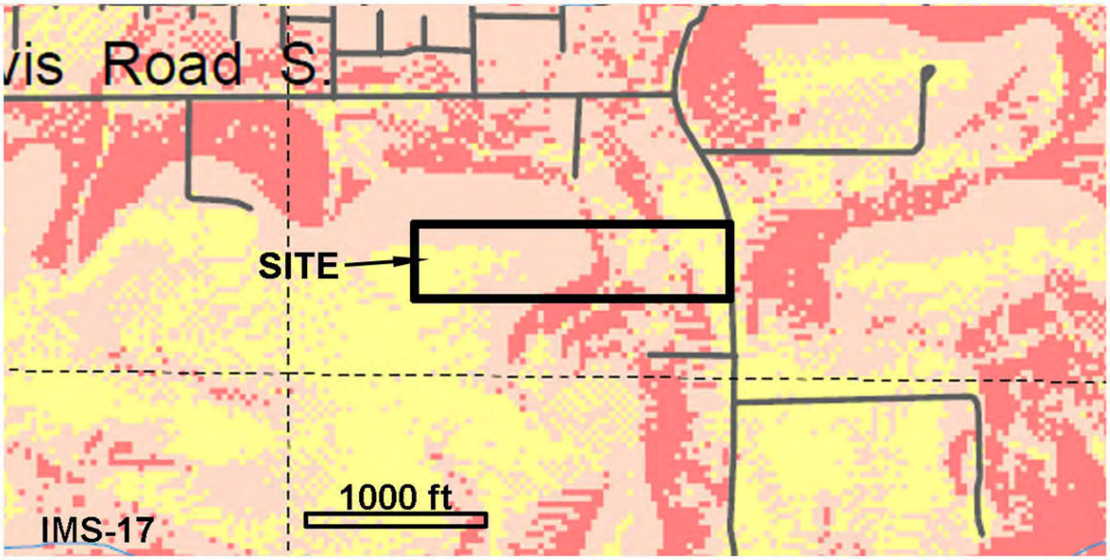


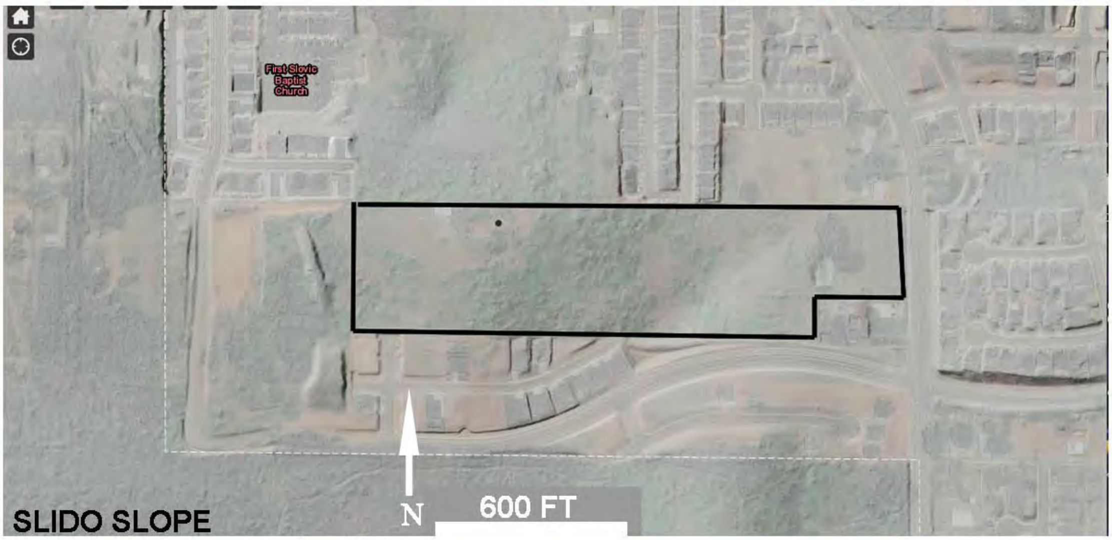
NOTE: Aerial photos from USGS

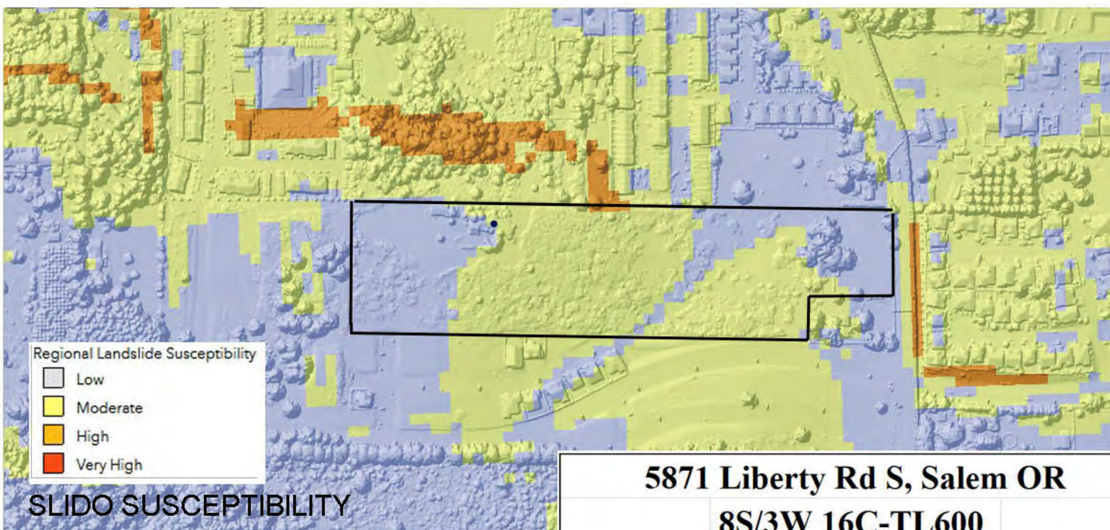
300 feet

N

5871 Liberty Rd S, Salem OR		
8S/3W 16C-TL600		
Geologic Assessment		
1990 & 2008 Aerial Images		
NGS, Inc.	November 2020	**Figure 5**

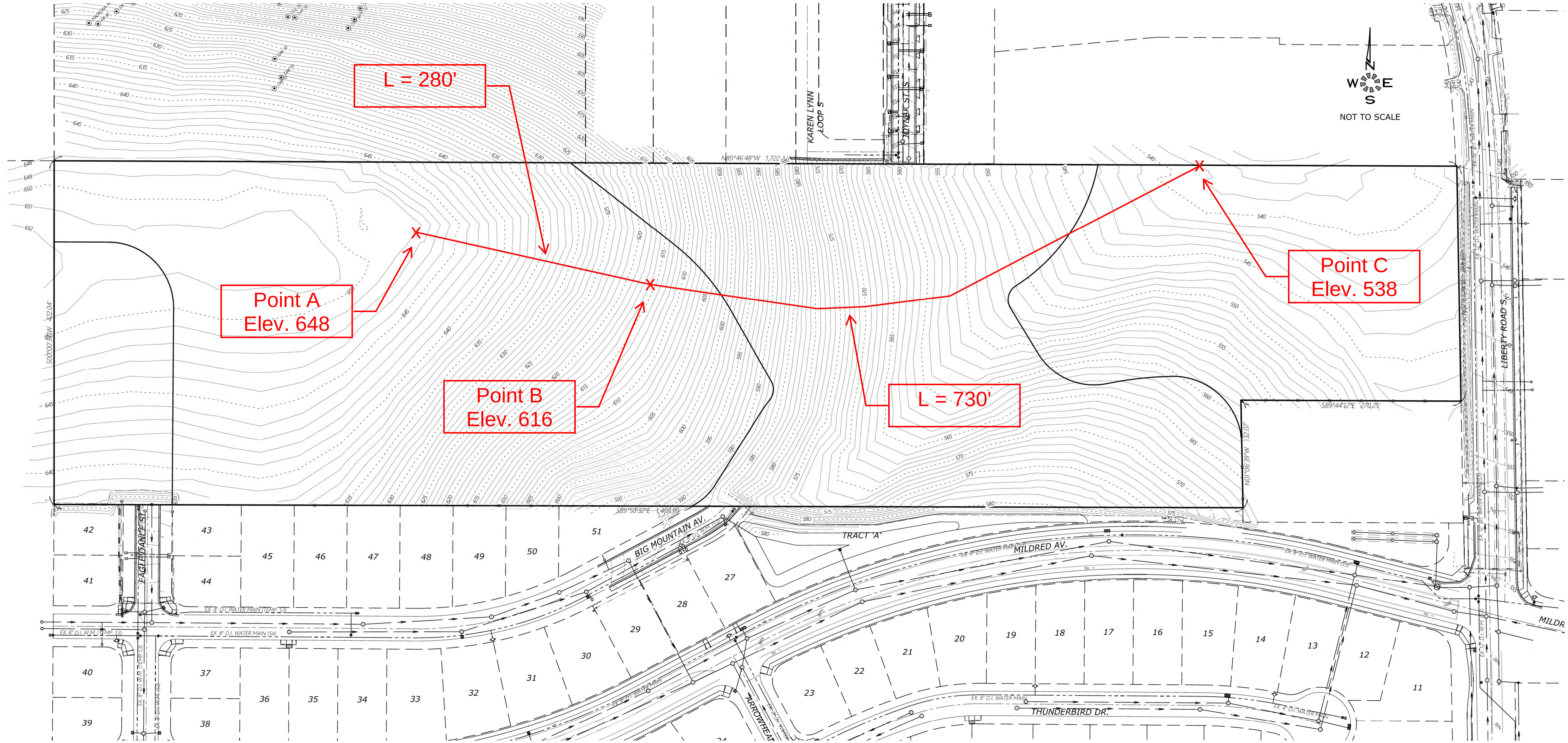






5871 Liberty Rd S, Salem OR		
8S/3W 16C-TL600		
Geologic Assessment		
DOGAMI Hazard Estimates		
NGS, Inc.	November 2020	**Figure 6**

Appendix C



Time of Concentration Map

MULTI/TECH
ENGINEERING SERVICES, INC.
1155 13th ST. S.E. SALEM, OR. 97302
PH. (503) 363-9227 FAX (503) 364-1260
www.mtengineering.net office@mtengineering.net

EXISTING CONDITIONS PLAN

LIBERTY ROAD SUBDIVISION AND APARTMENTS

NO CHANGES, MODIFICATIONS OR REPRODUCTIONS TO BE MADE TO THESE DRAWINGS WITHOUT WRITTEN AUTHORIZATION FROM THE DESIGN ENGINEER.

DIMENSIONS & NOTES TAKE PRECEDENCE OVER GRAPHICAL REPRESENTATION.

Design: M.D.G.
Drawn: P.H.S.
Checked: M.D.G.
Date: JAN. 2020
Scale: AS SHOWN

JOB # 6727

1 of 1

Worksheet 3: Time of Concentration (T_c) or travel time (T_t)

Project: Liberty Road Condominiums	By: C. O'Sullivan	Date: 10/29/2021
Location: Salem, Oregon	Checked:	Date:

Check one: ☑ Present ☐ Developed

Check one: ☑ T_c ☑ T_t through subarea

Notes: Space for as many as two segments per flow type can be used for each worksheet. Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to Tc only)

	Segment ID	A-B		
1.	Surface description (Table 4D-4)	Pre-developed Mixed		
2.	Manning's roughness coefficient, n (Table 4D-4)	0.3		
3.	Flow length, L (total L † 300 ft) ... ft	280		
4.	Two-year 24-hour rainfall, P_2 ... in	2.2		
5.	Land slope, s ... ft/ft	0.114		
6.	$T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{\ 0.5}\ s^{0.4}}$ Compute T_t ... hr	0.390	+	= 0.390

Shallow concentrated flow

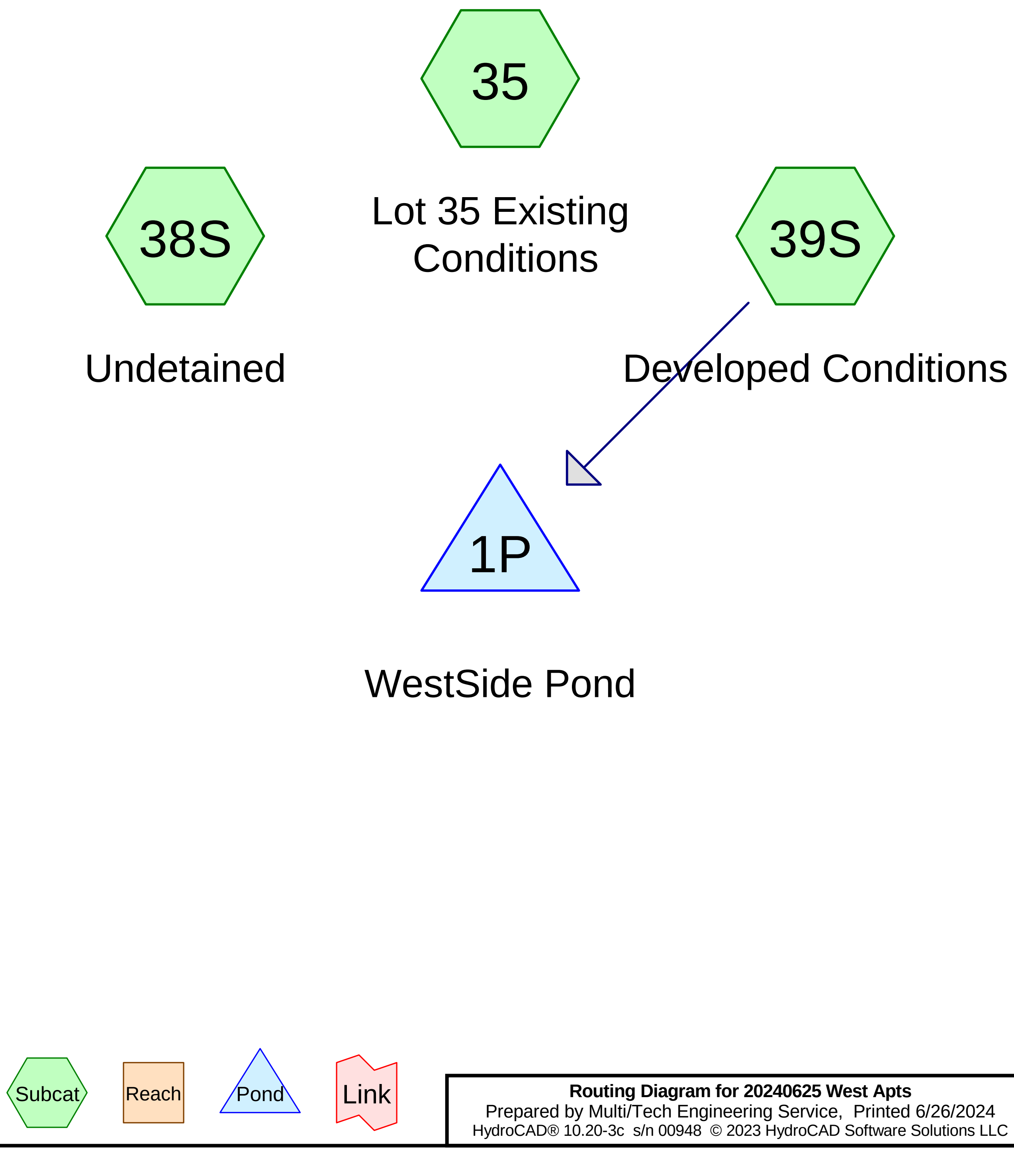
	Segment ID	B-C		
7.	Surface description (paved or unpaved)	Minimum Tillage		
8.	Flow length, L ... ft	730		
9.	Watercourse slope, s ... ft/ft	0.107		
10.	Average velocity, V (figure 3-1) ... ft/s	1.5		
11.	$T_t = \frac{L}{3600\ V}$ Compute T_t ... hr	0.135	+	= 0.135

Channel flow

	Segment ID			
12.	Cross sectional flow area, a ... ft^2			
13.	Wetted perimeter, p_w ... ft			
14.	Hydraulic radius, $r = \frac{a}{p_w}$ Compute r ... ft			
15	Channel slope, s ... ft/ft			
16.	Manning's roughness coefficient, n ...			
17.	$V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ... ft/s			
18.	~~Flow length~~, L ... ft			
19.	$T_t = \frac{L}{3600\ V}$ Compute T_t ... hr		+	=
20.	Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19) ... Hr			0.525

0.525 Hrs = 31.5 Minutes

Appendix D



Routing Diagram for 20240625 West Apts
Prepared by Multi/Tech Engineering Service, Printed 6/26/2024

Summary for Subcatchment 35: Lot 35 Existing Conditions

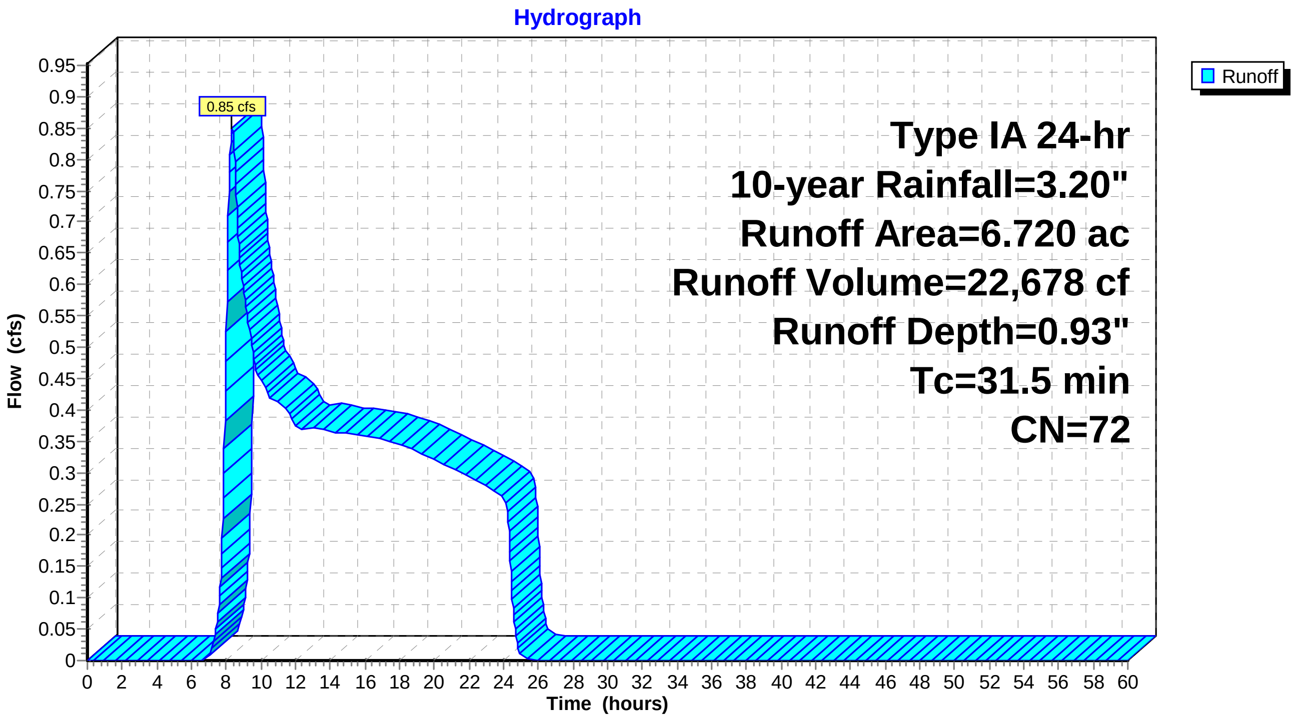
Runoff = 0.85 cfs @ 8.33 hrs, Volume= 22,678 cf, Depth= 0.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-60.00 hrs, dt= 0.03 hrs
Type IA 24-hr 10-year Rainfall=3.20"

	Area (ac)	CN	Description
*	6.720	72	City of Salem Pre-developed, HSG C
	6.720		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
31.5					**Direct Entry, TR-55 Worksheet**

Subcatchment 35: Lot 35 Existing Conditions



Summary for Subcatchment 38S: Undetained

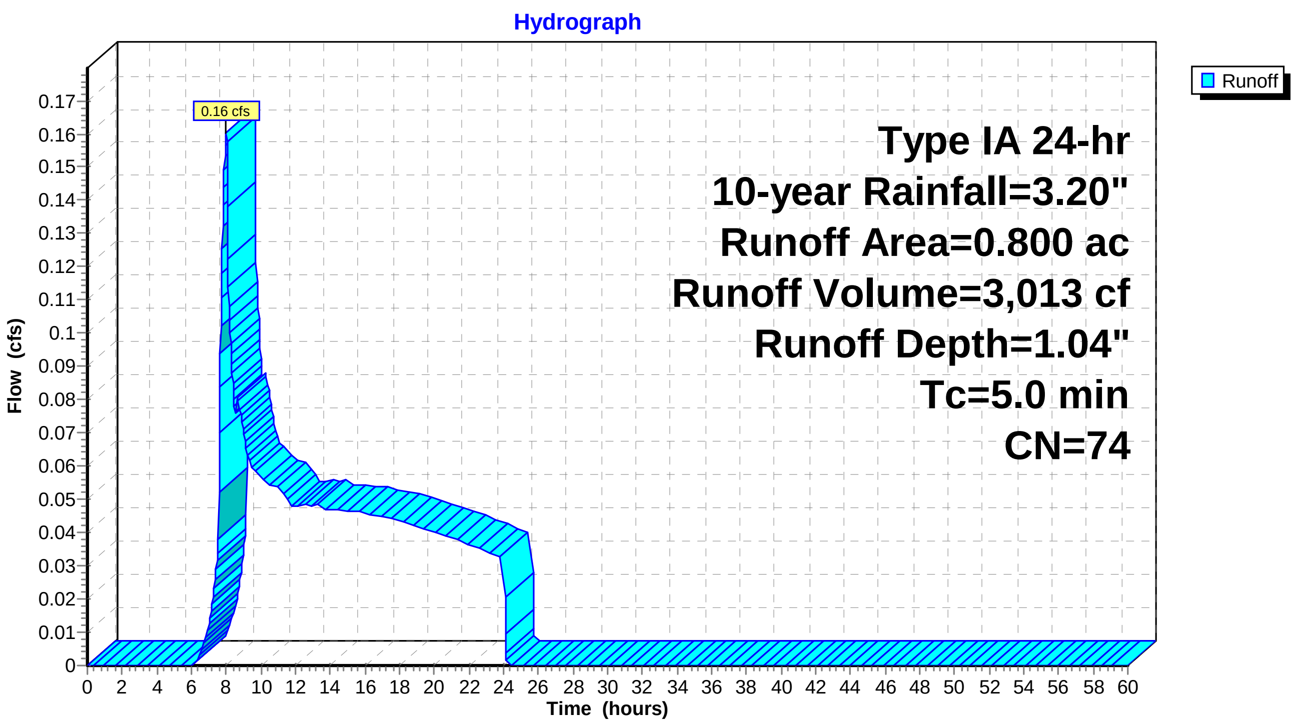
Runoff = 0.16 cfs @ 8.00 hrs, Volume= 3,013 cf, Depth= 1.04"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-60.00 hrs, dt= 0.03 hrs
Type IA 24-hr 10-year Rainfall=3.20"

Area (ac)	CN	Description
0.800	74	>75% Grass cover, Good, HSG C
0.800		100.00% Pervious Area

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

Subcatchment 38S: Undetained



Summary for Subcatchment 39S: Developed Conditions

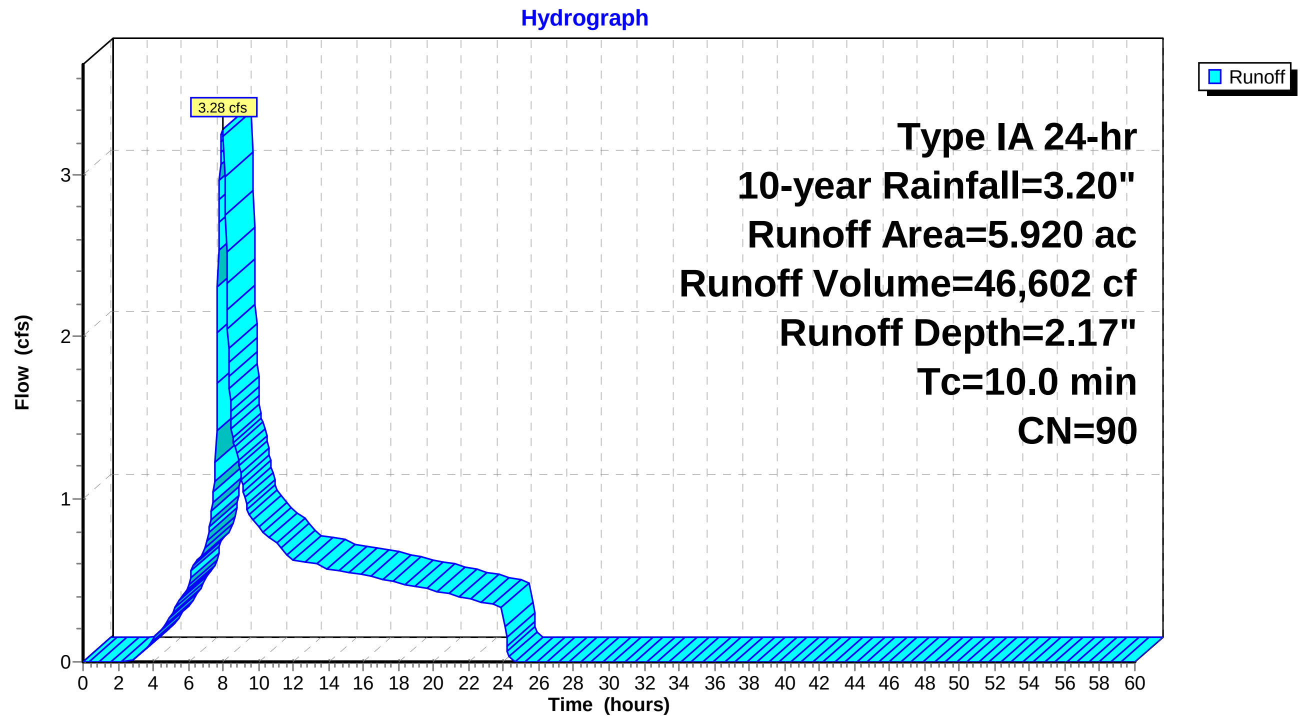
Runoff = 3.28 cfs @ 7.99 hrs, Volume= 46,602 cf, Depth= 2.17"
Routed to Pond 1P : WestSide Pond

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-60.00 hrs, dt= 0.03 hrs
Type IA 24-hr 10-year Rainfall=3.20"

Area (ac)	CN	Description
4.032	98	Unconnected pavement, HSG C
1.888	74	>75% Grass cover, Good, HSG C
5.920	90	Weighted Average
1.888		31.89% Pervious Area
4.032		68.11% Impervious Area
4.032		100.00% Unconnected

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

Subcatchment 39S: Developed Conditions



Summary for Pond 1P: WestSide Pond

Inflow Area = 257,875 sf, 68.11% Impervious, Inflow Depth = 2.17" for 10-year event
Inflow = 3.28 cfs @ 7.99 hrs, Volume= 46,602 cf
Outflow = 0.63 cfs @ 11.81 hrs, Volume= 41,286 cf, Atten= 81%, Lag= 229.3 min
Primary = 0.63 cfs @ 11.81 hrs, Volume= 41,286 cf

Routing by Stor-Ind method, Time Span= 0.00-60.00 hrs, dt= 0.03 hrs
Peak Elev= 607.21' @ 11.81 hrs Surf.Area= 4,238 sf Storage= 16,708 cf

Plug-Flow detention time= 453.0 min calculated for 41,286 cf (89% of inflow)
Center-of-Mass det. time= 378.7 min (1,129.7 - 751.0)

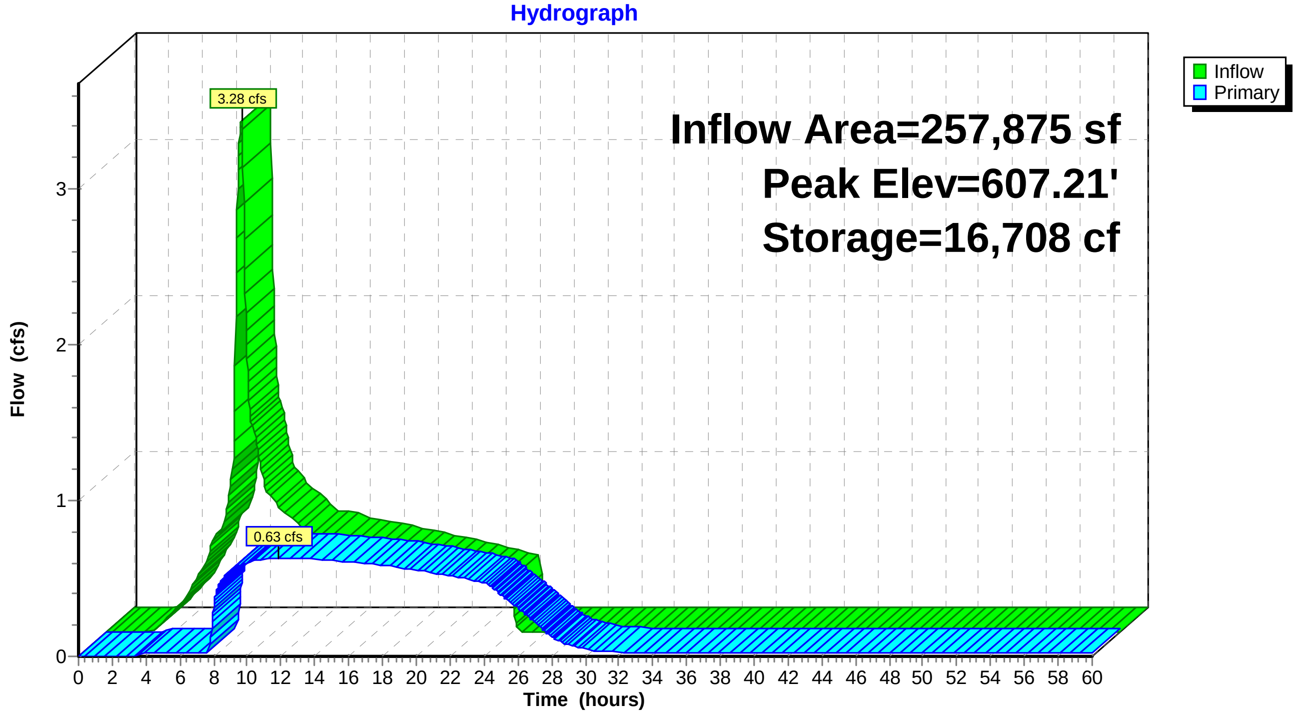
Volume	Invert	Avail.Storage	Storage Description
#1	601.99'	28,817 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)
#2	595.00'	255 cf	**5.00'D x 13.00'H Vertical Cone/Cylinder**
		29,073 cf	Total Available Storage

Elevation (feet)	Surf.Area (sq-ft)	Voids (%)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
601.99	1	0.0	0	0
602.99	3,591	0.0	0	0
603.00	3,591	100.0	36	36
604.00	3,735	100.0	3,663	3,699
605.00	3,885	100.0	3,810	7,509
606.00	4,035	100.0	3,960	11,469
607.00	4,186	100.0	4,111	15,579
608.00	4,337	100.0	4,262	19,841
609.00	4,488	100.0	4,413	24,253
610.00	4,640	100.0	4,564	28,817

Device	Routing	Invert	Outlet Devices
#1	Primary	598.50'	**12.000" Round Culvert** L= 100.5' RCP, rounded edge headwall, Ke= 0.100 Inlet / Outlet Invert= 598.50' / 578.78' S= 0.1962 '/' Cc= 0.900 n= 0.013, Flow Area= 0.79 sf
#2	Device 1	599.00'	**0.625" Vert. Orifice/Grate** C= 0.600 Limited to weir flow at low heads
#3	Device 1	605.00'	**4.000" Vert. Orifice/Grate** C= 0.600 Limited to weir flow at low heads
#4	Device 1	607.75'	**0.50' long x 0.75' rise Weir Notch** Cv= 2.62 (C= 3.28)
#5	Device 1	608.50'	**12.000" Horiz. Standpipe** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=0.63 cfs @ 11.81 hrs HW=607.21' (Free Discharge)
└─**1=Culvert** (Passes 0.63 cfs of 14.78 cfs potential flow)
  ├─**2=Orifice/Grate** (Orifice Controls 0.03 cfs @ 13.78 fps)
  ├─**3=Orifice/Grate** (Orifice Controls 0.60 cfs @ 6.89 fps)
  ├─**4=Weir Notch** (Controls 0.00 cfs)
  └─**5=Standpipe** (Controls 0.00 cfs)

Pond 1P: WestSide Pond



Summary for Subcatchment 35: Lot 35 Existing Conditions

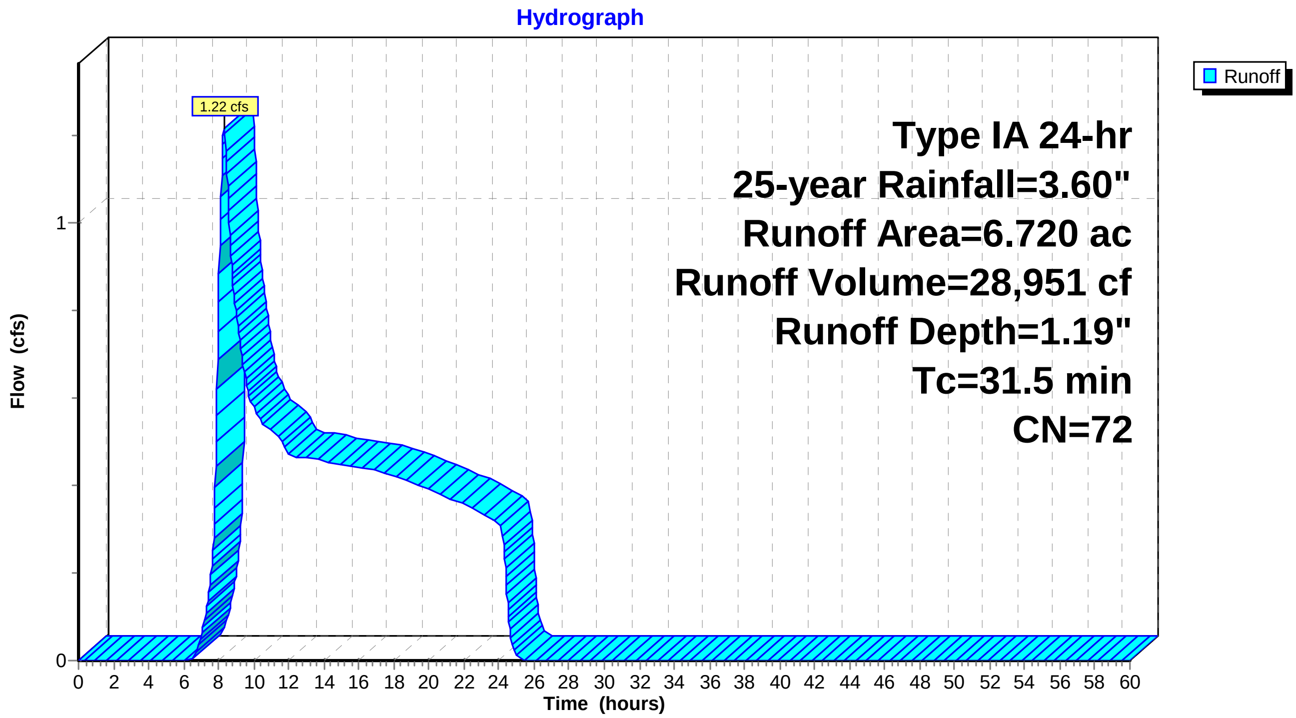
Runoff = 1.22 cfs @ 8.31 hrs, Volume= 28,951 cf, Depth= 1.19"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-60.00 hrs, dt= 0.03 hrs
Type IA 24-hr 25-year Rainfall=3.60"

	Area (ac)	CN	Description
*	6.720	72	City of Salem Pre-developed, HSG C
	6.720		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
31.5					**Direct Entry, TR-55 Worksheet**

Subcatchment 35: Lot 35 Existing Conditions



Summary for Subcatchment 38S: Undetained

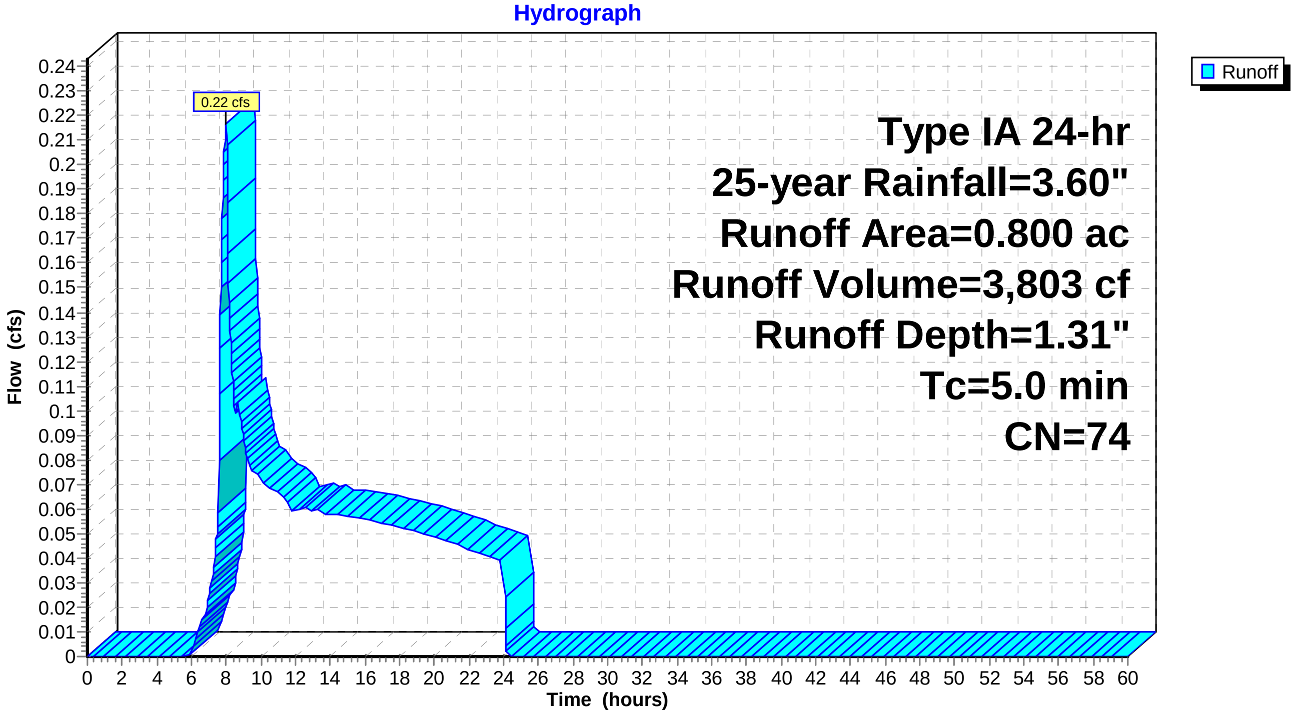
Runoff = 0.22 cfs @ 8.00 hrs, Volume= 3,803 cf, Depth= 1.31"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-60.00 hrs, dt= 0.03 hrs
Type IA 24-hr 25-year Rainfall=3.60"

Area (ac)	CN	Description
0.800	74	>75% Grass cover, Good, HSG C
0.800		100.00% Pervious Area

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

Subcatchment 38S: Undetained



Summary for Subcatchment 39S: Developed Conditions

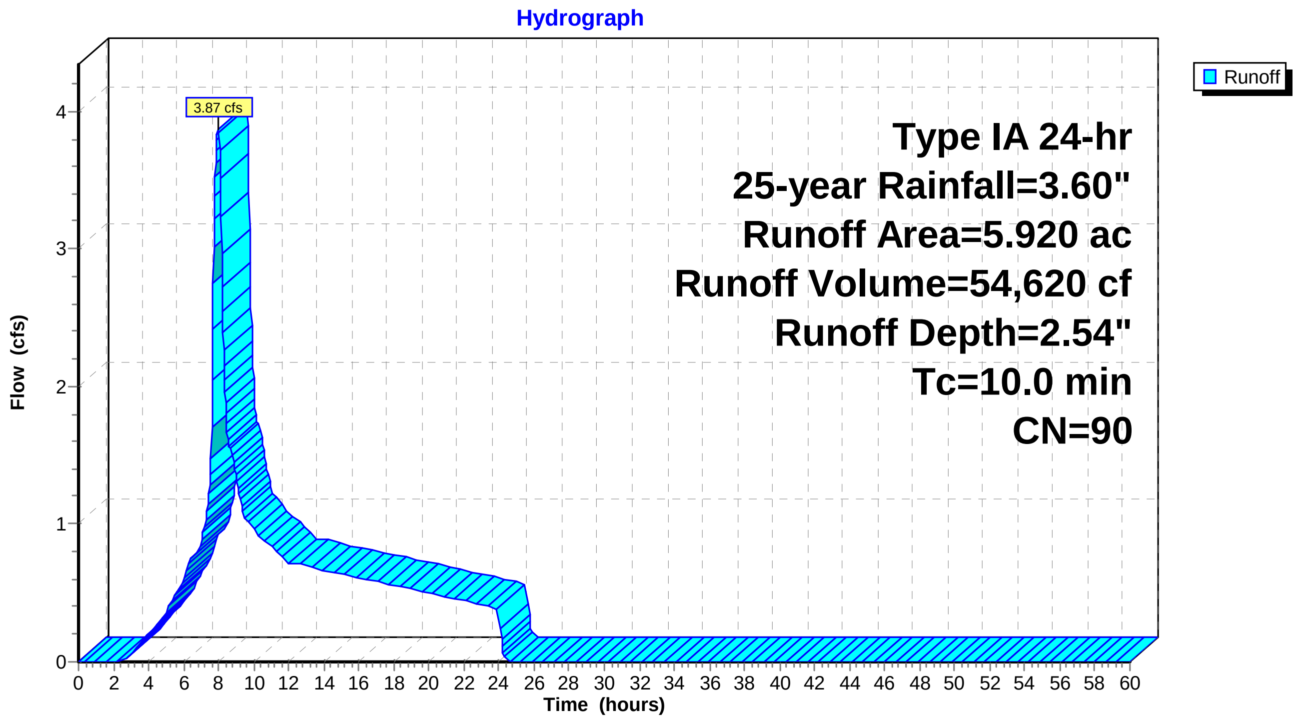
Runoff = 3.87 cfs @ 7.98 hrs, Volume= 54,620 cf, Depth= 2.54"
Routed to Pond 1P : WestSide Pond

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-60.00 hrs, dt= 0.03 hrs
Type IA 24-hr 25-year Rainfall=3.60"

Area (ac)	CN	Description
4.032	98	Unconnected pavement, HSG C
1.888	74	>75% Grass cover, Good, HSG C
5.920	90	Weighted Average
1.888		31.89% Pervious Area
4.032		68.11% Impervious Area
4.032		100.00% Unconnected

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

Subcatchment 39S: Developed Conditions



Summary for Pond 1P: WestSide Pond

Inflow Area = 257,875 sf, 68.11% Impervious, Inflow Depth = 2.54" for 25-year event
Inflow = 3.87 cfs @ 7.98 hrs, Volume= 54,620 cf
Outflow = 0.80 cfs @ 11.34 hrs, Volume= 49,218 cf, Atten= 79%, Lag= 201.2 min
Primary = 0.80 cfs @ 11.34 hrs, Volume= 49,218 cf

Routing by Stor-Ind method, Time Span= 0.00-60.00 hrs, dt= 0.03 hrs
Peak Elev= 607.88' @ 11.34 hrs Surf.Area= 4,338 sf Storage= 19,554 cf

Plug-Flow detention time= 440.2 min calculated for 49,193 cf (90% of inflow)
Center-of-Mass det. time= 375.6 min (1,118.2 - 742.6)

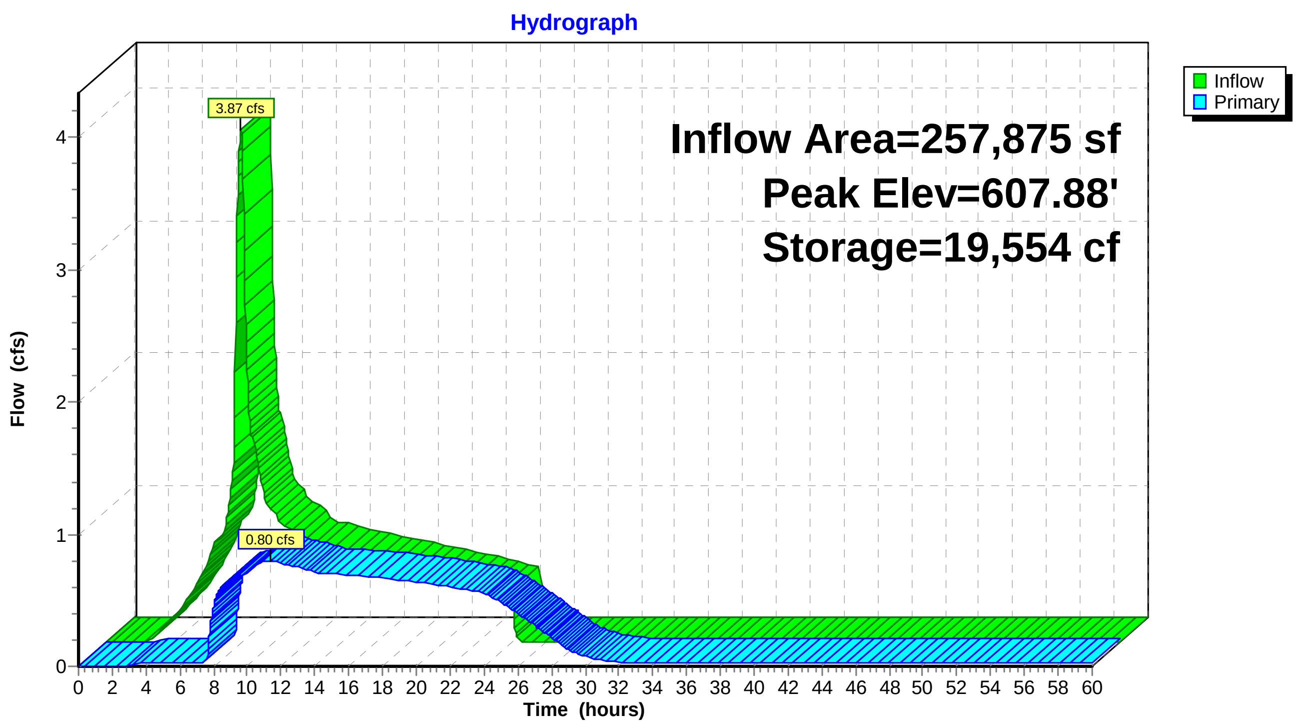
Volume	Invert	Avail.Storage	Storage Description
#1	601.99'	28,817 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)
#2	595.00'	255 cf	**5.00'D x 13.00'H Vertical Cone/Cylinder**
		29,073 cf	Total Available Storage

Elevation (feet)	Surf.Area (sq-ft)	Voids (%)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
601.99	1	0.0	0	0
602.99	3,591	0.0	0	0
603.00	3,591	100.0	36	36
604.00	3,735	100.0	3,663	3,699
605.00	3,885	100.0	3,810	7,509
606.00	4,035	100.0	3,960	11,469
607.00	4,186	100.0	4,111	15,579
608.00	4,337	100.0	4,262	19,841
609.00	4,488	100.0	4,413	24,253
610.00	4,640	100.0	4,564	28,817

Device	Routing	Invert	Outlet Devices
#1	Primary	598.50'	**12.000" Round Culvert** L= 100.5' RCP, rounded edge headwall, Ke= 0.100 Inlet / Outlet Invert= 598.50' / 578.78' S= 0.1962 '/' Cc= 0.900 n= 0.013, Flow Area= 0.79 sf
#2	Device 1	599.00'	**0.625" Vert. Orifice/Grate** C= 0.600 Limited to weir flow at low heads
#3	Device 1	605.00'	**4.000" Vert. Orifice/Grate** C= 0.600 Limited to weir flow at low heads
#4	Device 1	607.75'	**0.50' long x 0.75' rise Weir Notch** Cv= 2.62 (C= 3.28)
#5	Device 1	608.50'	**12.000" Horiz. Standpipe** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=0.79 cfs @ 11.34 hrs HW=607.88' (Free Discharge)
└─**1=Culvert** (Passes 0.79 cfs of 15.36 cfs potential flow)
  ├─**2=Orifice/Grate** (Orifice Controls 0.03 cfs @ 14.32 fps)
  ├─**3=Orifice/Grate** (Orifice Controls 0.69 cfs @ 7.92 fps)
  ├─**4=Weir Notch** (Weir Controls 0.07 cfs @ 1.16 fps)
  └─**5=Standpipe** (Controls 0.00 cfs)

Pond 1P: WestSide Pond



Summary for Subcatchment 35: Lot 35 Existing Conditions

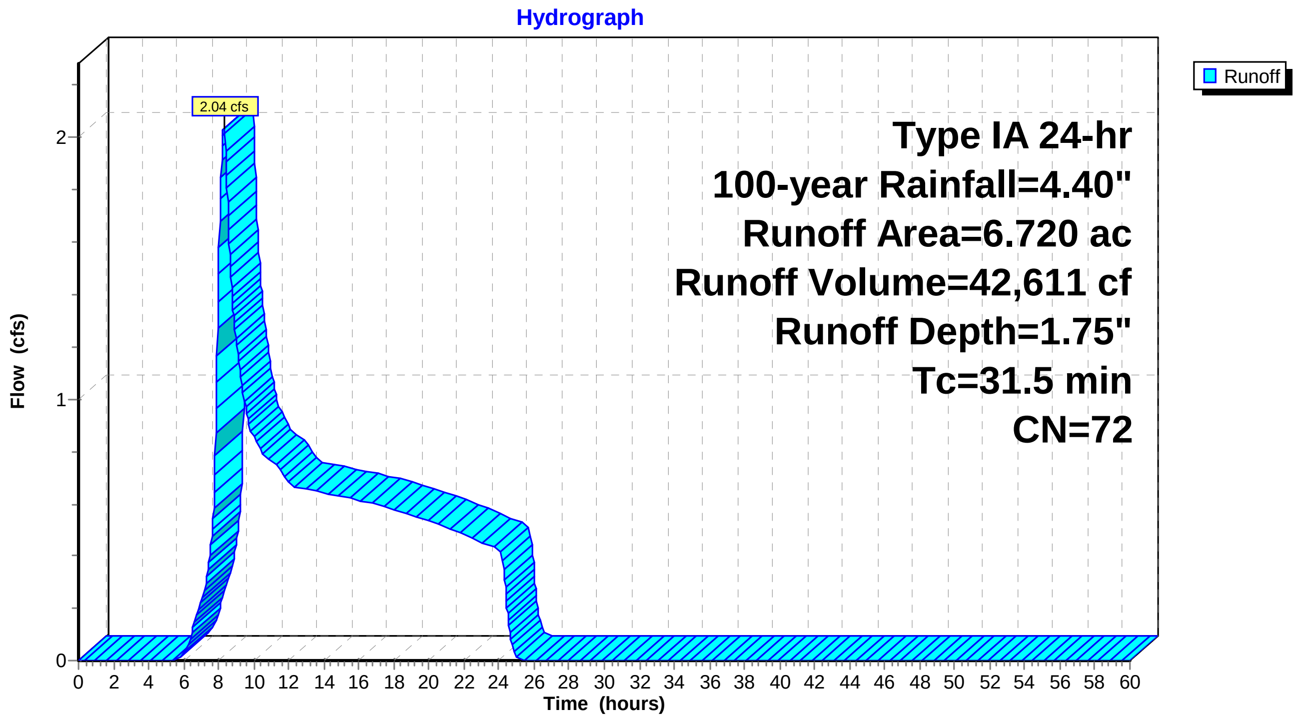
Runoff = 2.04 cfs @ 8.28 hrs, Volume= 42,611 cf, Depth= 1.75"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-60.00 hrs, dt= 0.03 hrs
Type IA 24-hr 100-year Rainfall=4.40"

	Area (ac)	CN	Description
*	6.720	72	City of Salem Pre-developed, HSG C
	6.720		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
31.5					**Direct Entry, TR-55 Worksheet**

Subcatchment 35: Lot 35 Existing Conditions



Summary for Subcatchment 38S: Undetained

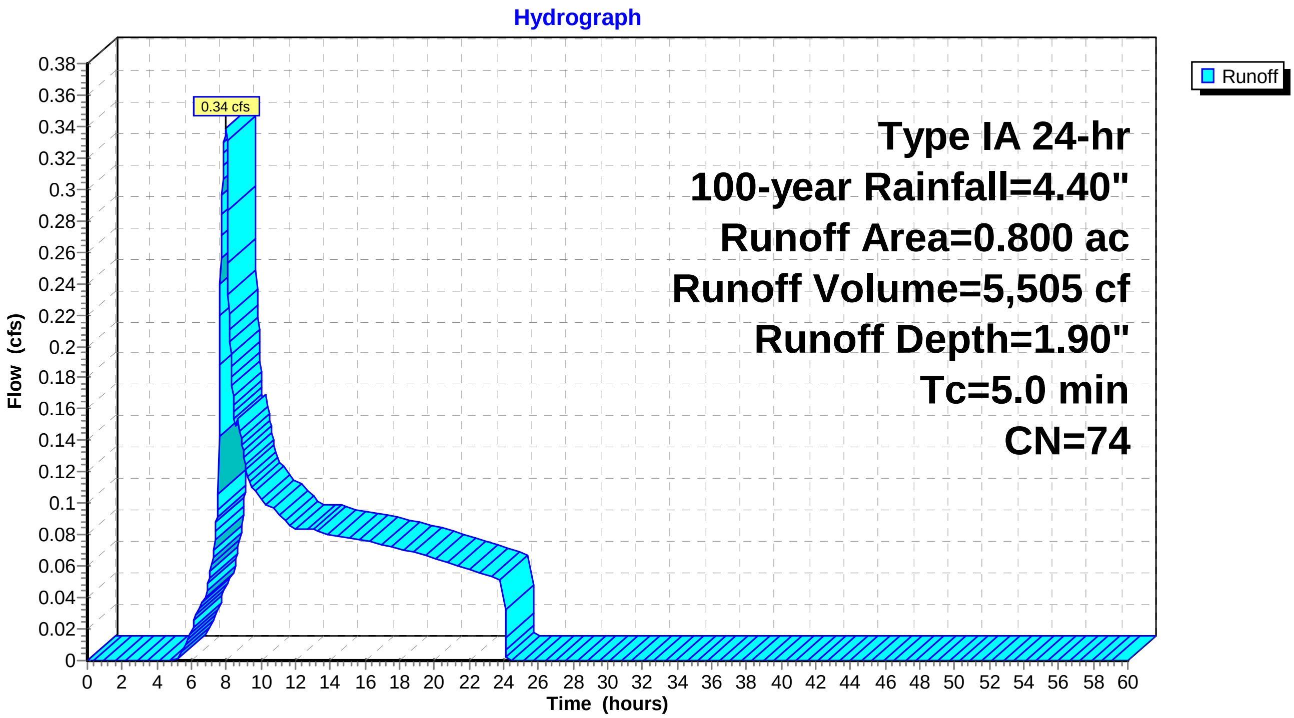
Runoff = 0.34 cfs @ 7.98 hrs, Volume= 5,505 cf, Depth= 1.90"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-60.00 hrs, dt= 0.03 hrs
Type IA 24-hr 100-year Rainfall=4.40"

Area (ac)	CN	Description
0.800	74	>75% Grass cover, Good, HSG C
0.800		100.00% Pervious Area

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

Subcatchment 38S: Undetained



Summary for Subcatchment 39S: Developed Conditions

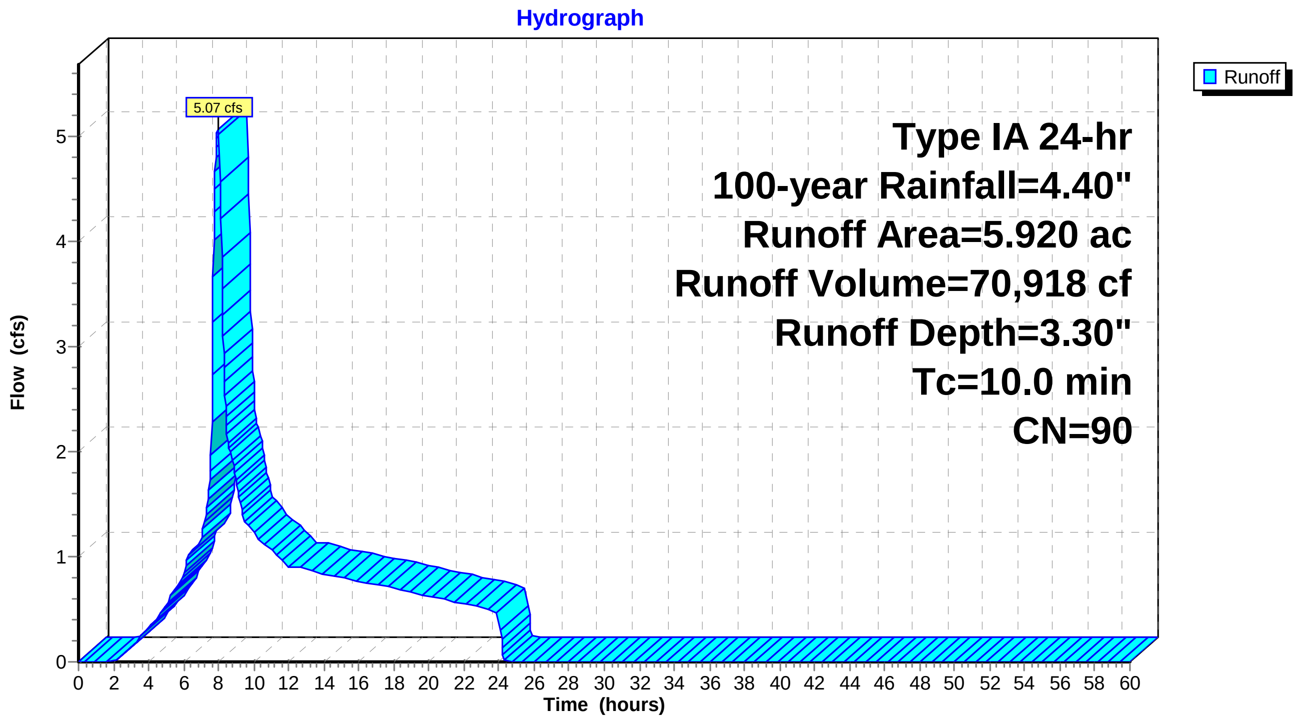
Runoff = 5.07 cfs @ 7.97 hrs, Volume= 70,918 cf, Depth= 3.30"
Routed to Pond 1P : WestSide Pond

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-60.00 hrs, dt= 0.03 hrs
Type IA 24-hr 100-year Rainfall=4.40"

Area (ac)	CN	Description
4.032	98	Unconnected pavement, HSG C
1.888	74	>75% Grass cover, Good, HSG C
5.920	90	Weighted Average
1.888		31.89% Pervious Area
4.032		68.11% Impervious Area
4.032		100.00% Unconnected

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

Subcatchment 39S: Developed Conditions



Summary for Pond 1P: WestSide Pond

Inflow Area = 257,875 sf, 68.11% Impervious, Inflow Depth = 3.30" for 100-year event
Inflow = 5.07 cfs @ 7.97 hrs, Volume= 70,918 cf
Outflow = 1.66 cfs @ 9.06 hrs, Volume= 65,422 cf, Atten= 67%, Lag= 65.4 min
Primary = 1.66 cfs @ 9.06 hrs, Volume= 65,422 cf

Routing by Stor-Ind method, Time Span= 0.00-60.00 hrs, dt= 0.03 hrs
Peak Elev= 608.41' @ 9.06 hrs Surf.Area= 4,418 sf Storage= 21,866 cf

Plug-Flow detention time= 374.6 min calculated for 65,422 cf (92% of inflow)
Center-of-Mass det. time= 321.6 min (1,050.9 - 729.3)

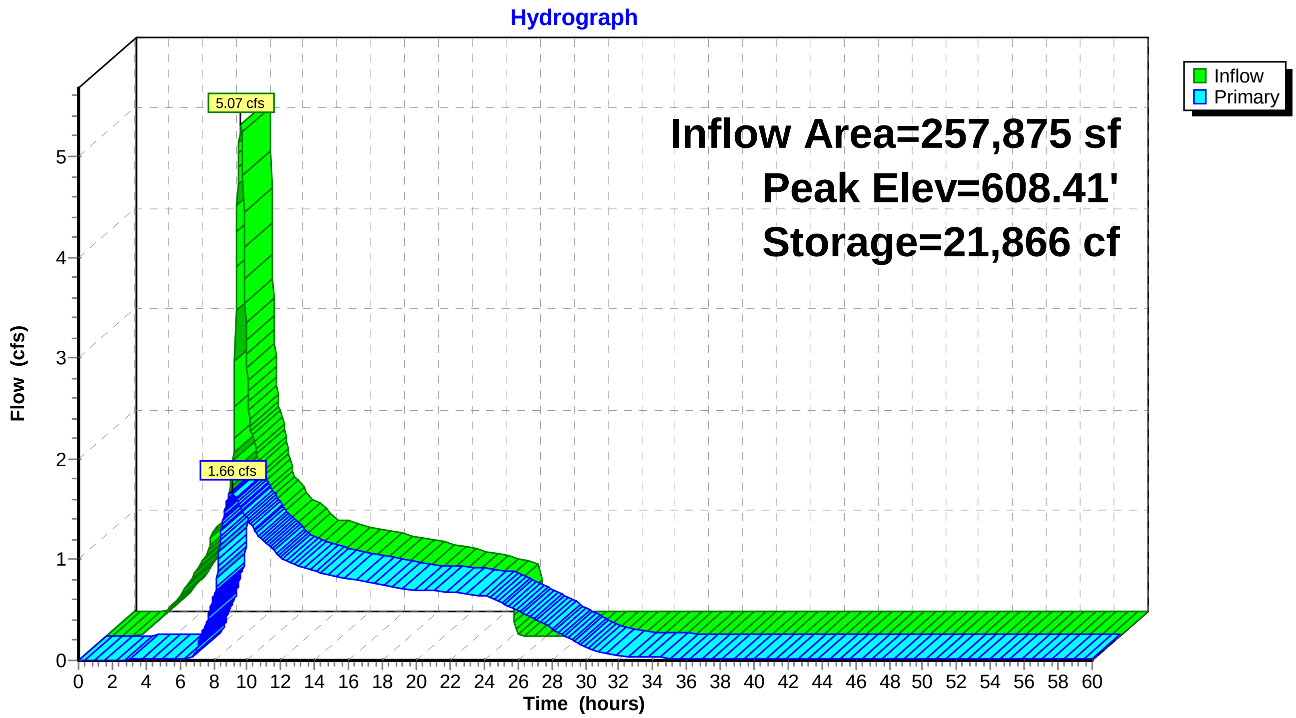
Volume	Invert	Avail.Storage	Storage Description
#1	601.99'	28,817 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)
#2	595.00'	255 cf	**5.00'D x 13.00'H Vertical Cone/Cylinder**
		29,073 cf	Total Available Storage

Elevation (feet)	Surf.Area (sq-ft)	Voids (%)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
601.99	1	0.0	0	0
602.99	3,591	0.0	0	0
603.00	3,591	100.0	36	36
604.00	3,735	100.0	3,663	3,699
605.00	3,885	100.0	3,810	7,509
606.00	4,035	100.0	3,960	11,469
607.00	4,186	100.0	4,111	15,579
608.00	4,337	100.0	4,262	19,841
609.00	4,488	100.0	4,413	24,253
610.00	4,640	100.0	4,564	28,817

Device	Routing	Invert	Outlet Devices
#1	Primary	598.50'	**12.000" Round Culvert** L= 100.5' RCP, rounded edge headwall, Ke= 0.100 Inlet / Outlet Invert= 598.50' / 578.78' S= 0.1962 '/' Cc= 0.900 n= 0.013, Flow Area= 0.79 sf
#2	Device 1	599.00'	**0.625" Vert. Orifice/Grate** C= 0.600 Limited to weir flow at low heads
#3	Device 1	605.00'	**4.000" Vert. Orifice/Grate** C= 0.600 Limited to weir flow at low heads
#4	Device 1	607.75'	**0.50' long x 0.75' rise Weir Notch** Cv= 2.62 (C= 3.28)
#5	Device 1	608.50'	**12.000" Horiz. Standpipe** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=1.66 cfs @ 9.06 hrs HW=608.41' (Free Discharge)
└─**1=Culvert** (Passes 1.66 cfs of 15.81 cfs potential flow)
  ├─**2=Orifice/Grate** (Orifice Controls 0.03 cfs @ 14.75 fps)
  ├─**3=Orifice/Grate** (Orifice Controls 0.76 cfs @ 8.67 fps)
  ├─**4=Weir Notch** (Weir Controls 0.87 cfs @ 2.65 fps)
  └─**5=Standpipe** (Controls 0.00 cfs)

Pond 1P: WestSide Pond



Summary for Subcatchment 35: Lot 35 Existing Conditions

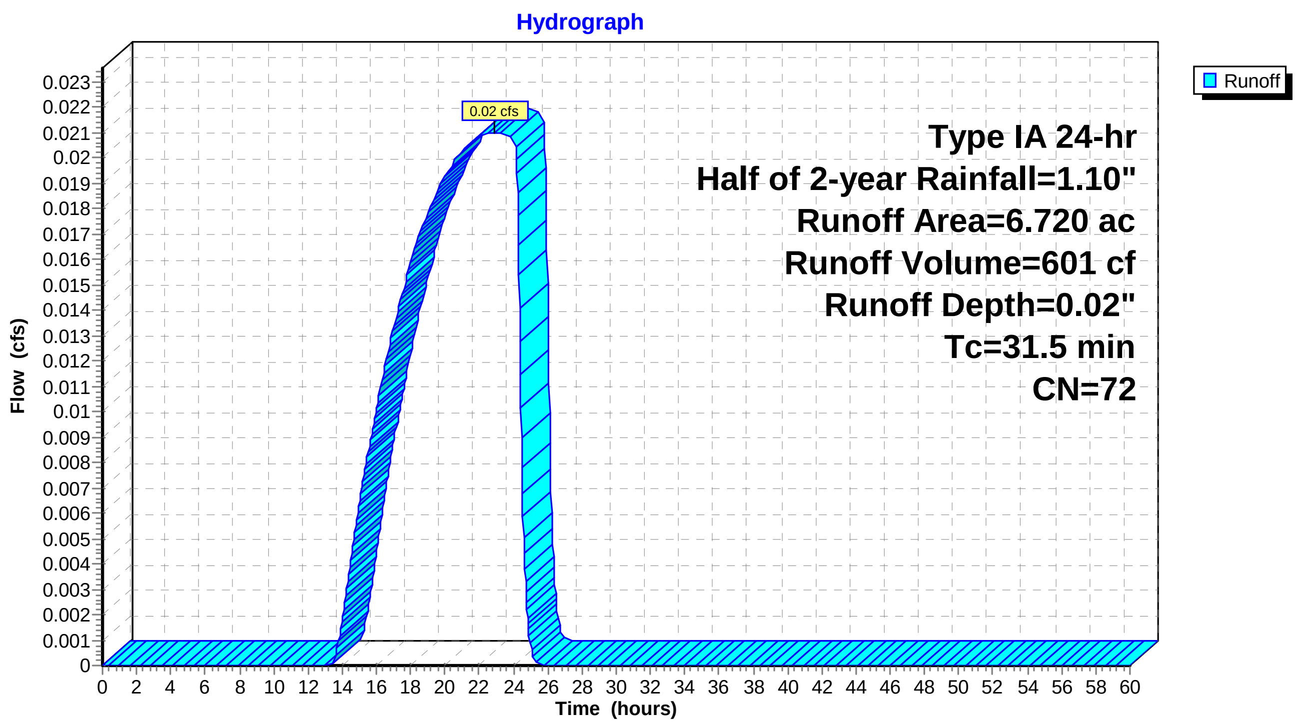
Runoff = 0.02 cfs @ 22.92 hrs, Volume= 601 cf, Depth= 0.02"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-60.00 hrs, dt= 0.03 hrs
Type IA 24-hr Half of 2-year Rainfall=1.10"

	Area (ac)	CN	Description
*	6.720	72	City of Salem Pre-developed, HSG C
	6.720		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
31.5					**Direct Entry, TR-55 Worksheet**

Subcatchment 35: Lot 35 Existing Conditions



Summary for Subcatchment 38S: Undetained

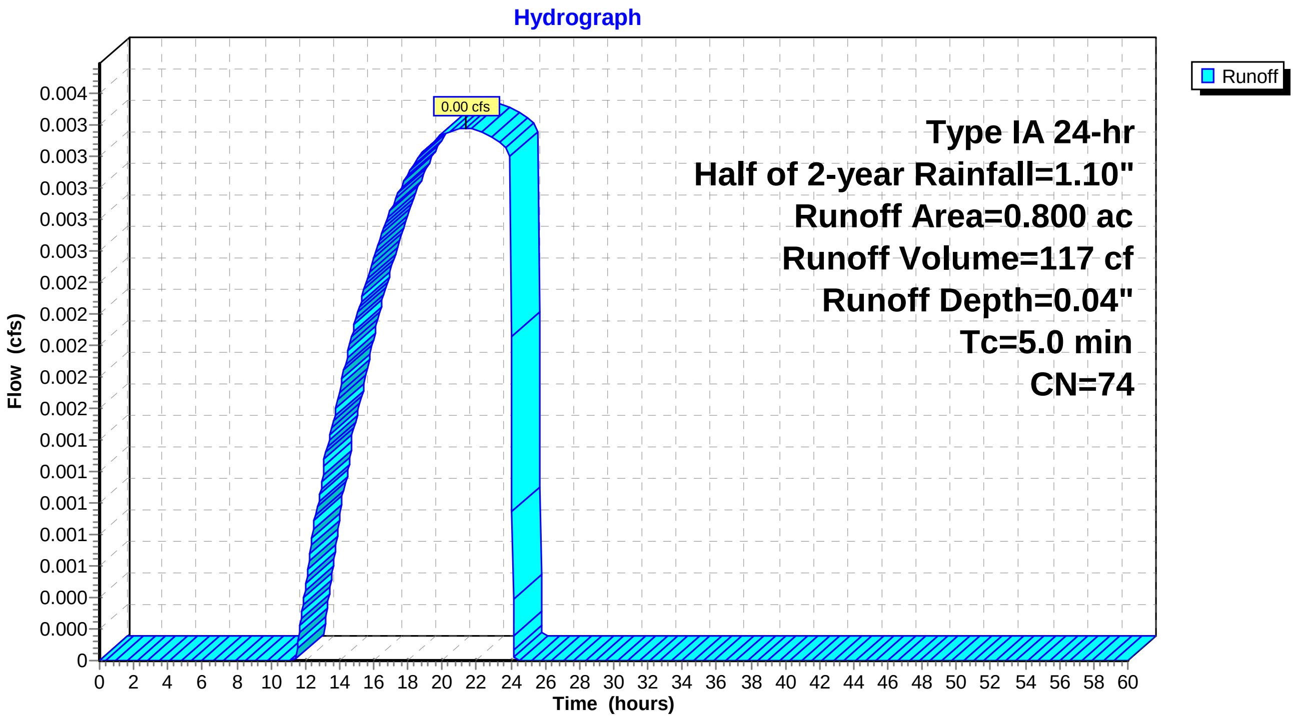
Runoff = 0.00 cfs @ 21.35 hrs, Volume= 117 cf, Depth= 0.04"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-60.00 hrs, dt= 0.03 hrs
Type IA 24-hr Half of 2-year Rainfall=1.10"

Area (ac)	CN	Description
0.800	74	>75% Grass cover, Good, HSG C
0.800		100.00% Pervious Area

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

Subcatchment 38S: Undetained



Summary for Subcatchment 39S: Developed Conditions

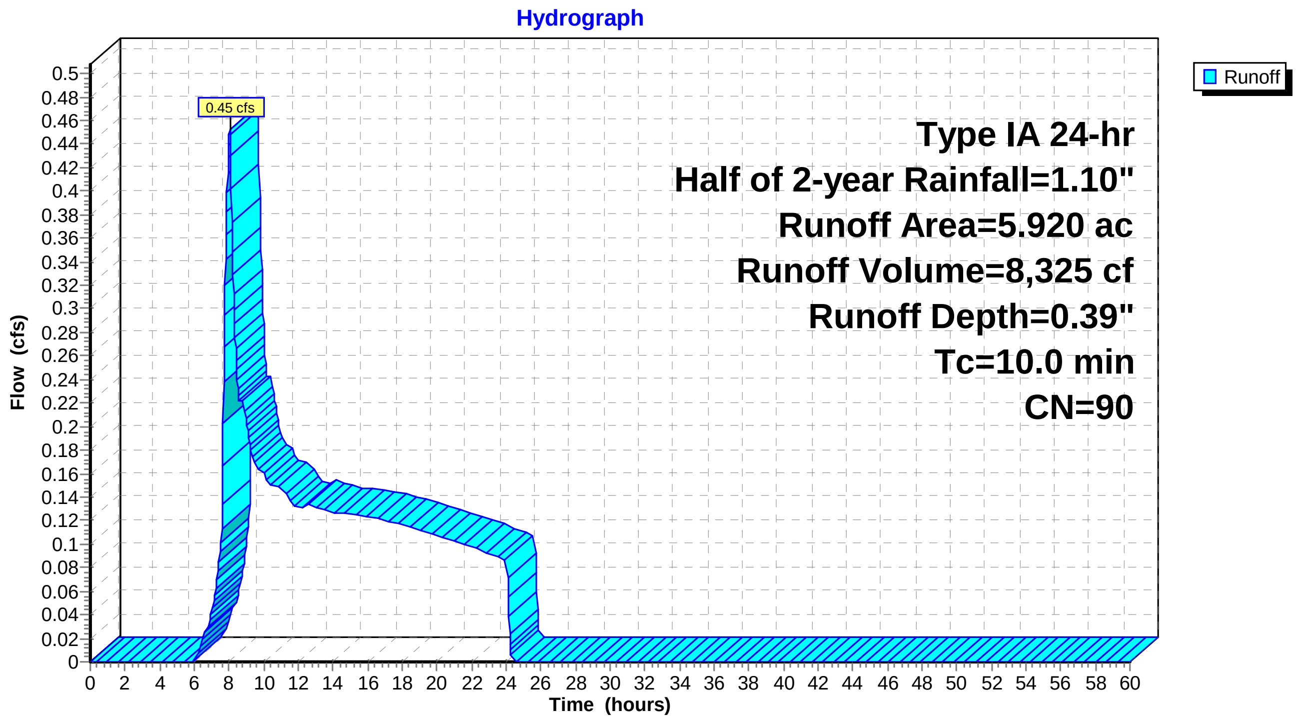
Runoff = 0.45 cfs @ 8.04 hrs, Volume= 8,325 cf, Depth= 0.39"
Routed to Pond 1P : WestSide Pond

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-60.00 hrs, dt= 0.03 hrs
Type IA 24-hr Half of 2-year Rainfall=1.10"

Area (ac)	CN	Description
4.032	98	Unconnected pavement, HSG C
1.888	74	>75% Grass cover, Good, HSG C
5.920	90	Weighted Average
1.888		31.89% Pervious Area
4.032		68.11% Impervious Area
4.032		100.00% Unconnected

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

Subcatchment 39S: Developed Conditions



Summary for Pond 1P: WestSide Pond

Inflow Area = 257,875 sf, 68.11% Impervious, Inflow Depth = 0.39" for Half of 2-year event
Inflow = 0.45 cfs @ 8.04 hrs, Volume= 8,325 cf
Outflow = 0.02 cfs @ 24.18 hrs, Volume= 4,472 cf, Atten= 95%, Lag= 968.6 min
Primary = 0.02 cfs @ 24.18 hrs, Volume= 4,472 cf

Routing by Stor-Ind method, Time Span= 0.00-60.00 hrs, dt= 0.03 hrs
Peak Elev= 604.80' @ 24.18 hrs Surf.Area= 3,874 sf Storage= 6,912 cf

Plug-Flow detention time= 1,409.2 min calculated for 4,472 cf (54% of inflow)
Center-of-Mass det. time= 1,167.0 min (2,022.9 - 855.8)

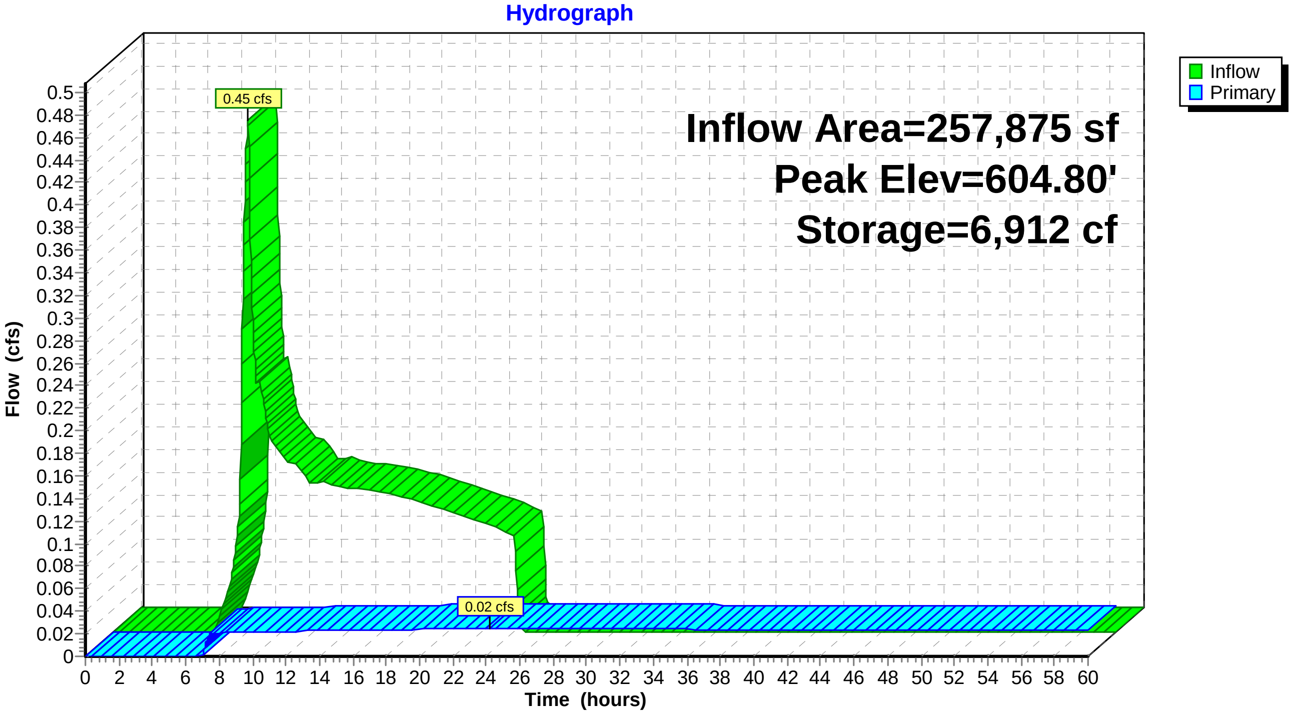
Volume	Invert	Avail.Storage	Storage Description
#1	601.99'	28,817 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)
#2	595.00'	255 cf	**5.00'D x 13.00'H Vertical Cone/Cylinder**
		29,073 cf	Total Available Storage

Elevation (feet)	Surf.Area (sq-ft)	Voids (%)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
601.99	1	0.0	0	0
602.99	3,591	0.0	0	0
603.00	3,591	100.0	36	36
604.00	3,735	100.0	3,663	3,699
605.00	3,885	100.0	3,810	7,509
606.00	4,035	100.0	3,960	11,469
607.00	4,186	100.0	4,111	15,579
608.00	4,337	100.0	4,262	19,841
609.00	4,488	100.0	4,413	24,253
610.00	4,640	100.0	4,564	28,817

Device	Routing	Invert	Outlet Devices
#1	Primary	598.50'	**12.000" Round Culvert** L= 100.5' RCP, rounded edge headwall, Ke= 0.100 Inlet / Outlet Invert= 598.50' / 578.78' S= 0.1962 '/' Cc= 0.900 n= 0.013, Flow Area= 0.79 sf
#2	Device 1	599.00'	**0.625" Vert. Orifice/Grate** C= 0.600 Limited to weir flow at low heads
#3	Device 1	605.00'	**4.000" Vert. Orifice/Grate** C= 0.600 Limited to weir flow at low heads
#4	Device 1	607.75'	**0.50' long x 0.75' rise Weir Notch** Cv= 2.62 (C= 3.28)
#5	Device 1	608.50'	**12.000" Horiz. Standpipe** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=0.02 cfs @ 24.18 hrs HW=604.80' (Free Discharge)
- **1=Culvert** (Passes 0.02 cfs of 12.42 cfs potential flow)
  - **2=Orifice/Grate** (Orifice Controls 0.02 cfs @ 11.57 fps)
  - **3=Orifice/Grate** (Controls 0.00 cfs)
  - **4=Weir Notch** (Controls 0.00 cfs)
  - **5=Standpipe** (Controls 0.00 cfs)

Pond 1P: WestSide Pond



Appendix E

Routing Diagram for 20230321 West Apts
Prepared by Multi/Tech Engineering Service, Printed 5/15/2023

Rainfall Events Listing (selected events)

Event#	Event Name	Storm Type	Curve	Mode	Duration (hours)	B/B	Depth (inches)	AMC
1	WQ	Type IA 24-hr		Default	24.00	1	1.38	2

Summary for Subcatchment 11S: Developed Conditions

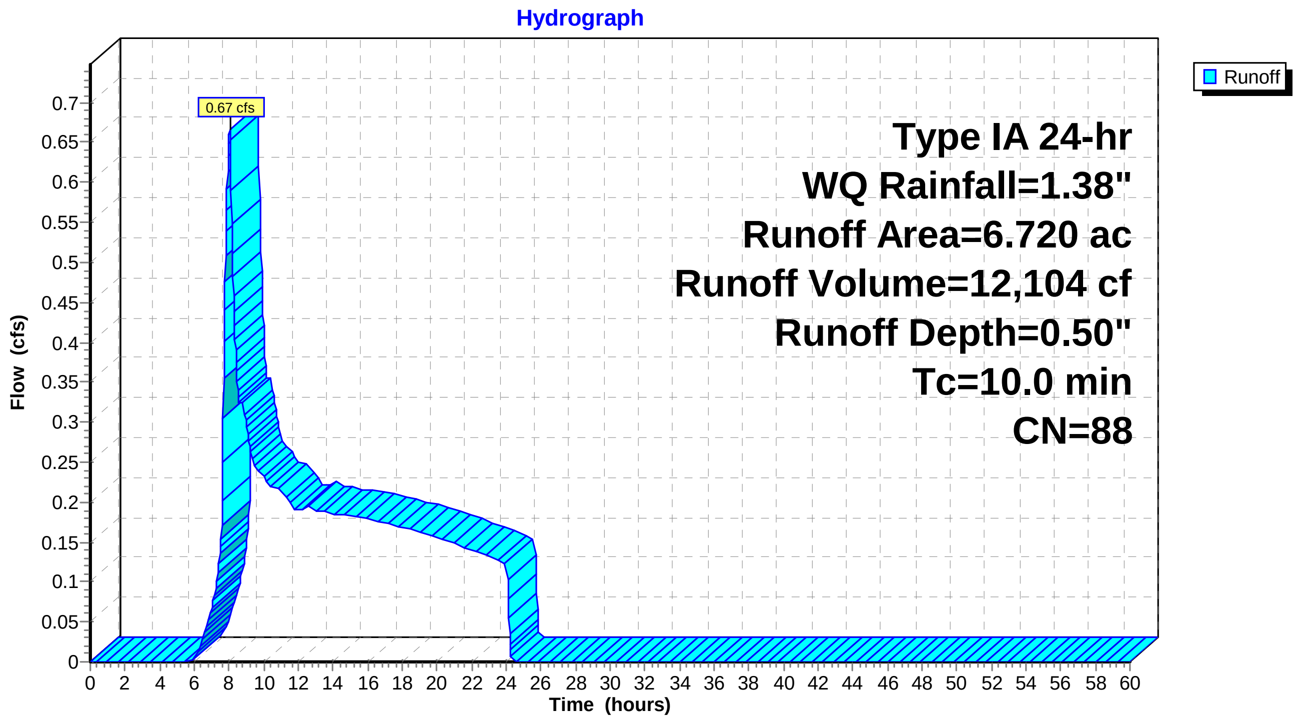
Runoff = 0.67 cfs @ 8.04 hrs, Volume= 12,104 cf, Depth= 0.50"
Routed to Pond 1P : WestSide Pond

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-60.00 hrs, dt= 0.03 hrs
Type IA 24-hr WQ Rainfall=1.38"

Area (ac)	CN	Description
4.032	98	Unconnected pavement, HSG C
2.688	74	>75% Grass cover, Good, HSG C
6.720	88	Weighted Average
2.688		40.00% Pervious Area
4.032		60.00% Impervious Area
4.032		100.00% Unconnected

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

Subcatchment 11S: Developed Conditions



Summary for Subcatchment A1: Lots 2-34

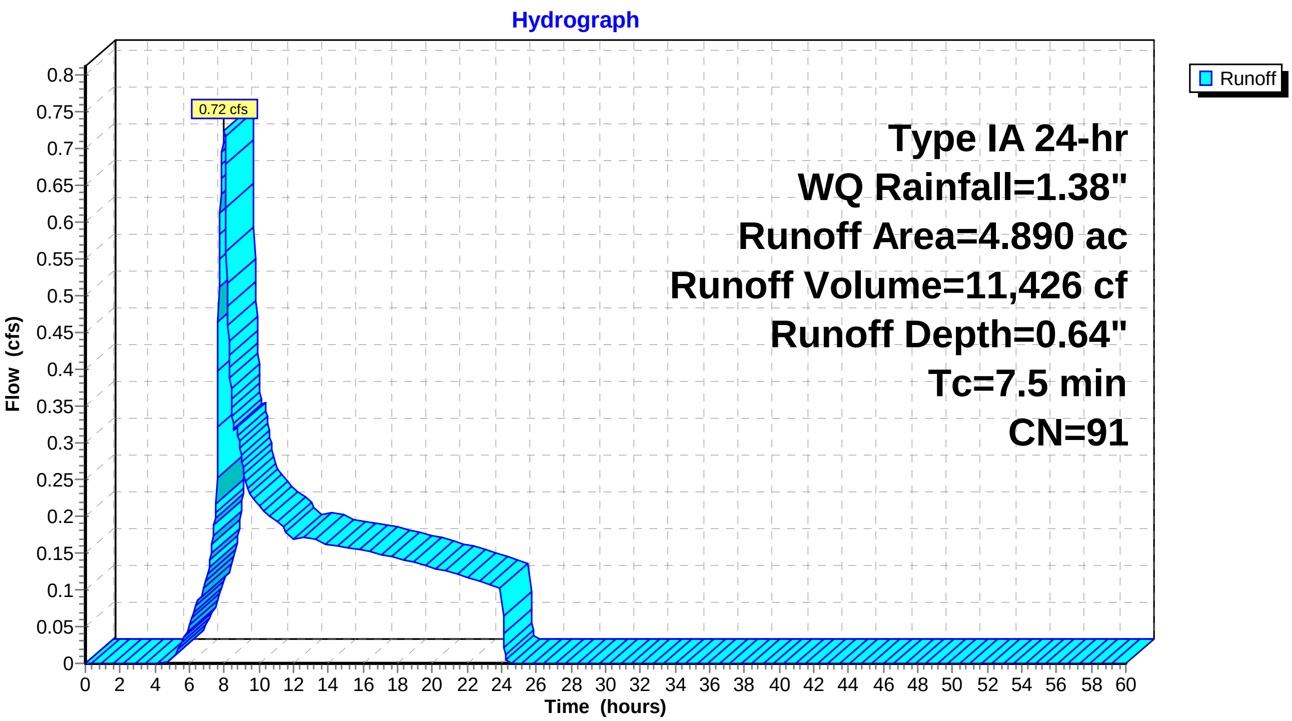
Runoff = 0.72 cfs @ 8.00 hrs, Volume= 11,426 cf, Depth= 0.64"
Routed to Pond WQ : Water Quality

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-60.00 hrs, dt= 0.03 hrs
Type IA 24-hr WQ Rainfall=1.38"

	Area (ac)	CN	Description
*	4.890	91	Lots 2-34
	4.890		100.00% Pervious Area

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

Subcatchment A1: Lots 2-34



Summary for Pond 1P: WestSide Pond

Inflow Area = 292,723 sf, 60.00% Impervious, Inflow Depth = 0.50" for WQ event
Inflow = 0.67 cfs @ 8.04 hrs, Volume= 12,104 cf
Outflow = 0.10 cfs @ 24.09 hrs, Volume= 4,726 cf, Atten= 85%, Lag= 962.9 min
Primary = 0.10 cfs @ 24.09 hrs, Volume= 4,726 cf
Routed to Pond WQ : Water Quality

Routing by Stor-Ind method, Time Span= 0.00-60.00 hrs, dt= 0.03 hrs
Peak Elev= 605.68' @ 24.09 hrs Surf.Area= 4,064 sf Storage= 10,537 cf

Plug-Flow detention time= 1,427.0 min calculated for 4,726 cf (39% of inflow)
Center-of-Mass det. time= 1,120.6 min (1,973.5 - 853.0)

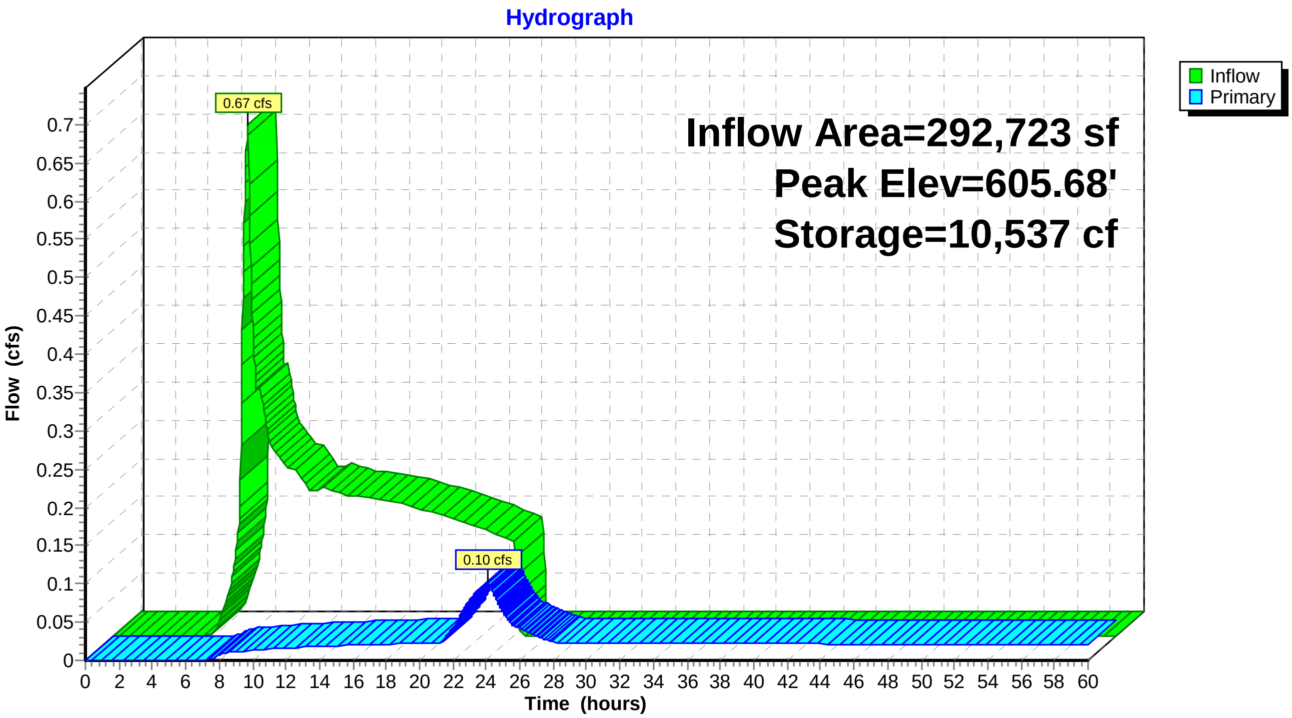
Volume	Invert	Avail.Storage	Storage Description
#1	601.99'	29,109 cf	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (sq-ft)	Voids (%)	Inc.Store (cubic-feet)	Cum.Store (cubic-feet)
601.99	1	0.0	0	0
602.99	3,775	0.0	0	0
603.00	3,775	100.0	38	38
604.00	3,883	100.0	3,829	3,867
605.00	3,991	100.0	3,937	7,804
606.00	4,099	100.0	4,045	11,849
607.00	4,207	100.0	4,153	16,002
608.00	4,315	100.0	4,261	20,263
609.00	4,423	100.0	4,369	24,632
610.00	4,531	100.0	4,477	29,109

Device	Routing	Invert	Outlet Devices
#1	Primary	598.50'	**12.000" Round Culvert** L= 100.5' RCP, rounded edge headwall, Ke= 0.100 Inlet / Outlet Invert= 598.50' / 578.78' S= 0.1962 '/' Cc= 0.900 n= 0.013, Flow Area= 0.79 sf
#2	Device 1	603.00'	**0.750" Vert. Orifice/Grate** C= 0.600 Limited to weir flow at low heads
#3	Device 1	605.50'	**4.750" Vert. Orifice/Grate** C= 0.600 Limited to weir flow at low heads
#4	Device 1	607.75'	**0.42' long x 0.75' rise Weir Notch** Cv= 2.62 (C= 3.28)
#5	Device 1	608.50'	**12.000" Horiz. Standpipe** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=0.10 cfs @ 24.09 hrs HW=605.68' (Free Discharge)
└─**1=Culvert** (Passes 0.10 cfs of 13.33 cfs potential flow)
  ├─**2=Orifice/Grate** (Orifice Controls 0.02 cfs @ 7.83 fps)
  ├─**3=Orifice/Grate** (Orifice Controls 0.08 cfs @ 1.44 fps)
  ├─**4=Weir Notch** (Controls 0.00 cfs)
  └─**5=Standpipe** (Controls 0.00 cfs)

Pond 1P: WestSide Pond



Summary for Pond WQ: Water Quality

Inflow Area = 505,732 sf, 34.73% Impervious, Inflow Depth > 0.38" for WQ event
Inflow = 0.73 cfs @ 8.00 hrs, Volume= 16,152 cf
Outflow = 0.17 cfs @ 17.17 hrs, Volume= 16,147 cf, Atten= 77%, Lag= 550.4 min
Discarded = 0.17 cfs @ 17.17 hrs, Volume= 16,147 cf
Primary = 0.00 cfs @ 0.00 hrs, Volume= 0 cf

Routing by Stor-Ind method, Time Span= 0.00-60.00 hrs, dt= 0.03 hrs
Peak Elev= 542.39' @ 17.17 hrs Surf.Area= 3,641 sf Storage= 2,338 cf
Flood Elev= 549.00' Surf.Area= 3,792 sf Storage= 7,397 cf

Plug-Flow detention time= 152.1 min calculated for 16,139 cf (100% of inflow)
Center-of-Mass det. time= 151.4 min (1,304.0 - 1,152.6)

Volume	Invert	Avail.Storage	Storage Description
#1	541.74'	7,397 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)
541.74	3,581	0.0	0	0	3,581
541.75	3,581	100.0	36	36	3,583
542.75	3,675	100.0	3,628	3,664	3,816
543.75	3,792	100.0	3,733	7,397	4,063

Device	Routing	Invert	Outlet Devices
#1	Discarded	541.74'	**2.000 in/hr Exfiltration over Surface area**
#2	Primary	542.50'	**24.000" Horiz. Orifice/Grate** C= 0.600 Limited to weir flow at low heads

Discarded OutFlow Max=0.17 cfs @ 17.17 hrs HW=542.39' (Free Discharge)
↑—**1=Exfiltration** (Exfiltration Controls 0.17 cfs)

Primary OutFlow Max=0.00 cfs @ 0.00 hrs HW=541.74' (Free Discharge)
↑—**2=Orifice/Grate** (Controls 0.00 cfs)

Pond WQ: Water Quality