WCE

Whipple Consulting Engineers, Inc.

June 10, 2021
W.O. No. 2020-2744

City of Spokane
808 W Spokane Falls Blvd #3343
Spokane, WA 99201

Attn: Tami Palmquist. AICP, CFM

RE: Concept Storm Drainage Report for Nexcore Assisted Living & Memory Care
B21M0033PDEV

Dear Tami,

This letter is intended to provide an overview of the potential impacts from storm drainage created by the proposed development of 53rd & Regal with parcel #34032.0494.

The proposed project is zoned as Centers & Corridor/Downtown center (CC2-DC). The site is located within City of Spokane and lies within NW ¼, SEC. 03, T. 24, R. 43E., W.M. The site is located southeast of the intersection of Regal Street & 53rd Avenue.

As stated in the Geotech Report completed by Allwest, the soils onsite mostly consisted of topsoil and native fine-grained loess, coarse grained alluvium, and broken basalt bedrock. In addition, Allwest does not recommend stormwater infiltration with drywells. This is due to the shallow proximity to the limiting layers of bedrock. Bedrock was encountered at depths ranging between 0.5 feet and 4.5 feet below existing ground.

Currently, the stormwater generated onsite generally sheet flows from the southeast to the northwest of the project site until it leaves the site and enters the City of Spokane public stormwater system.

## Table 1 – Overall Pre-Basin Summary

| Basin | Total Basin Area (sf) | Total Basin Area (ac) | Impervious Area (sf) | Impervious Area (ac) | Pervious Area (sf) | Pervious Area (ac) |
|---|---|---|---|---|---|---|
| **Pre-Onsite** | 400,333 | 9.19 | 2,500 | 0.06 | 397,833 | 9.13 |
| **Pre-Offsite** | 54,410 | 1.25 | 41,124 | 0.94 | 13,286 | 0.31 |
| **Total** | **454,743** | **10.44** | **43,624** | **1.00** | **411,119** | **9.44** |

*Refer to basin calculations in Appendix for areas and peak flows for all basins.

In the overall post basin, the increase in stormwater and PGIS on the various roads will be treated/stored within the onsite bio-retention ponds. It is assumed that the future commercial buildings west of the onsite north-south drive aisle will have a separate, but similar storm drainage system. In the case that stormwater overflows the onsite ponds, the stormwater will discharge into the Hazels Creek Storm Basin as allowed per the Hazels Creek Sub-Basin Planning & Schematic Design report completed by CH2MHill.

In order to ensure that any stormwater overflowing the onsite pond will be release at the allotted rate, the stormwater will be equipped with a properly sized overflow structure. The overflow structure requires that the stormwater entering the structure to crest the rim of the structure and filling the structure for release via an orifice. The orifice will be size appropriate to allow stormwater to be released at the maximum discharge rate. Please see below for the allotted discharge rate from the Hazels Creek Sub-Basin Planning & Schematic Design report completed by CH2MHill.Please see the attached report for details.

Per the Hazels Creek Sub-Basin Planning & Schematic Design report completed by CH2MHill, the site is allotted to discharge at a rate of 1.5 gallon per minute per acre. This will be considered as the pre-development flow rate of the site which when calculated, results in a rate of 0.0349 cfs:

$$1.5\,\frac{\frac{gallon}{minute}}{acre} \times 10.44\,acre = 12.66195\,\frac{gallon}{minute} \times \frac{0.133681\,cubic\,feet}{1.0\,gallon} \times \frac{1\,minute}{60\,seconds} = 0.0349\,cfs$$

Though the project site is not anticipated to encounter any stormwater issues, but in the event that there are heavy rain periods, the option to discharge to Hazel Creek basin provides assurance that the project site will not inundate and negatively impact the surrounding properties with excess stormwater.

## Table 2 – Overall Post Basin Summary

| Basin | Total Basin Area (sf) | Total Basin Area (ac) | Impervious Area (sf) | Impervious Area (ac) | Pervious Area (sf) | Pervious Area (ac) |
|---|---|---|---|---|---|---|
| **A** | 111,432 | 2.56 | 77,052 | 1.77 | 34,380 | 0.79 |
| **B** | 263,379 | 6.05 | 121,574 | 2.79 | 141,804 | 3.25 |
| **Offsite** (Regal/53rd /55th /Fiske) | 79,932 | 1.83 | 56,178 | 1.3 | 23,754 | 0.55 |
| **Total Overall Site** | **454,743** | **10.44** | **254,804** | **5.85** | **199,938** | **4.59** |

*Refer to basin calculations in Appendix for areas and peak flows for all basins.

Within the overall Post Basin, there are new proposed onsite features and several offsite areas that are part of the public road system and are existing. These offsite areas include the existing Regal St, Fiske St, 53rd & 55th Avenue. The stormwater generated on these areas will continue to flow, be treated, and will infiltrate underground as it did in pre-developed conditions. As part of the frontage improvements, 55th Avenue will have roadside swales to treat, store, and infiltrate the stormwater currently generated by the existing asphalt as required by the SRSM. In order to determine and confirm that the additional proposed site features will not burden the existing storm drainage system that discharges to Hazel's Creek, the onsite ponds for Post Basin A & B have been analyzed for treatment volumes and storage. Basins Post A & B have been analyzed as they are the two largest, contributing basins to the Hazel's Creek drainage system.

Post Basin A, which makes up the western 1/3 of the parcel, consists of future commercial buildings, drive aisles, and parking lots. Post Basin B, which, makes up the western 2/3 of the parcel, consists of the senior assisted living building, drive aisles, and parking lots. The stormwater for both basins will gutter flow to the local low point, where it will enter a catch basin and be conveyed via storm pipes to the onsite ponds. The onsite ponds will be sized deep enough to store the 25-year storm while infiltrating per the rate of $1.54\text{x } 10^{-4}$ as provided in the Geotech Report completed by Allwest. If we were to consider this infiltration rate applied to the total pond bottom areas (Senior Living - 3,476 sf & Lt Commercial – 2,650) over a 24-hr storm, the volume of stormwater that would be disposed of through infiltration would be approximately 46,250 cf of stormwater. This amount is significantly greater than the calculated required storage for the proposed development. The onsite Post Basins were also analyzed for storage via the Bowstring Method. Post Basin A requires 1,629 CF of storage and Post Basin B requires 4,621 CF of storage for the 25-yr storm. The required storage volume analyzed and is significantly less than the potential amount of stormwater infiltrated per the rate provided by the geotech. This infiltration rate has also been utilized as an outflow rate in the Bowstring Calculations. This provides further assurance that the site should not experience any stormwater issue. Please see below and the Appendix for further details on the Bowstring Analysis that was performed. If the stormwater level crests over the top of the pond and overflow, the stormwater will overflow into an overflow structure and enter the City's public stormwater system at a controlled rate to alleviate any flooding potential.

It is assumed that this method of conveying and treating the stormwater will be uniform for all basins & onsite ponds, please see Tables 3 for the Bowstring Analysis of Post Basins 1 & 2.

## Table 3 – Pre/Post Onsite Analysis

| Basin | Basin Size (SF) | Basin Size (AC) | Total PGIS Area (SF) | Required Treatment Vol. (CF) – 1815A | Provided Treatment Vol. (CF) | Required 25 Year Storage (CF)* | Provided 25-Year Storage (CF)* |
|---|---|---|---|---|---|---|---|
| **Pre-Overall** | 450,743 | 10.44 | 0 | 0 | 0 | 0 | 0 |
| **Post A** | 111,432 | 2.56 | 46,552 | 1,940 | 2,959 | 1,629 | 3,967 |
| **Post B** | 263,379 | 6.05 | 80,732 | 3,364 | 3,830 | 4,621 | 16,798 |

As summarized in Tables 3, the swales provide the required treatment volume and storage volume required for the 25-year storm for the amount of PGIS proposed. The amount of Treatment Volume required was calculated using Equation 6-1d per the SRSM for conservatism due to soil type and infiltration rate provided by the geotech report for the project site. Please see attached Weighted C calculations, pond volume calculations, and basin maps in the Appendix for more information.

As we have analyzed the two largest, contributing basins to the City of Spokane's public storm system, we therefore; believe that this proposed commercial projects will meet the intent of the SRSM with the onsite ponds and not burden the public's storm system.

***Designer's Note: This analysis was completed under the assumption that method for treating, storing, and infiltrating stormwater for Post Basin A & Post Basin B will be consistent. The location, size, and shape for onsite pond for Basin A and the other onsite features (buildings, parking lots, and storm drainage structures) have not been fully designed and therefore any analysis cannot be deemed accurate or final. These site elements are shown purely for representation only.***

Should you have any questions or concerns related to this issue do not hesitate to call @ 509-893-2617

Sincerely,

TODD R. WHIPPLE
STATE OF WASHINGTON
25462
REGISTERED
PROFESSIONAL ENGINEER

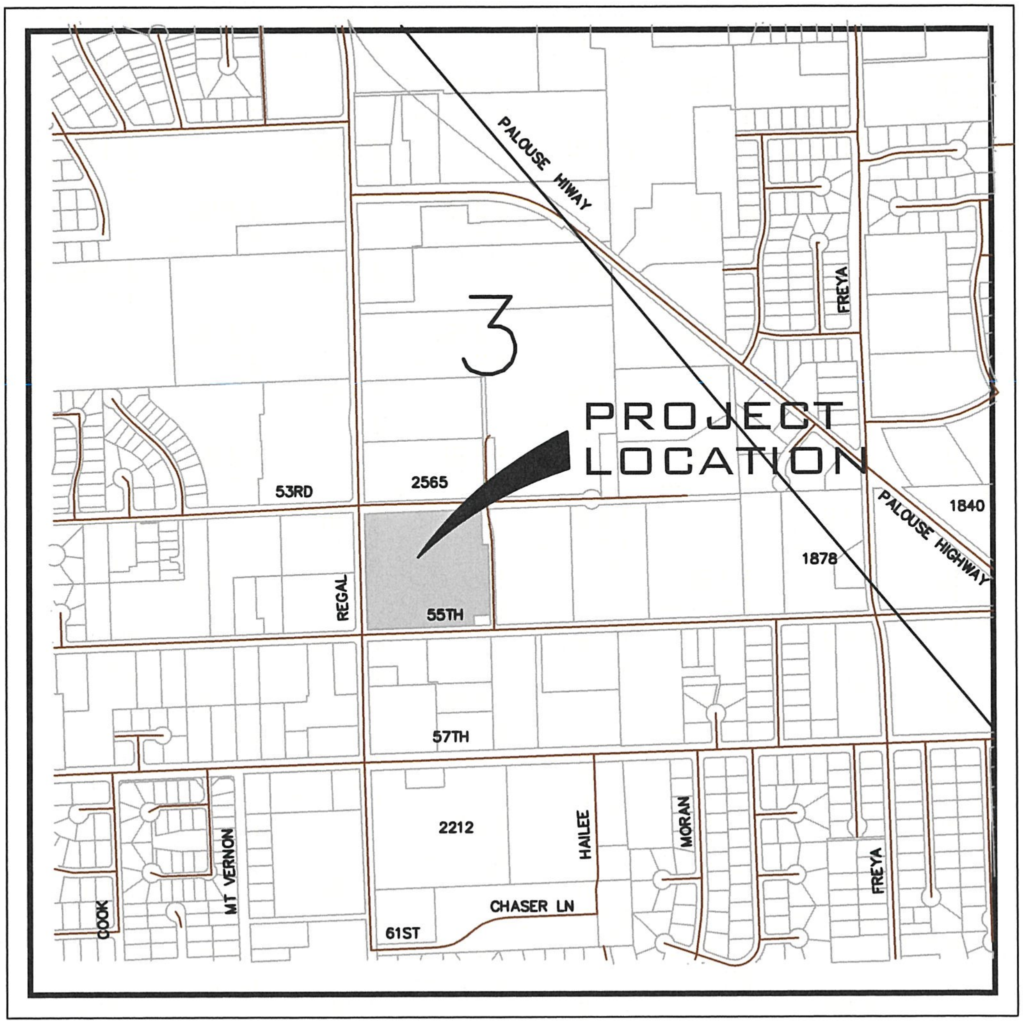
6/23/21

Todd R. Whipple P.E.

Enc. Basin Maps, Weighted C Calculations, Pond Calculations, Geotech Report

cc: File

# VICINITY MAP




## VICINITY MAP

N

W E

S

NOT TO SCALE

| | | |
|---|---|---|
| PROJ #: 20-2744<br>DATE: 06/01/21<br>DRAWN: STT<br>APPROVED: TRW | CONCEPT DRAINAGE REPORT<br>**53RD & REGAL BSP**<br>53RD & REGAL ST<br>SPOKANE, WASHINGTON | WCE<br>WHIPPLE CONSULTING ENGINEERS<br>21 SOUTH PINES ROAD<br>SPOKANE VALLEY, WASHINGTON 99206<br>PH: 509-893-2617 FAX: 509-926-0227 |
| FIGURE 1 | VICINITY MAP | |

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# BASIN MAPS

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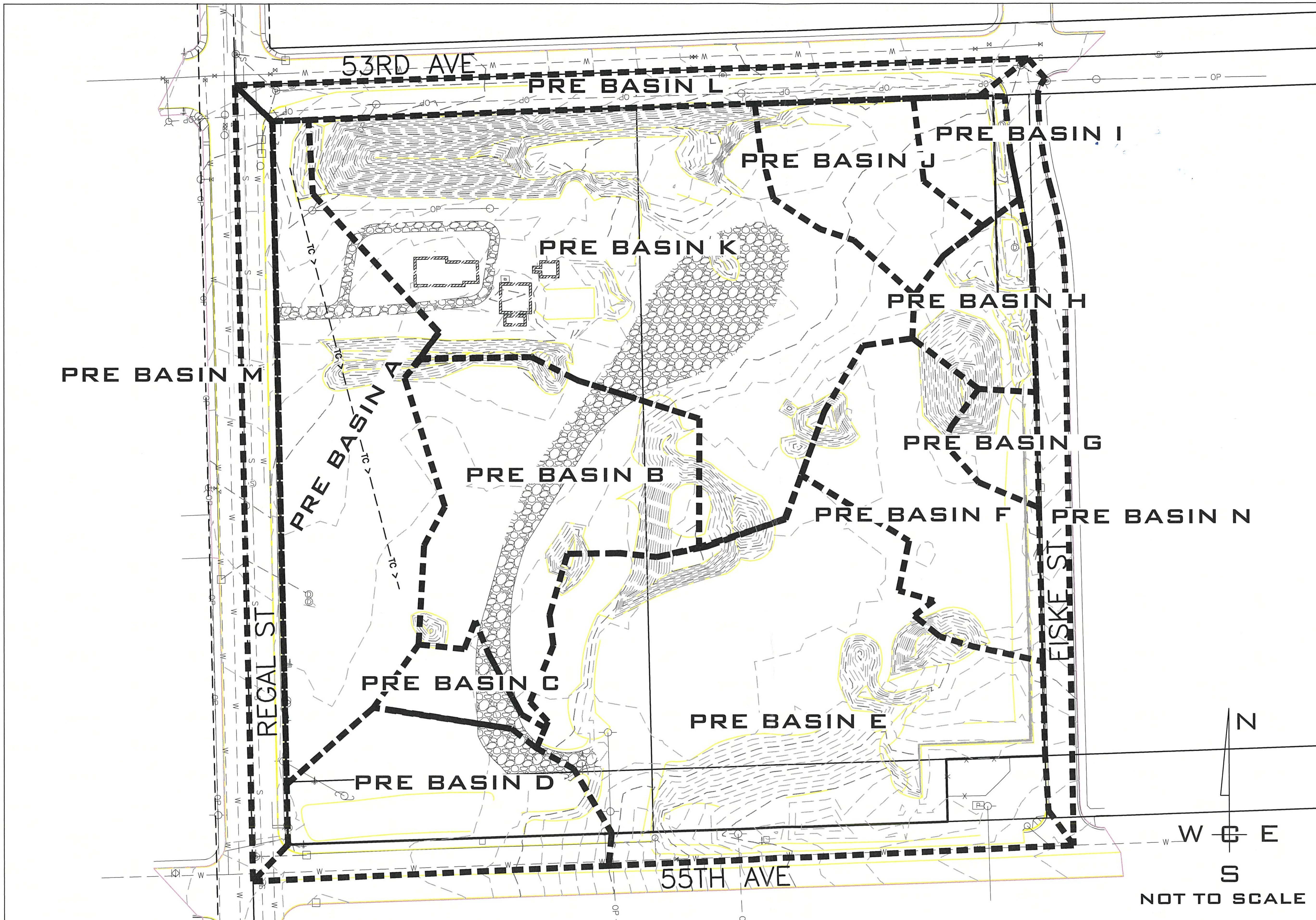
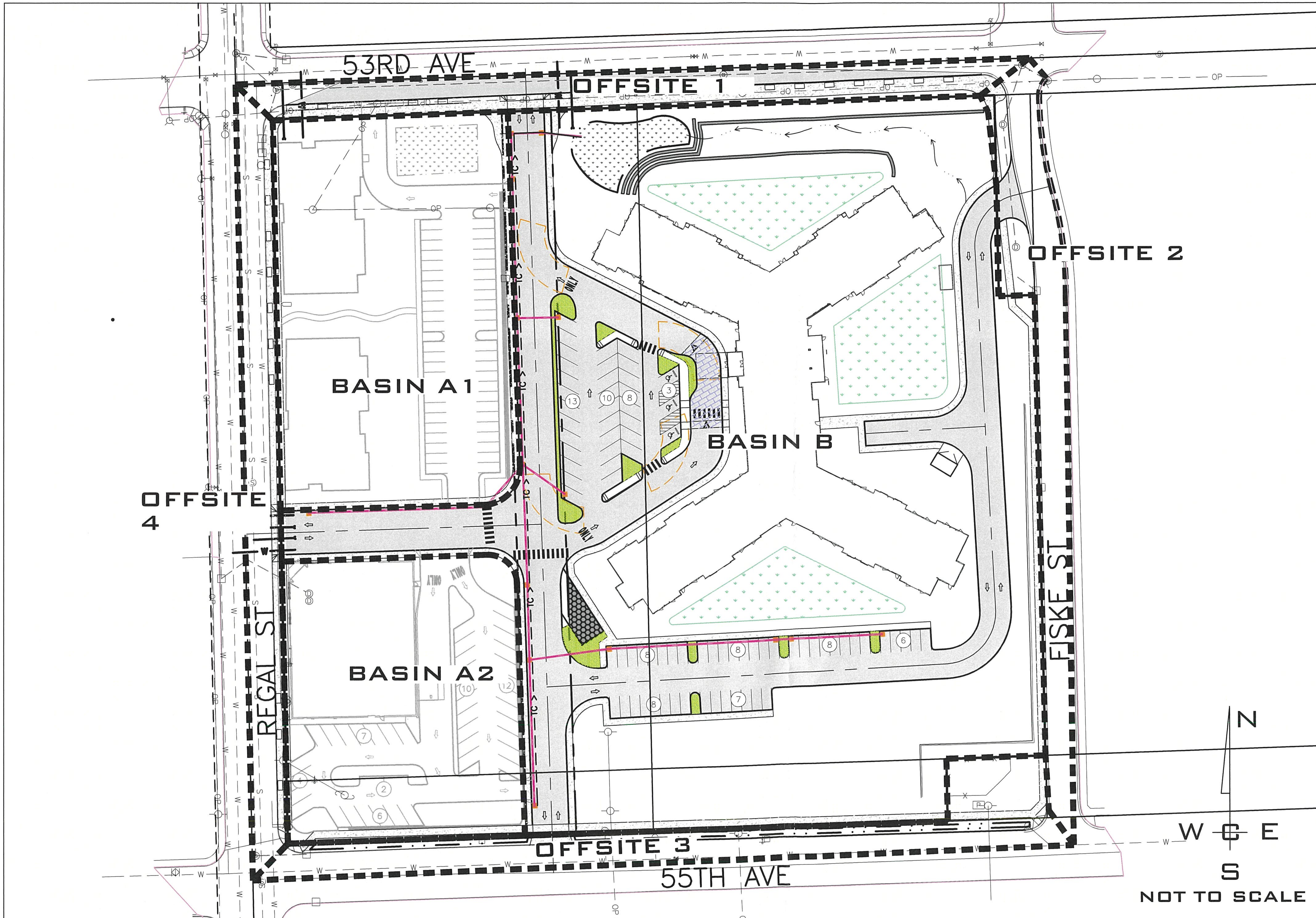



WCE
WHIPPLE CONSULTING ENGINEERS
CIVIL, STRUCTURAL AND
TRANSPORTATION ENGINEERING
21 SOUTH PINES ROAD
SPOKANE VALLEY, WASHINGTON 99206
PH: 509-893-2617 FAX: 509-926-0227

PROJ #: 20-2744
DATE: 06/01/21
DRAWN: STT
APPROVED: TRW

PRE-DEVELOPMENT BASIN MAP
53RD & REGAL BSP
REGAL ST & 53RD AVE
SPOKANE, WASHINGTON

SHEET
1
OF
2

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PROJ #: 20-2744
DATE: 06/01/21
DRAWN: STT
APPROVED: TRW

# POST-DEVELOPMENT BASIN MAP
## 53RD & REGAL BSP
REGAL ST & 53RD AVE
SPOKANE, WASHINGTON

SHEET
2
OF
2

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# BASIN SUMMARY SHEET

Whipple Consulting Engineers
Basin Calculation Worksheet

| | | | |
|---|---|---|---|
| Imp | 0.9 | | |
| Per | 0.15 | | |

Intensities from SRSM eqn. 5-13, per Table 5-7, Assumes Tc = 5 min

| | | | | | |
|---|---|---|---|---|---|
| I (2 yr) = | 1.418 inches | I (10 yr)= | 2.619 inches | NOTE: | |
| I (25 yr) = | 3.319 inches | I (50 yr)= | 3.843 inches | | |
| I (100 yr) = | 4.381 inches | | | | |

| | WCE No. | Project Name |
|---|---|---|
| 6/1/2021 | 20-2744 | Nexcore |
| STT | | |

**SPOKANE COUNTY - SRSM - GRASSED PERCOLATION METHOD**

| | | | | | | | | | | 1815 A | | Q=CIA (cfs) | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Basin | Total sf | Access/Parking /Street (sf) | Sidewalk sf | Adj. SW sf | Buildings sf | Total Impervious | Total Pervious | Weighted "C" | PGIS sf | Pond Area (sf) | Pond Vol (cf) | 2 yr | 10 yr | 25 yr | 50 yr | 100 yr |
| **PRE A** | 50,592 | 0 | 0 | 0 | 0 | 0 | 50,592 | 0.15 | 0 | 0.00 | 0.00 | 0.25 | 0.46 | 0.58 | 0.67 | 0.76 |
| **PRE B** | 44,197 | 0 | 0 | 0 | 0 | 0 | 44,197 | 0.15 | 0 | 0.00 | 0.00 | 0.22 | 0.40 | 0.51 | 0.58 | 0.67 |
| **PRE C** | 6,162 | 0 | 0 | 0 | 0 | 0 | 6,162 | 0.15 | 0 | 0.00 | 0.00 | 0.03 | 0.06 | 0.07 | 0.08 | 0.09 |
| **PRE D** | 30,052 | 0 | 0 | 0 | 0 | 0 | 30,052 | 0.15 | 0 | 0.00 | 0.00 | 0.15 | 0.27 | 0.34 | 0.40 | 0.45 |
| **PRE E** | 98,981 | 0 | 0 | 0 | 0 | 0 | 98,981 | 0.15 | 0 | 0.00 | 0.00 | 0.48 | 0.89 | 1.13 | 1.31 | 1.49 |
| **PRE F** | 28,862 | 0 | 0 | 0 | 0 | 0 | 28,862 | 0.15 | 0 | 0.00 | 0.00 | 0.14 | 0.26 | 0.33 | 0.38 | 0.44 |
| **PRE G** | 5,516 | 0 | 0 | 0 | 0 | 0 | 5,516 | 0.15 | 0 | 0.00 | 0.00 | 0.03 | 0.05 | 0.06 | 0.07 | 0.08 |
| **PRE H** | 11,537 | 0 | 0 | 0 | 0 | 0 | 11,537 | 0.15 | 0 | 0.00 | 0.00 | 0.06 | 0.10 | 0.13 | 0.15 | 0.17 |
| **PRE I** | 7,100 | 0 | 0 | 0 | 0 | 0 | 7,100 | 0.15 | 0 | 0.00 | 0.00 | 0.03 | 0.06 | 0.08 | 0.09 | 0.11 |
| **PRE J** | 17,533 | 0 | 0 | 0 | 0 | 0 | 17,533 | 0.15 | 0 | 0.00 | 0.00 | 0.09 | 0.16 | 0.20 | 0.23 | 0.26 |
| **PRE K** | 99,801 | 0 | 0 | 0 | 2,500 | 2,500 | 97,301 | 0.17 | 0 | 0.00 | 0.00 | 0.55 | 1.01 | 1.28 | 1.49 | 1.69 |
| **PRE L** | 19,019 | 6,378 | 0 | 0 | 0 | 6,378 | 12,641 | 0.40 | 6,378 | 531.50 | 265.75 | 0.25 | 0.46 | 0.58 | 0.67 | 0.77 |
| **PRE M** | 19,276 | 14,383 | 0 | 4,893 | 0 | 19,276 | 0 | 0.90 | 19,276 | 1,606.33 | 803.17 | 0.56 | 1.04 | 1.32 | 1.53 | 1.74 |
| **PRE N** | 16,115 | 15,470 | 0 | 0 | 0 | 15,470 | 645 | 0.87 | 15,470 | 1,289.17 | 644.58 | 0.46 | 0.84 | 1.07 | 1.24 | 1.41 |
| **PostTotal** | **454,743** | **36,231** | **0** | **4,893** | **2,500** | **43,624** | **411,119** | **0.22** | **41,124** | **3,427.00** | **1,713.50** | **3.29** | **6.07** | **7.69** | **8.90** | **10.15** |

| **POST BASIN** | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **A** | 111,432 | 41,952 | 0 | 4,600 | 30,500 | 77,052 | 34,380 | 0.67 | 46,552 | 3,879.33 | 1,939.67 | 2.43 | 4.48 | 5.68 | 6.57 | 7.49 |
| **B** | 263,379 | 75,732 | 0 | 5,000 | 40,842 | 121,574 | 141,804 | 0.50 | 80,732 | 6,727.68 | 3,363.84 | 4.25 | 7.86 | 9.96 | 11.53 | 13.14 |
| **OVERALL ONSITE TOTAL** | **374,811** | **117,684** | **0** | **9,600** | **71,342** | **198,626** | **176,185** | **0.55** | **127,284** | **10,607.02** | **5,303.51** | **6.68** | **12.34** | **15.63** | **18.10** | **20.64** |
| **OFFSITE (1-4)** | 79,932 | 32,478 | 0 | 23,700 | 0 | 56,178 | 23,754 | 0.68 | 56,178 | 4,681.50 | 2,340.75 | 1.76 | 3.25 | 4.12 | 4.77 | 5.44 |
| **OVERALL TOTAL** | **79,932** | **32,478** | **0** | **23,700** | **0** | **56,178.00** | **23,754.16** | **0.68** | **56,178.00** | **4,681.50** | **2,340.75** | **1.76** | **3.25** | **4.12** | **4.77** | **5.44** |

# POND VOLUME WORKSHEET

WHIPPLE CONSULTING ENGINEERS
POND VOLUME CALC SHEET
6/2/2021
20-2744 NEXCORE
Engineer stt

| Basins | Ponds/ Swales | Bottom Area sf | Squared Side lf | *Total Treatment Area sf | Pond Bottom Elevation at Drywell | Pond Outlet Elevation | Pond Inlet Elevation (avg) | Treatment: Conic Volume to Rim cf | Treatment: Side Slope Volume cf | Treatment: Total Volume to Rim cf | Storage: Conic Volume to Inlet cf | Storage: Side Slope Volume cf | Storage: Total Volume to Inlet cf |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | | | | |
| A | A | **2,650** | 51 | 3,005 | 1000.00 | 1001.00 | 1003.55 | 2,650 | 309 | **2,959** | 9,407 | 3,893 | **13,300** |
| B | B | **3,476** | 59 | 3,883 | 1000.00 | 1001.00 | 1001.30 | 3,476 | 354 | **3,830** | 4,519 | 598 | **5,117** |
| | | | | | | | | | | | | | |
| | | | | | | | | | | | | | |

# STORM EVENT BOWSTRING CALCULATIONS

PEAK FLOW CALCULATION PROJECT:

**25-Year Design Storm** LT COMM

BASIN: A

| | | | |
|---|---|---|---|
| Tot. Area | 111,432 SF | 2.56 Acres | |
| Imp. Area | 56,178 SF | C= | 0.9 |
| Perv. Area | 55,254 SF | C= | 0.15 |
| Wt. C = | 0.53 | PGIS Area = | 46,552 |

| WCE Applicable Travel Time Ground Cover Coefficients | | |
|---|---|---|
| Per Table 5-6 SRSM | | |
| Type of Cover | | K (ft/min) |
| Short Pasture | | 420 |
| Nearly Bare Ground | | 600 |
| Small Roadside Ditch/ Grass | | 900 |
| Paved Area (use for parking lots) | | 1200 |
| Gutter - 4 inches deep | | 1500 |
| Gutter - 6 inches deep | | 2400 |
| Pipe - 12-inch PVC/DI | | 3000 |
| Pipe - 15/18-inch PVC/DI | | 3900 |
| Pipe - 24-inch PVC/DI | | 4700 |

| Reaches | | |
|---|---|---|
| Reach 1 | Offsite | also applicable for Pre-Developed Tc |
| Length | 175.00 | |
| K | 420.00 | |
| Slope (ft/ft) | 0.0400 | be sure this is decimal equivalent slope 0.0000 |
| Travel Time | 2.08 | Minutes |
| | | |
| Reach 2 | Finished Lot from House to Street | |
| Length | 0.00 | |
| K | 420.00 | |
| Slope (ft/ft) | 0.0300 | be sure this is decimal equivalent slope 0.0000 |
| Travel Time | 0.00 | Minutes |
| | | |
| Reach 3 | Gutter Flow to Inlet/Catch Basin | |
| Length | 0.00 | |
| K | 2400.00 | |
| Slope (ft/ft) | 0.0200 | be sure this is decimal equivalent slope 0.0000 |
| Travel Time | 0.00 | Minutes |
| | | |
| Reach 4 | Pipe Flow | Pipe Reach One (only need one if no Dia change) |
| Length | 0.00 | |
| K | 3000.00 | 12-inch Pipe minimum |
| Slope (ft/ft) | 0.0200 | Average Slope for total pipe run |
| Travel Time | 0.00 | Minutes |
| | | |
| Reach 5 | Pipe Flow | Add additional pipe reacheds for other Dia |
| Length | 0.00 | |
| K | 3900.00 | 15/18-inch Pipe |
| Slope (ft/ft) | 0.0200 | Average Slope for total pipe run |
| Travel Time | 0.00 | Minutes |
| | | |
| Sum of Tc | 2.08 | Minutes |
| | | |
| Tc for Analysis | 5.00 | Minutes |

BOWSTRING METHOD PROJECT: LT COMM

DETENTION BASIN DESIGN BASIN: A

DESIGNER: STT

DATE: 2-Jun-21

| | |
|---|---|
| Time Increment (min) | 10 |
| Time of Conc. (min) | 5.00 |
| Outflow (cfs) | 0.70 |
| Design Year Flow | 25 |
| Area (acres) | 2.56 |
| Impervious Area (sq ft) | 56,178 |
| 'C' Factor | 0.53 |
| Area * C | 1.351 |
| PGIS Area | 46,552 |

| Time (min) | Time Inc. (sec) | Intens. (in/hr) | Q Devel. (cfs) | Vol.In (cu ft) | Vol.Out (cu ft) | Storage (cu ft) | |
|---|---|---|---|---|---|---|---|
| 5.00 | 300 | 3.32 | 4.48 | 1803 | 210 | 1593 | |
| 15 | 900 | 1.67 | 2.25 | 2259 | 630 | 1629 | <== |
| 25 | 1500 | 1.21 | 1.64 | 2623 | 1050 | 1573 | |
| 35 | 2100 | 0.98 | 1.33 | 2920 | 1470 | 1450 | |
| 45 | 2700 | 0.84 | 1.13 | 3175 | 1890 | 1285 | |
| 55 | 3300 | 0.74 | 1.00 | 3400 | 2310 | 1090 | |
| 65 | 3900 | 0.67 | 0.90 | 3602 | 2730 | 872 | |
| 75 | 4500 | 0.61 | 0.82 | 3788 | 3150 | 638 | |
| 85 | 5100 | 0.56 | 0.76 | 3959 | 3570 | 389 | |
| 95 | 5700 | 0.53 | 0.71 | 4118 | 3990 | 128 | |
| 105 | 6300 | 0.49 | 0.67 | 4268 | 4410 | -142 | |
| 115 | 6900 | 0.47 | 0.63 | 4410 | 4830 | -420 | |
| 125 | 7500 | 0.44 | 0.60 | 4544 | 5250 | -706 | |
| 135 | 8100 | 0.42 | 0.57 | 4672 | 5670 | -998 | |
| 145 | 8700 | 0.40 | 0.54 | 4795 | 6090 | -1295 | |
| 155 | 9300 | 0.39 | 0.52 | 4912 | 6510 | -1598 | |
| 165 | 9900 | 0.37 | 0.50 | 5025 | 6930 | -1905 | |
| 175 | 10500 | 0.36 | 0.48 | 5134 | 7350 | -2216 | |
| 185 | 11100 | 0.35 | 0.47 | 5239 | 7770 | -2531 | |
| 195 | 11700 | 0.33 | 0.45 | 5341 | 8190 | -2849 | |
| 205 | 12300 | 0.32 | 0.44 | 5439 | 8610 | -3171 | |
| 215 | 12900 | 0.32 | 0.43 | 5535 | 9030 | -3495 | |
| 225 | 13500 | 0.31 | 0.41 | 5628 | 9450 | -3822 | |
| 235 | 14100 | 0.30 | 0.40 | 5718 | 9870 | -4152 | |
| 245 | 14700 | 0.29 | 0.39 | 5806 | 10290 | -4484 | |
| 255 | 15300 | 0.28 | 0.38 | 5892 | 10710 | -4818 | |
| 265 | 15900 | 0.28 | 0.37 | 5976 | 11130 | -5154 | |
| 275 | 16500 | 0.27 | 0.36 | 6058 | 11550 | -5492 | |
| 285 | 17100 | 0.26 | 0.36 | 6138 | 11970 | -5832 | |
| 295 | 17700 | 0.26 | 0.35 | 6217 | 12390 | -6173 | |
| 305 | 18300 | 0.25 | 0.34 | 6294 | 12810 | -6516 | |
| 315 | 18900 | 0.25 | 0.34 | 6369 | 13230 | -6861 | |
| 325 | 19500 | 0.24 | 0.33 | 6443 | 13650 | -7207 | |
| 335 | 20100 | 0.24 | 0.32 | 6515 | 14070 | -7555 | |
| 345 | 20700 | 0.23 | 0.32 | 6586 | 14490 | -7904 | |
| 355 | 21300 | 0.23 | 0.31 | 6613 | 14910 | -8297 | |
| 365 | 21900 | 0.22 | 0.30 | 6669 | 15330 | -8661 | |
| 375 | 22500 | 0.22 | 0.30 | 6678 | 15750 | -9072 | |
| 385 | 23100 | 0.22 | 0.30 | 6856 | 16170 | -9314 | |

Rainfall Intensity Coefficients for ***Spokane***

taken from Table 5-7 SRSM

$M_{25}$ = 9.09 Flow (weighted c)

$N_{25}$ = 0.626 Qwc= 4.48 cfs

Flow (time of concentration)

Qtc= 4.48 cfs

| Time (min) | Time Inc. (sec) | Intens. (in/hr) | Q Devel (cfs) | Vol.In (cu ft) | Vol.Out (cu ft) | Storage (cu ft) |
|---|---|---|---|---|---|---|
| 385 | | | | | | |
| 395 | 23700 | 0.21 | 0.28 | 6712 | 16590 | -9878 |
| 405 | 24300 | 0.21 | 0.28 | 6881 | 17010 | -10129 |
| 415 | 24900 | 0.20 | 0.27 | 6712 | 17430 | -10718 |
| 425 | 25500 | 0.20 | 0.27 | 6873 | 17850 | -10977 |
| 435 | 26100 | 0.19 | 0.25 | 6680 | 18270 | -11590 |
| 445 | 26700 | 0.19 | 0.25 | 6833 | 18690 | -11857 |
| 455 | 27300 | 0.18 | 0.24 | 6616 | 19110 | -12494 |
| 465 | 27900 | 0.18 | 0.24 | 6761 | 19530 | -12769 |
| 475 | 28500 | 0.17 | 0.23 | 6519 | 19950 | -13431 |
| 485 | 29100 | 0.17 | 0.23 | 6656 | 20370 | -13714 |
| 495 | 29700 | 0.16 | 0.21 | 6390 | 20790 | -14400 |
| 505 | 30300 | 0.16 | 0.21 | 6519 | 21210 | -14691 |
| 515 | 30900 | 0.15 | 0.20 | 6229 | 21630 | -15401 |
| 525 | 31500 | 0.15 | 0.20 | 6349 | 22050 | -15701 |
| 535 | 32100 | 0.14 | 0.19 | 6035 | 22470 | -16435 |
| 545 | 32700 | 0.14 | 0.19 | 6147 | 22890 | -16743 |
| 555 | 33300 | 0.13 | 0.17 | 5808 | 23310 | -17502 |
| 565 | 33900 | 0.13 | 0.17 | 5913 | 23730 | -17817 |
| 575 | 34500 | 0.12 | 0.16 | 5550 | 24150 | -18600 |
| 585 | 35100 | 0.12 | 0.16 | 5646 | 24570 | -18924 |
| 595 | 35700 | 0.11 | 0.15 | 5258 | 24990 | -19732 |
| 605 | 36300 | 0.11 | 0.15 | 5347 | 25410 | -20063 |
| 615 | 36900 | 0.10 | 0.13 | 4935 | 25830 | -20895 |
| 625 | 37500 | 0.10 | 0.13 | 5015 | 26250 | -21235 |
| 635 | 38100 | 0.09 | 0.12 | 4579 | 26670 | -22091 |
| 645 | 38700 | 0.09 | 0.12 | 4651 | 27090 | -22439 |
| 655 | 39300 | 0.08 | 0.11 | 4190 | 27510 | -23320 |
| 665 | 39900 | 0.08 | 0.11 | 4254 | 27930 | -23676 |
| 675 | 40500 | 0.07 | 0.09 | 3769 | 28350 | -24581 |
| 685 | 41100 | 0.07 | 0.09 | 3825 | 28770 | -24945 |
| 695 | 41700 | 0.06 | 0.08 | 3316 | 29190 | -25874 |
| 705 | 42300 | 0.06 | 0.08 | 3364 | 29610 | -26246 |
| 715 | 42900 | 0.05 | 0.07 | 2830 | 30030 | -27200 |
| 725 | 43500 | 0.05 | 0.07 | 2870 | 30450 | -27580 |
| 735 | 44100 | 0.04 | 0.05 | 2312 | 30870 | -28558 |
| 745 | 44700 | 0.04 | 0.05 | 2343 | 31290 | -28947 |

| "1815A" TREATMENT REQUIREMENTS | |
|---|---|
| Minimum "1815A" Volume Required | 1,940 cu ft |
| Provided Treatment Volume - Min. | 2,959 cu ft |
| STORAGE REQ. - 25 YEAR DESIGN STORM | |
| Maximum Storage Required by Bowstring | 1,629 cu ft |
| Provided Pond Storage Volume to Inlet - Min. | 3,967 cu ft |
| Provided Drywell/Gallery Storage Volume | 0 cu ft |
| **Total Provided Volume** | **3,967 cu ft** |

PEAK FLOW CALCULATION PROJECT:

**25-Year Design Storm** **LT COMM**

BASIN: A

| | | | | |
|---|---|---|---|---|
| Tot. Area | 111,432 SF | | 2.56 | Acres |
| Imp. Area | 56,178 SF | | C= | 0.9 |
| Perv. Area | 55,254 SF | | C= | 0.15 |
| Wt. C = | 0.53 | | PGIS Area = | 46,552 |

| WCE Applicable Travel Time Ground Cover Coefficients | | |
|---|---|---|
| Per Table 5-6 SRSM | | |
| Type of Cover | | K (ft/min) |
| Short Pasture | | 420 |
| Nearly Bare Ground | | 600 |
| Small Roadside Ditch/ Grass | | 900 |
| Paved Area (use for parking lots) | | 1200 |
| Gutter - 4 inches deep | | 1500 |
| Gutter - 6 inches deep | | 2400 |
| Pipe - 12-inch PVC/DI | | 3000 |
| Pipe - 15/18-inch PVC/DI | | 3900 |
| Pipe - 24-inch PVC/DI | | 4700 |

| **Reaches** | | |
|---|---|---|
| Reach 1 | Offsite | also applicable for Pre-Developed Tc |
| Length | 175.00 | |
| K | 420.00 | |
| Slope (ft/ft) | 0.0400 | be sure this is decimal equivalent slope 0.0000 |
| Travel Time | 2.08 | Minutes |
| | | |
| Reach 2 | Finished Lot from House to Street | |
| Length | 0.00 | |
| K | 420.00 | |
| Slope (ft/ft) | 0.0300 | be sure this is decimal equivalent slope 0.0000 |
| Travel Time | 0.00 | Minutes |
| | | |
| Reach 3 | Gutter Flow to Inlet/Catch Basin | |
| Length | 0.00 | |
| K | 2400.00 | |
| Slope (ft/ft) | 0.0200 | be sure this is decimal equivalent slope 0.0000 |
| Travel Time | 0.00 | Minutes |
| | | |
| Reach 4 | Pipe Flow | Pipe Reach One (only need one if no Dia change) |
| Length | 0.00 | |
| K | 3000.00 | 12-inch Pipe minimum |
| Slope (ft/ft) | 0.0200 | Average Slope for total pipe run |
| Travel Time | 0.00 | Minutes |
| | | |
| Reach 5 | Pipe Flow | Add additional pipe reacheds for other Dia |
| Length | 0.00 | |
| K | 3900.00 | 15/18-inch Pipe |
| Slope (ft/ft) | 0.0200 | Average Slope for total pipe run |
| Travel Time | 0.00 | Minutes |
| | | |
| Sum of Tc | 2.08 | Minutes |
| | | |
| Tc for Analysis | 5.00 | Minutes |

Whipple Consulting Engineers

BOWSTRING METHOD

DETENTION BASIN DESIGN

PROJECT: LT COMM
BASIN: A
DESIGNER: STT
DATE: 2-Jun-21

| | |
|---|---|
| Time Increment (min) | 10 |
| Time of Conc. (min) | 5.00 |
| Outflow (cfs) | 0.70 |
| Design Year Flow | 25 |
| Area (acres) | 2.56 |
| Impervious Area (sq ft) | 56,178 |
| 'C' Factor | 0.53 |
| Area * C | 1.351 |
| PGIS Area | 46,552 |

| Time (min) | Time Inc. (sec) | Intens. (in/hr) | Q Devel. (cfs) | Vol.In (cu ft) | Vol.Out (cu ft) | Storage (cu ft) | |
|---|---|---|---|---|---|---|---|
| 5.00 | 300 | 3.32 | 4.48 | 1803 | 210 | 1593 | |
| 15 | 900 | 1.67 | 2.25 | 2259 | 630 | 1629 | <== |
| 25 | 1500 | 1.21 | 1.64 | 2623 | 1050 | 1573 | |
| 35 | 2100 | 0.98 | 1.33 | 2920 | 1470 | 1450 | |
| 45 | 2700 | 0.84 | 1.13 | 3175 | 1890 | 1285 | |
| 55 | 3300 | 0.74 | 1.00 | 3400 | 2310 | 1090 | |
| 65 | 3900 | 0.67 | 0.90 | 3602 | 2730 | 872 | |
| 75 | 4500 | 0.61 | 0.82 | 3788 | 3150 | 638 | |
| 85 | 5100 | 0.56 | 0.76 | 3959 | 3570 | 389 | |
| 95 | 5700 | 0.53 | 0.71 | 4118 | 3990 | 128 | |
| 105 | 6300 | 0.49 | 0.67 | 4268 | 4410 | -142 | |
| 115 | 6900 | 0.47 | 0.63 | 4410 | 4830 | -420 | |
| 125 | 7500 | 0.44 | 0.60 | 4544 | 5250 | -706 | |
| 135 | 8100 | 0.42 | 0.57 | 4672 | 5670 | -998 | |
| 145 | 8700 | 0.40 | 0.54 | 4795 | 6090 | -1295 | |
| 155 | 9300 | 0.39 | 0.52 | 4912 | 6510 | -1598 | |
| 165 | 9900 | 0.37 | 0.50 | 5025 | 6930 | -1905 | |
| 175 | 10500 | 0.36 | 0.48 | 5134 | 7350 | -2216 | |
| 185 | 11100 | 0.35 | 0.47 | 5239 | 7770 | -2531 | |
| 195 | 11700 | 0.33 | 0.45 | 5341 | 8190 | -2849 | |
| 205 | 12300 | 0.32 | 0.44 | 5439 | 8610 | -3171 | |
| 215 | 12900 | 0.32 | 0.43 | 5535 | 9030 | -3495 | |
| 225 | 13500 | 0.31 | 0.41 | 5628 | 9450 | -3822 | |
| 235 | 14100 | 0.30 | 0.40 | 5718 | 9870 | -4152 | |
| 245 | 14700 | 0.29 | 0.39 | 5806 | 10290 | -4484 | |
| 255 | 15300 | 0.28 | 0.38 | 5892 | 10710 | -4818 | |
| 265 | 15900 | 0.28 | 0.37 | 5976 | 11130 | -5154 | |
| 275 | 16500 | 0.27 | 0.36 | 6058 | 11550 | -5492 | |
| 285 | 17100 | 0.26 | 0.36 | 6138 | 11970 | -5832 | |
| 295 | 17700 | 0.26 | 0.35 | 6217 | 12390 | -6173 | |
| 305 | 18300 | 0.25 | 0.34 | 6294 | 12810 | -6516 | |
| 315 | 18900 | 0.25 | 0.34 | 6369 | 13230 | -6861 | |
| 325 | 19500 | 0.24 | 0.33 | 6443 | 13650 | -7207 | |
| 335 | 20100 | 0.24 | 0.32 | 6515 | 14070 | -7555 | |
| 345 | 20700 | 0.23 | 0.32 | 6586 | 14490 | -7904 | |
| 355 | 21300 | 0.23 | 0.31 | 6613 | 14910 | -8297 | |
| 365 | 21900 | 0.22 | 0.30 | 6669 | 15330 | -8661 | |
| 375 | 22500 | 0.22 | 0.30 | 6678 | 15750 | -9072 | |
| 385 | 23100 | 0.22 | 0.30 | 6856 | 16170 | -9314 | |

Rainfall Intensity Coefficients for ***Spokane***
taken from Table 5-7 SRSM

$M_{25}$ = 9.09

$N_{25}$ = 0.626

Flow (weighted c)
Qwc= 4.48 cfs
Flow (time of concentration)
Qtc= 4.48 cfs

| Time (min) | Time Inc. (sec) | Intens. (in/hr) | Q Devel (cfs) | Vol.In (cu ft) | Vol.Out (cu ft) | Storage (cu ft) |
|---|---|---|---|---|---|---|
| 385 | | | | | | |
| 395 | 23700 | 0.21 | 0.28 | 6712 | 16590 | -9878 |
| 405 | 24300 | 0.21 | 0.28 | 6881 | 17010 | -10129 |
| 415 | 24900 | 0.20 | 0.27 | 6712 | 17430 | -10718 |
| 425 | 25500 | 0.20 | 0.27 | 6873 | 17850 | -10977 |
| 435 | 26100 | 0.19 | 0.25 | 6680 | 18270 | -11590 |
| 445 | 26700 | 0.19 | 0.25 | 6833 | 18690 | -11857 |
| 455 | 27300 | 0.18 | 0.24 | 6616 | 19110 | -12494 |
| 465 | 27900 | 0.18 | 0.24 | 6761 | 19530 | -12769 |
| 475 | 28500 | 0.17 | 0.23 | 6519 | 19950 | -13431 |
| 485 | 29100 | 0.17 | 0.23 | 6656 | 20370 | -13714 |
| 495 | 29700 | 0.16 | 0.21 | 6390 | 20790 | -14400 |
| 505 | 30300 | 0.16 | 0.21 | 6519 | 21210 | -14691 |
| 515 | 30900 | 0.15 | 0.20 | 6229 | 21630 | -15401 |
| 525 | 31500 | 0.15 | 0.20 | 6349 | 22050 | -15701 |
| 535 | 32100 | 0.14 | 0.19 | 6035 | 22470 | -16435 |
| 545 | 32700 | 0.14 | 0.19 | 6147 | 22890 | -16743 |
| 555 | 33300 | 0.13 | 0.17 | 5808 | 23310 | -17502 |
| 565 | 33900 | 0.13 | 0.17 | 5913 | 23730 | -17817 |
| 575 | 34500 | 0.12 | 0.16 | 5550 | 24150 | -18600 |
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| 605 | 36300 | 0.11 | 0.15 | 5347 | 25410 | -20063 |
| 615 | 36900 | 0.10 | 0.13 | 4935 | 25830 | -20895 |
| 625 | 37500 | 0.10 | 0.13 | 5015 | 26250 | -21235 |
| 635 | 38100 | 0.09 | 0.12 | 4579 | 26670 | -22091 |
| 645 | 38700 | 0.09 | 0.12 | 4651 | 27090 | -22439 |
| 655 | 39300 | 0.08 | 0.11 | 4190 | 27510 | -23320 |
| 665 | 39900 | 0.08 | 0.11 | 4254 | 27930 | -23676 |
| 675 | 40500 | 0.07 | 0.09 | 3769 | 28350 | -24581 |
| 685 | 41100 | 0.07 | 0.09 | 3825 | 28770 | -24945 |
| 695 | 41700 | 0.06 | 0.08 | 3316 | 29190 | -25874 |
| 705 | 42300 | 0.06 | 0.08 | 3364 | 29610 | -26246 |
| 715 | 42900 | 0.05 | 0.07 | 2830 | 30030 | -27200 |
| 725 | 43500 | 0.05 | 0.07 | 2870 | 30450 | -27580 |
| 735 | 44100 | 0.04 | 0.05 | 2312 | 30870 | -28558 |
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| "1815A" TREATMENT REQUIREMENTS | |
|---|---|
| Minimum "1815A" Volume Required | 1,940 cu ft |
| Provided Treatment Volume - Min. | 2,959 cu ft |
| STORAGE REQ. - 25 YEAR DESIGN STORM | |
| Maximum Storage Required by Bowstring | 1,629 cu ft |
| Provided Pond Storage Volume to Inlet - Min. | 3,967 cu ft |
| Provided Drywell/Gallery Storage Volume | 0 cu ft |
| **Total Provided Volume** | **3,967 cu ft** |

# GEOTECHNICAL REPORT

# LIMITED GEOTECHNICAL EVALUATION

PROPOSED EXPERIENCE SENIOR LIVING FACILITY
NW OF EAST 55TH AVENUE AND SOUTH FISKE STREET
SPOKANE, WASHINGTON
ALLWEST PROJECT NO. 220-252G

NOVEMBER 19, 2020

November 19, 2020

Mr. Tyler Lundsgaard
NexCore Group
1550 Market Street, Suite 200
Denver, Colorado 80202

**RE: Limited Geotechnical Evaluation**
**Proposed Experience Senior Living Facility**
**NW of East 55$^{th}$ Avenue and South Fiske Street**
**Spokane, Washington**
**ALLWEST Project No. 220-252G**

Mr. Lundsgaard,

**ALLWEST** has completed the authorized Limited Geotechnical Evaluation for the proposed project at the above-referenced site in Spokane, Washington. The attached report presents the results of the field evaluation, laboratory testing, and our recommendations to assist the design and construction of the proposed project.

We appreciate the opportunity to work with you on this project. If you have any questions, or need additional information, please do not hesitate to call us at (509) 534-4411.

Sincerely,
ALLWEST Testing & Engineering, Inc.

Todd DeMico, P.E.
Engineering Services Manager

David Rauch, P.E.
Project Manager

Attachment: Limited Geotechnical Evaluation Report

**TABLE OF CONTENTS**

ALLWEST Project No. 220-252G
Proposed Experience Senior Living Facility
NW of East 55th Avenue and South Fiske Street
Spokane, Washington

Page

**LIMITED GEOTECHNICAL EVALUATION**
**PROPOSED EXPERIENCE SENIOR LIVING FACILITY**
**NW OF EAST 55$^{TH}$ AVENUE AND SOUTH FISKE STREET**
**SPOKANE, WASHINGTON**

**ALLWEST** has completed the authorized Limited Geotechnical Evaluation for the proposed senior living facility to be located immediately northwest of the intersection of East 55$^{th}$ avenue and South Fisk Street in Spokane, Washington. The general location of the project is shown on the Site Location Map, Figure 1, in Appendix A of this report. The purpose of the limited evaluation was to assess the subsurface soil conditions at the subject property with respect to the proposed construction. This report presents the results of the field exploration and laboratory testing and presents our recommendations to assist the design and construction of the proposed property development.

## 1.0 SCOPE OF SERVICES

To complete our services, we accomplished the following:

1) Reviewed the USDA Natural Resources Conservation Service and Washington Geological Survey geologic mapping information for the project site area. We also reviewed the following document prepared for the site:

    a) Preliminary Site Layout provided by NexCore Group

2) Completed a site reconnaissance by walking the property and observing exposed surface conditions including pavement, vegetation, and drainage.

3) Performed a field evaluation by observing the excavation of five (5) test pits and drilling of three (3) exploratory borings within the proposed construction areas. We obtained samples of the soils encountered in the test pits and borings and retained them for laboratory testing. The soils were visually described and classified, and the subsurface profiles were logged.

4) Performed laboratory tests on select soil samples to assess some of the soil engineering characteristics.

5) Reviewed the results of the field evaluation and laboratory testing with respect to the proposed construction.

6) Performed engineering analyses and prepared recommendations to assist project planning, design, and construction.

7) Prepared this report.

Our services were provided in general accordance with our proposal dated August 14, 2020.

## 2.0 PROJECT DESCRIPTION

We understand the proposed project will consist of constructing a new two-story senior living facility on approximately 5.6 acres of land in Spokane, Washington. The development is proposed on a partially developed lot immediately northwest of East 55th Avenue and South Fiske Street. In addition to the senior living facility, we anticipate the remainder of the property will be utilized for parking and drive lanes, and stormwater management facilities. Specific loading conditions were not available at the time this report was prepared. For our purposes, we have assumed wall loads will be on the order of 2 to 5 kips per lineal foot and column loads, if any, will be on the order of 75 kips or less. We have further assumed traffic loads in the parking and drive areas will consist of passenger car traffic in the parking area with occasional delivery truck traffic in the parking/drive areas.

## 3.0 EVALUATION PROCEDURES

To complete this limited evaluation, we reviewed soil and geologic literature for the project area. We also reviewed the document referenced in Section 1.0 of this report. We conducted a field evaluation of the property including a site reconnaissance to assist in planning the field evaluation and provide a general overview of the property. Information obtained from the field evaluation, review of the referenced documents, laboratory testing, and engineering analysis were utilized to develop recommendations for the geotechnical aspects of the project.

The test pits were loosely backfilled at the conclusion of the field evaluation. The backfill will consolidate with time. We recommend the test pits be re-located at the time of construction. If the test pits underlie building or pavement areas, the backfill should be re-excavated and the material replaced and compacted to at least 95 percent of the maximum dry density as established by ASTM D1557 (modified Proctor).

## 4.0 SITE CONDITIONS

At the time of our subsurface exploration, the lot consisted predominately of vacant land with large soil stockpiles present across the property. Several structures previously occupied the property but were demolished prior to our subsurface investigation. We did not observe rock outcrops or standing water at the existing ground surface on the subject site during our site evaluation. The site is bounded by East 53rd Avenue to the north, Fiske Street to the east, East 55th Avenue to the south, and residential property/vacant land to the west.

### 4.1 General Soil Conditions

The USDA Natural Resources Conservation Service (NRCS) has mapped the soil on the property primarily as Urban land and Urban land – Seaboldt, disturbed complex, 0 to 3 percent slopes (see Appendix A, Figure 2). Urban land soils are often highly variable and difficult to characterize as they are derived from human transported material. Seaboldt soils are described as well drained ashy loam to extremely gravelly sandy loam overlying shallow bedrock. Seaboldt soils are derived from loess mixed with minor amounts of volcanic ash overlying glaciofluvial deposits and basalt residuum. The native soils in the borings consisted predominately of silt loess and poorly graded sand alluvium overlying broken basalt bedrock.

### 4.2 Hydrogeologic Conditions

The depth to groundwater is estimated to be as shallow as approximately 15 feet below the site based on nearby water well logs provided by the State of Washington Department of Ecology. Site-specific groundwater reports were not identified for this project site. Groundwater was observed in all borings at an approximate depth between 15.0 and 16.5 feet below the ground surface. Based on the reference water well logs from the area and our subsurface findings, we do not anticipate groundwater may adversely affect the proposed construction.

## 5.0 SUBSURFACE CONDITIONS

Three (3) borings and five (5) test pits were observed at the site at the approximate locations shown on the Boring Location Map, Figure 3, in Appendix A of this report. The test pit and boring locations were field located based on features shown on the site plan provided. The borings were drilled and test pits were excavated with equipment and crew under contract with ALLWEST. The soil conditions observed in the test pits/borings were visually described and classified in general accordance with ASTM D2487 and D2488 and the subsurface profiles were logged. Representative soil samples were obtained from the test pits/borings and returned to our Spokane Valley laboratory for further classification and laboratory testing.

The soil sampling in the borings was performed using standard penetration test procedures in accordance with ASTM D 1586. With this method, a hollow-stem auger or casing is advanced to the desired test depth. A 140-pound hammer falling 30 inches was used to drive a split-barrel sampler a total of 18 inches below the tip of the hollow-stem auger. The blows required to advance the sampler are recorded for each 6-inch increment. The blows for the last foot of penetration are called the N-value and are an indication of the soil strength characteristics. The N-values are shown on the Log of Boring sheets in Appendix B. Penetration test samples were taken at vertical intervals ranging between 2.5 feet 5 feet as shown on the boring logs.

### 5.1 Subsurface Soil Conditions

Soils observed in the borings/test pits generally consisted of topsoil and/or fill overlying native fine grained loess, coarse grained alluvium, and broken basalt bedrock. Topsoil was encountered in Test Pit TP-2 and extended to a depth of 0.6 feet below the ground surface. Buried topsoil was observed to a depth of 2.0 feet below in Test Pit TP-1 below a thin layer of undocumented fill. Topsoil consisted of silt. With the exception of Test Pit TP-2, all borings and test pits encountered a layer of either undocumented or uncontrolled fill overlying native soil/bedrock. Fill at the test locations extended to depths ranging between 0.5 and 4.0 feet below the ground surface and consisted of silty to well graded gravel. Uncontrolled fill containing metal and brick fragments was observed in Test Pits TP-3 and TP-4 and extended to depths of up to 4.0 feet below the ground surface. Undocumented fill consisting of recycled asphalt was observed in Test Pit TP-5 to a depth of 1.5 feet below the ground surface. Fill extended to basalt bedrock (or suspected basalt bedrock) at all locations with the exception of Test Pits TP-1 and TP-2. Native soils in Test Pits TP-1 and TP-2 consisted of silt to silty sand loess overlying poorly graded to silty sand alluvium.

Detailed descriptions of the soil observed in the borings and test pits are presented on the Logs of Borings/Test Pits in Appendix B of this report. The descriptive soil terms used on the boring/test pit logs and in this report can be referenced by the Unified Soil Classification System (USCS). A copy of the USCS is included in Appendix B. The subsurface conditions may vary between exploration locations. Such changes in conditions would not be apparent until construction. **If the subsurface conditions do change from those observed in the borings/test pits, the construction timing, plans, and costs may change.**

### 5.2 Bedrock

Suspected basalt bedrock was encountered at all test pit locations due to refusal of the excavator bucket. Suspected basalt bedrock in the test pits was encountered at depths ranging between 0.5 and 4.5 feet below the ground surface. The presence of basalt bedrock was confirmed by drilling of three (3) borings within the footprint of the proposed structure on the property. Borings encountered basalt bedrock ad depths ranging between 0.5 and 3.0 feet below the ground surface. Basalt bedrock was described as slightly weathered, medium hard to very hard, and broken with sand and clay infilling.

At the boring locations, borings were advanced via air rotary drilling to a depth in which suspected competent bedrock was encountered. At that depth diamond rock coring was attempted in an effort to obtain rock cores and evaluate rock quality. Due to the brokenness of the rock encountered in the borings, diamond rock coring resulted in slow drilling rates and low recovery. Due to the difficulty in advancement by diamond rock coring, all borings were advanced to the final termination depth via air rotary drilling. While this method is not an indicator of rock quality, the drilling rate and drilling spoils can confirm basalt bedrock was encountered to the final termination depth, and borings did not advance through a rock shelf into underlying soils.

### 5.3 Groundwater Conditions

Groundwater was encountered in all borings during our investigation between a depth of 15.0 and 16.5 feet below the ground surface. Changes in precipitation, construction or other factors may impact this depth to groundwater on the property. Based on a review of nearby water well records and our subsurface findings, we do not anticipate groundwater may affect the proposed construction. Should below grade structures be proposed at the site, there is some degree of risk that groundwater may impact construction. ALLWEST should be contacted if below grade structures are proposed for the site or if groundwater is encountered during construction so that we can evaluate whether the recommendations provided herein apply.

## 6.0 LABORATORY TESTING

Laboratory testing was performed to supplement field classifications and to assess some of the soil engineering parameters. The laboratory testing conducted included particle size distribution tests (ASTM D6913) and organic content testing (ASTM D2974) on select samples. The laboratory test results are summarized in Appendix C. The laboratory testing was performed by ALLWEST.

## 7.0 SLOPE STABILITY ANALYSIS

A final grading plan for the site was not available at the time this report was prepared. In general, 2H:1V slopes taller than 5 feet in height, slopes of any height that are steeper than 2H:1V, or slopes near vehicular or structural loading should be evaluated for global stability. In addition, retaining walls in excess of 4 feet in height or retaining walls of any height adjacent to building/vehicular surcharge loads or sloping ground conditions should be evaluated for internal, external, and global stability. We recommend ALLWEST be provided with a final grading plan when available to evaluate potential global stability concerns.

## 8.0 CONCLUSIONS AND RECOMMENDATIONS

It is our opinion the site is suitable for the proposed construction provided the recommendations in this report are followed and the associated risks are acceptable to the owner. We encountered native fine grained silt and silty sand loess soils with a very high fines content in select test pits to depths of approximately 2.7 feet below the ground surface. In addition, undocumented/uncontrolled fill, including fill placed over topsoil was encountered at all test pit locations with the exception of TP-2. We recommend all fine grained soils/silty sand soils with a high fines content (as defined by soils having at least 25 percent of material passing the No. 200 sieve by mass) and undocumented/uncontrolled fill be overexcavated and backfilled with properly compacted structural fill as outlined within the Structural Fill, Placement, and Compaction section of this report (Section 8.4).

In addition to our investigative findings, we understand multiple structures on the property have recently been demolished. ALLWEST is unaware of whether these structures included basements/below grade structures. We recommend all fill and building debris resulting from the recent demolition of these structures be excavated and wasted off site. Due to the variation in depth and composition of fill on the property and the recent demolition of site structures, we recommend probe test pits be excavated within the footprint of proposed structures and pavements at the time of construction. These probe pits will be beneficial in identify and delineating unsuitable soils within the footprint of the proposed development.

The following recommendations are presented to assist the planning and design of the proposed building, pavement, and stormwater management facilities. The recommendations are based on our understanding of the proposed construction, the conditions observed in the test pits, borings, laboratory test results, and engineering analysis. **If the scope of construction changes, or if conditions different than those described herein are encountered during construction, we should be notified so we can review our recommendations and provide revisions if necessary**.

## 8.1 Site Preparation

General

Test pit locations were loosely backfilled with onsite material following completion. Due to the nature of this backfill, it should be assumed that these locations consist of pockets of loose uncontrolled fill and/or disturbed native soils. In order to limit the potential for future settlement or subsurface disturbance, we recommend that test pit locations be overexcavated in their entirety below pavements and structures and backfilled with properly compacted native soils or structural fill. The approximate test pit locations are indicated on Figure A-3 of this report, and the final test pit depths are included on the test pit logs included in Appendix B of this report.

Building

Prior to conducting site grading, vegetation, deleterious material, native fine grained silt and very silty sand loess soils, disturbed soil including undocumented/uncontrolled fill, and soil containing significant amounts of roots and organics, should be removed in entirety below foundations, slabs, pavement, and flatwork. In the event any material is considered unsuitable for foundation bearing material, it should be removed its entire depth below foundations, slabs, pavement, and flatwork and replaced with compacted structural fill. Over-excavation and replacement of unsuitable material should extend a minimum of five (5) feet horizontally beyond the building perimeters.

Prior to placing structural fill, the exposed subgrade should be scarified to a minimum depth of eight (8) inches; then properly moisture conditioned and compacted to at least 95 percent of the modified Proctor maximum dry density as established by ASTM D1557. Compaction of the subgrade may be reduced to proof rolling at the discretion of the geotechnical engineer based on conditions at the time of construction. If the subgrade is observed to significantly deflect, it should be over-excavated to firm, non-yielding soil and replaced with properly compacted fill or stabilized as recommended in the Subgrade Stabilization section of this report.

Pavement Areas

Subsequent to removal of topsoil, native fine grained/very silty sand soils, and undocumented/uncontrolled fill, the exposed subgrade should be scarified to an approximate depth of eight (8) inches; properly moisture conditioned and compacted to at least 95 percent of the modified Proctor maximum dry density as established by ASTM D1557. Compaction of the subgrade may be reduced to proof rolling at the discretion of the geotechnical engineer based on conditions. If the subgrade is observed to deflect, it should be over-excavated to firm, non-yielding soil and replaced with compacted fill or stabilized as recommended in the Subgrade Stabilization section (Section 8.2) of this report.

Utilities

Support soil for underground utilities will likely consist of silty to poorly graded sand or basalt bedrock. We recommend existing fine grained soils and any existing fill below utilities be overexcavated and backfilled with properly compacted structural fill to utility bearing grade. It is our opinion the on-site native sand should generally provide adequate support for utilities so long as they are properly bedded per the applicable standard and particles larger than four (4) inches are removed within twelve (12) inches of the bearing grade. Should utilities be proposed to bear on bedrock, we recommend a minimum of twelve (12) inches of properly compacted crushed aggregate and/or poorly graded sand be placed between basalt bedrock and the utility bearing grade. If undocumented fill or loose soil is encountered in utility excavations below the utility bearing surface, we recommend these soils be removed from below utilities and replaced with compacted structural fill in properly sized lifts. It is further our opinion the native on-site sand may be used as backfill for utilities outside of the pipe bedding zone provided particles larger than four (4) inches are removed.

**8.2 Subgrade Stabilization**

If the subgrade is observed to deflect significantly during grading, it should be stabilized prior to placing fill. The subgrade may be stabilized using either fractured, angular cobble or with geosynthetics in conjunction with imported structural fill. The required thickness of crushed cobble or structural fill (used in conjunction with geosynthetic reinforcement) will depend on the construction traffic loads which are unknown at the time of this report. Therefore, a certain degree of trial and error may be needed to verify the recommended stabilization section thicknesses.

If fractured, angular cobble is selected to stabilize the subgrade, it should have a maximum particle size of 6 inches and should be relatively free of sand, silt, and clay. The first layer of cobble should be placed in an 18-inch-thick loose lift and trafficked with tracked-construction and vibratory drum compaction equipment until it is observed to densify. If vibratory compaction destabilizes the subgrade, it should be discontinued. If the cobble is placed in a confined excavation, it should be mechanically densified from outside the excavation with vibratory compaction equipment.

If geosynthetic reinforcement is selected, it should consist of Tensar TX-160 or equivalent. Alternatives to Tensar TX-160 should be approved by the geotechnical engineer prior to use on site. The following recommendations are provided for subgrade stabilization using geosynthetic reinforcement.

- Geosynthetic reinforcement materials should be placed on a properly prepared subgrade with a smooth surface. Loose and disturbed soil should be removed prior to placement of geosynthetic reinforcement materials.

- A woven geotextile filter fabric should be placed on the properly prepared subgrade. The geosynthetic reinforcement should be placed six (6) inches above the filter fabric. The filter fabric and geosynthetic reinforcement should be unrolled in the primary direction of fill placement and should be over-lapped at least three (3) feet. The geosynthetic materials should be pulled taut to remove slack and pinned in place. If the material does not remain taut during fill placement, its effectiveness will be reduced.

- Construction equipment should not be operated directly on the geosynthetic materials. Fill should be placed from outside the excavation to create a pad on which equipment may be operated. We recommend a minimum of twelve (12) inches of structural fill be placed over the geosynthetic reinforcement before operating construction equipment on the fill. Low pressure, track-mounted equipment should be used to place fill over the geosynthetic reinforcement.

- Fill placed directly over the geosynthetic reinforcement should be properly moisture conditioned prior to placement and should meet the following gradation.

| Sieve Size | Percent Passing |
|---|---|
| 1 ½ inch | 100 |
| ¾ inch | 50 - 100 |
| #4 | 25 - 50 |
| #40 | 10 - 20 |
| #100 | 5 - 15 |
| #200 | ≤ 10 |

- The fill material should be properly compacted. Care should be taken with the use of vibratory compaction equipment. Vibration should be discontinued if it reduces the subgrade stability.

A representative of ALLWEST should be on site during subgrade stabilization activities to verify our recommendations are followed and to provide additional recommendations as appropriate.

### 8.3 Excavation

Excavation of the on-site soils can be conducted with typical excavation equipment. Excavation of shallow bedrock is likely feasible with rock hammering equipment due to the brokenness of the rock mass. Significant excavation of bedrock at depth may require other means such as blasting. Should blasting be selected as an excavation alternative, we recommend a qualified blasting contractor be contacted to evaluate the feasibility of blasting at the site. Typically, blasting should include preblast surveys and vibration monitoring as determined by a blasting contractor at the time of construction. We recommend excavations greater than four (4) feet deep be sloped no steeper than 1.5H:1V (horizontal to vertical). Alternatively, deeper excavations may be shored or braced in accordance with OSHA specifications and local codes. Regarding trench wall support, the site soil is considered Type C soil according to Occupational Safety and Health Administration (OSHA) guidelines. The Contractor is responsible to provide appropriate trench wall support and/or sloping.

The Contractor shall limit excavations near roadways, existing structures, or existing slopes. Sloping or excavating in front of roadways/structures/slopes, may create slope instabilities. If the Contractor needs to excavate near these features, ALLWEST should be given an opportunity to review the proposed excavation prior to construction.

<u>Dewatering</u>
Groundwater was encountered in select borings during our investigation; however, we do not anticipate excavations will extend to depths in which groundwater was encountered. In the event groundwater is encountered, all excavations shall be continuously dewatered as determined by the Contractor's means and methods.

### 8.4 Structural Fill, Placement, and Compaction

Structural fill is defined as soil placed or moved on a site that will support any structural element including buildings, slabs, or pavement. Structural fill typically includes the footprint area and five (5) feet beyond for structures. Non-structural fill is soil placed beyond the structural fill area. Structural fill should be free of organic matter, frozen soil, and deleterious debris. Prior to placing structural fill, topsoil, organic material, native fine grained soils/loess soils with a high fines content, and undocumented/uncontrolled fill and debris should be removed. The ground surface should be relatively level. Structural fill should be placed on subgrades prepared as directed in the Site Preparation section (Section 8.1) of this report. In wet weather or spring conditions, using silty or fine-grained soil for fill may delay construction and increase costs.

It is our opinion the native silt soils and shallow silty sand loess soils are not suitable as structural fill on the property. Based on our subsurface findings, the fines content of the native sand soils appears to decrease with depth. The shallow silty sand loess soil encountered in the test pits included a fines content near 50 percent (borderline silt); therefore, we do not recommend the shallow silty sand loess soil be utilized as structural fill. We recommend this material be wasted off-site or utilized outside of the footprint of pavements and structures. It is our opinion the native poorly graded to silty sand encountered at depth and the silty to well graded gravel undocumented/uncontrolled fill encountered on site, may be suitable for reuse as structural fill provided it is free of foreign debris, organics, and deleterious debris. We do not recommend recycled asphalt encountered on site be reused as structural fill. Material that is to be reused on site as structural fill is only considered suitable if it can be kept at or near optimum moisture content for compaction and particles larger than six (6) inches in diameter are separated. We recognize some of the granular soils may contain higher percentages of fine-grained particles that can be difficult to properly compact. If these soils are found to be over-optimum moisture at the time of construction, it may be necessary to blend these soils more permeable, coarse gravel or sand to stabilize the soils. All soils to be utilized as structural fill on the property should be evaluated by a representative of ALLWEST prior to construction.

We recommend structural fill consist of granular material meeting the particle size requirements of the Washington State Department of Transportation Standard Specifications section 9-03.14(2) for Select Borrow as shown below in the following table. ALLWEST can review alternate structural fill submittals if requested prior to construction.

| Sieve Size | Percent Passing |
|---|---|
| 6-inch | 99-100 |
| 3-inch | 75-100 |
| No. 40 | 50 max. |
| No. 200 | 10.0 max. |

Structural fill should be placed in lift thicknesses which are appropriate for the compaction equipment used. Typically, six (6) to eight (8) inch loose lifts are appropriate for typical rubber tire and steel drum compaction equipment. Lift thicknesses should be reduced to four (4) inches for hand operated compaction equipment. Structural fill should be moisture conditioned to within two (2) percentage points of the optimum moisture content prior to placement to facilitate compaction. We recommend structural fill be compacted to a minimum of 95 percent of the modified Proctor maximum dry density up to subgrade elevation in building, parking, drive, and slab areas. The compaction efforts should result in the soil being compacted to a firm, dense, and unyielding condition (to the point where no further compression is observed. We recommend ALLWEST be retained to observe the placement and compaction to assess if sufficient compaction has been achieved.

The following recommendations are provided for placement of fill materials which cannot be tested due to the percentage of oversize particles (+3/4" diameter) being more than allowed by ASTM specifications.

- The structural fill should be placed in maximum 12-inch-thick lifts with a minimum 10-ton vibratory compactor. The compactor should impart a minimum dynamic force of 30,000 pounds of impact per vibration with a minimum of 1,000 vibrations per minute. These recommendations are based on Washington State Department of Transportation Standard Specifications for placement of rock fill, WSDOT 2-03.3(14) A.
- A minimum of six (6), full coverage passes should be made for each six (6) inches of lift thickness.
- For fill materials not able to be tested by nuclear densometer due to the large amount of oversize particles, we recommend an ALLWEST representative observe the placement and compaction effort on a full-time basis.

### 8.5 Wet Weather Construction

We recommend earthwork for this site be scheduled for the drier seasons of the year. If construction is undertaken in wet periods of the year, it will be important to slope the ground surface to provide drainage away from construction areas. Additional earthwork may be needed to compact soils to recommended soil density levels if earthwork is performed during the wetter periods of the year.

### 8.6 Cold Weather Construction

The near surface soils encountered in the borings are considered to be frost susceptible. If site grading and construction are anticipated during cold weather, we recommend good winter construction practices be observed. Snow and ice should be removed from excavated and fill areas prior to additional earthwork or construction. Footings, floors slabs, or any structural portions of the construction should not be placed on frozen ground; nor should the supporting soils for buildings be permitted to freeze during or after construction. Frozen soils should not be used as backfill or fill.

### 8.7 Foundation Recommendations

It is our opinion the proposed building at the site may be supported on spread footings with on-grade floor slabs bearing on native poorly graded sand /silty sand encountered at depth or underlying basalt bedrock. We do not recommend foundations bear on silt or very silty sand loess soils encountered at shallow depths. We recommend fine grained (silt or clay) and/or granular soils with a high fines content and existing fill be overexcavated within the footprint of pavements and structures and replaced with properly compacted structural fill.

Spread Footing Foundation Design

- The proposed building may be supported on spread footings bearing on native silty to poorly graded sand compacted to at least 95 percent of modified Proctor maximum dry density, broken/intact basalt bedrock, and/or structural fill placed above the native sand/ basalt bedrock compacted to at least 95 percent of the modified Proctor maximum dry density placed above the native soil/bedrock. We recommend proof rolling of the subgrade be performed to locate soft zones. Local soft zones shall be overexcavated and properly compacted.

- We recommend a minimum footing width of 18 inches, or larger if dictated by local code/building code. Should smaller footings be recommended for the site ALLWEST shall be contacted to evaluate whether the recommendations provided herein apply.

- Spread footings may be designed for a net allowable bearing pressure of 2,000 pounds per square foot (psf) provided the footings bear on native poorly graded to silty sand encountered at depth and/or on structural fill placed above the native sand. This recommended bearing capacity does not apply to the silty sand loess soils encountered at shallow depth. We recommend a representative of ALLWEST be onsite to evaluate bearing subgrade conditions at the time of construction. Should spread footings extend to basalt bedrock, these footings may be designed for a net allowable bearing pressure of 3,500 psf. The recommended value for basalt bedrock has been limited due to the brokenness of the rock encountered in the borings. A minimal leveling course (i.e several inches) of properly compacted crushed aggregate may be utilized at localized uneven areas between the bottom of footing elevation and top of bedrock. Crushed aggregate should consist of angular 5/8 minus material or equivalent. The allowable bearing pressure value may be increased by one-third to account for transient loads such as wind and seismic.

- Footings should be embedded at least 24 inches below the lowest adjacent grade or to local jurisdiction required depth for frost protection. Isolated or unheated foundations, such as for canopies, should be placed at least 36 inches below the exposed ground surface. We do not recommend footings for individual structures bear on a combination of soil and bedrock due to the potential for differential settlement.

- If the previous recommendations are implemented, it is our opinion total settlement will be one (1) inch or less and differential settlement will be approximately ½ inch or less. Footings for individual structures should not bear on a combination of soil and bedrock due to the potential for differential settlement.

- A coefficient of friction of 0.35 may be used for sliding resistance between formed concrete footings and native soil/structural fill. A coefficient of friction of 0.45 may be used for sliding resistance between bedrock and formed concrete footings. In addition, for mass concrete placed on a vapor retarder, we recommend using a coefficient of friction against sliding of 0.35.

- The ground surface around foundations should be sloped away from the foundations at a minimum grade of five (5) percent in the first ten (10) feet. The slope may be reduced to two (2) percent if impermeable ground covering, such as pavement, is placed adjacent to the foundation.

- We recommend backfill placed adjacent to foundation walls be compacted to a minimum of 92 percent of the modified Proctor maximum dry density as established by ASTM D1557. Backfill should be placed in uniform lifts on both sides of the foundation walls to reduce displacement of the foundation walls.

## 8.8 Concrete On-Grade Slabs

We recommend placing a minimum of six (6) inches of crushed aggregate base immediately below slabs (i.e. 5/8" minus or equivalent) proposed on soil. At locations where floor slabs are proposed over bedrock, we recommend a minimum of twelve (12) inches of crushed aggregate base immediately below slabs. Aggregate base should be compacted as recommended in the Structural Fill, Placement, and Compaction section (Section 8.4) of this report. We recommend native fine grained soils, undocumented/uncontrolled fill, and local soft spots below floor slabs be over-excavated and backfilled with properly compacted structural fill as outlined in Section 8.4 of this report.

We recommend consideration be given to including a moisture vapor retarder beneath concrete on-grade floor slabs to retard moisture migration through the slabs if moisture sensitive floor coverings are planned. We recommend the moisture retarder be installed per American Concrete Institute (ACI) recommendations and specifications. To reduce the potential for moisture migration through the slabs, it is important to include the moisture vapor retarder as well as direct surface and subsurface water away from the slabs. In addition, concrete should be given adequate time to cure prior to placing impermeable flooring.

## 8.9 Lateral Earth Pressures

At the time this report was prepared, it was unknown whether below grade walls such as basements are proposed for the project. If below grade walls are incorporated into the design, these walls may retain low to significant amounts of soil. Additionally, if walls are proposed near adjacent structures, parking lots or roadways, they may need to be designed to account for surcharge loading from these adjacent features. To prevent hydrostatic pressures from developing against the walls, we recommend using a free-draining granular material with less than five (5) percent passing a No. 200 sieve as backfill.

The equivalent fluid pressure used to design potential walls will depend on the soil type used as backfill and whether the walls are designed to be flexible (allowed to move) or rigid (not allowed to move). We recommend using the following values for design provided the walls will be utilized to retain native silt/sand soils.

| Wall Type | Soil Type | Active Earth Pressure Coefficient ($K_a$) | At-Rest Earth Pressure Coefficient ($K_o$) | Equivalent Fluid Pressure (pcf) |
|---|---|---|---|---|
| Flexible | Native Silt/Sand | 0.33 | --- | 40 |
| Rigid | Native Silt/Sand | --- | 0.50 | 60 |

For the native silt/sand soils, we assumed a moist unit weight of 115 pcf and an angle of internal friction of 30 degrees for design. Should soils near walls differ from the native unsaturated silt/sand encountered during our investigation, ALLWEST shall be contacted to provide additional recommendations.

For passive pressures, we recommend using the following values for design. Note that passive pressure should not be considered above the frost depth.

| Soil Type | Passive Earth Pressure Coefficient, $K_p$ | Equivalent Fluid Pressure (pcf) |
|---|---|---|
| Native Silt/Sand | 3.0 | 230 |

The equivalent fluid pressures are for horizontal backfill without surcharge loading. Sloped backfill above the wall or surcharge loading will increase the above-mentioned equivalent fluid pressures. These pressures also do not include any potential loading from adjacent structures or vehicular loading. If sloped backfill will be present behind walls or if proposed structures or pavements are to be located near these proposed walls, we should be contacted to provide additional recommendations for these equivalent fluid pressures. The active and at-rest pressures should be increased by an equivalent fluid weight of 10 pounds per cubic foot (pcf) and the passive pressure should be reduced by 10 pcf for seismic design. The dynamic component of the active pressure acts at a height of approximately 0.6 times the height of the wall.

The above values assume groundwater levels will be below any proposed below grade structures. Retaining and/or basement walls should be drained so the potential for hydrostatic forces affecting the walls is reduced. We recommend placing free-draining gravel and drainpipes behind walls to assist with drainage and reduce the potential for the buildup of hydrostatic pressures. ALLWEST should be contacted if groundwater is encountered during construction to evaluate whether the recommendations provided herein apply.

### 8.10 Seismicity

We anticipate the 2018 International Building Code (IBC) will be used as the basis for design of the proposed structures. Based on information provided in the IBC, the soil at the site can be characterized as Site Class C for seismic design. Seismic design parameter values from 2016 ASCE 7 Standard have been adopted into the 2018 International Building Code.

Latitude: 47.60481099 °N        Longitude: -117.36636360 °W

The following maximum earthquake spectral response accelerations should be used for design:

Short Period Response ($S_S$) – 0.306g
One Second Response ($S_1$) – 0.111g

The Site Class C site coefficients are:

$F_a$ – 1.3        $F_v$ – 1.5

### 8.11 Stormwater and Drainage

Final stormwater management plans were not available at the time this report was prepared. We anticipate stormwater runoff will be directed to one or more grassed swale(s) and/or gravel galleries. Due to the shallow depth to limiting layers (bedrock), we do not recommend stormwater infiltration with drywells. Infiltration testing was performed at one location on the site utilizing the single ring infiltration method as outlined by Appendix 4D of the Spokane Regional Stormwater Manual (SRSM). The location of the test (INF-1) is shown on the Test Pit Location Map included in Appendix A of this report. Our observed and recommended infiltration and permeability rates are summarized in the table below.

**Recommended Infiltration and Permeability Rates**

| Test No. | Depth (ft) | Tested Infiltration Rate | Recommended Infiltration Rate | Recommended Permeability Rate |
|---|---|---|---|---|
| INF-1 | 3.0 | $6.76 \times 10^{-4}$ | $2.70 \times 10^{-4}$ | $1.54 \times 10^{-4}$ |

Our recommended infiltration rates include a factor of safety of 2.5 due to the shallow depth of limiting layers at all test locations.

Stormwater management features should be constructed to provide adequate separation between the infiltration grade and limiting layers as outlined within the SRSM. We recommend the site be graded such that storm run-off water is directed away from the building and pavement areas to a stormwater drainage system. We recommend landscape areas be sloped a minimum of six (6) inches within ten (10) feet of the building and slabs be sloped a minimum of two (2) percent. In addition, we recommend gutters and downspouts with long splash blocks or extensions. We do not recommend directing stormwater into a foundation drainpipe system.

### 8.12 Pavement

Pockets of silt/very silty sand soils were encountered in the test pits to depths of 2.7 feet below the ground surface. Additionally, existing fill soils were encountered in select test pits to depths of 4.0 feet below the ground surface. It is our recommendation silt soils and fill material be removed in entirety below the footprint of the proposed pavements. Depending on the final grading of the site this recommendation may be adjusted at the time of construction (as determined by ALLWEST) based on site conditions and the level of risk acceptable to the owner. Reducing the degree of overexcavation may result in premature pavement distresses such as rutting and fatigue cracking.

After removing topsoil, localized soft spots, fine grained soils, and any unsuitable uncontrolled/undocumented fill, and preparing the subgrade, we anticipate the subgrade will consist of native silty to poorly graded sand, basalt bedrock, or structural fill placed above the native sand/bedrock. It is our opinion the native sand/bedrock or structural fill will provide an adequate pavement section subgrade provided the subgrade is prepared as recommended in the Site Preparation section (Section 8.1) of this report. It is important the subgrade surface be shaped to provide for positive drainage to reduce the potential for water to pond in the subgrade.

Prior to placing the aggregate base, we recommend subgrade areas be compacted to at least 95 percent of the modified Proctor maximum dry density (ASTM D1557). In addition, the subgrade area should be proof-rolled with a loaded dump truck. This measure would assist in detecting any localized soft areas. Any soft areas discovered during the proof-rolling operation should be excavated and replaced with a suitable structural fill material. We recommend the proof-rolling process be observed by a geotechnical engineer to make the final evaluation of the subgrade.

Where pavement is proposed directly over basalt bedrock we recommend a pavement section consisting of a minimum of three (3) inches of hot mix asphalt pavement over four (4) inches of crushed gravel top or base course for the parking and drive areas. In addition, a 6-inch layer of compacted structural fill is recommended below the pavement section to promote drainage below the base layer. Should the upper layer of bedrock be highly fractured, this drainage layer may be omitted as determined by ALLWEST. For areas where pavement is proposed over native sand soils, we recommend a pavement section consisting of a minimum of three (3) inches of hot mix asphalt pavement over five (5) inches of crushed gravel top or base course for the parking and drive areas. The recommended pavement section assumes occasional delivery truck traffic with single axle loads up to 10 tons on the pavement surface. Our pavement design section was calculated assuming an average daily traffic rate of 250 vehicles with a 2 percent truck factor. Should traffic volumes or axle loadings vary from these assumptions, ALLWEST should be given an opportunity to review our recommendations and provide additional recommendations, if necessary.

We recommend specifying crushed gravel top or base course meeting the requirements of the Washington Department of Transportation (WSDOT) Standard Specification 9-03.9(3) for crushed gravel top or base course. We recommend the structural fill (subbase, if used) consist of a relatively free-draining, coarse gravel or sand with less than 7 percent by weight passing a No. 200 sieve. We recommend the asphalt concrete pavement meet the requirements of WSDOT Standard Specification for Hot Mix Asphalt (HMA) Class ½ inch asphalt concrete pavements. We recommend the crushed gravel base be compacted to a minimum of 95 percent of its modified Proctor maximum dry density (ASTM D1557). We recommend the asphaltic concrete surface be compacted to a minimum of 92 percent of the Rice density. If a high percentage of truck traffic is expected, we should be notified so we can review our pavement recommendations and provide revisions if necessary.

## 9.0 ADDITIONAL RECOMMENDED SERVICES

ALLWEST shall be provided with final plans prior to construction so that we can evaluate if the recommendations provided herein apply. We also recommend ALLWEST be retained to provide construction observation and materials testing to verify the soil and geologic conditions and the report recommendations are incorporated into the actual construction. In-place density testing should be performed by an experienced engineering technician at the time of construction to verify the recommended levels of compaction are achieved. If we are not retained to provide the recommended plan review and construction observation services, we cannot be responsible for soil engineering related construction errors or omissions.

## 10.0 EVALUATION LIMITATIONS

This report has been prepared to assist the planning and design of the proposed Experience Senior Living Facility to be located northwest of the intersection of East 55th Avenue and South Fiske Street in Spokane, Washington. Our services consist of professional opinions and conclusions made in accordance with generally accepted geotechnical engineering principles and practices in our local area at the time this report was prepared. This acknowledgement is in lieu of all warranties either expressed or implied.

## 11.0 PROFESSIONAL ACKNOWLEDGEMENT

This report was prepared by me or under my direct supervision and I am a duly registered engineer under the laws of the State of Washington.

Todd DeMico, P.E.
Engineering Services Manager

TODD BRANDON DEMICO
STATE OF WASHINGTON
57233
REGISTERED
PROFESSIONAL ENGINEER
11/19/2020

APPENDIX A
SITE LOCATION MAP, NRCS SOILS MAP,
BORING LOCATION MAP


Google Earth

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

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Spokane Valley, Washington
**www.allwesttesting.com**

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|---|
| **FIGURE A-1 - SITE LOCATION MAP** |
| PROPOSED EXPERIENCE SENIOR LIVING FACILITY |
| NW OF 55TH AVENUE AND SOUTH FISKE STREET |
| SPOKANE, WASHINGTON |
| Client Name: NexCore Group |
| Project No.: 220-252G |
| Date: November 2020 |


Anticipated Soil Types:
7106—Urban land, sandy substratum, 0 to 15 percent slopes
7150—Urban land—Seaboldt, disturbed complex, 0 to 3 percent slopes

United States Department of Agriculture (USDA)
National Resources Conservation Service (NRCS)
Web Soil Survey

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

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| **FIGURE A-2 - NRCS SOILS MAP** |
| PROPOSED EXPERIENCE SENIOR LIVING FACILITY |
| NW OF 55TH AVENUE AND SOUTH FISKE STREET |
| SPOKANE, WASHINGTON |
| Client Name: NexCore Group |
| Project No.: 220-252G |
| Date: November 2020 |


Approximate Boring Location

Approximate Test Pit Location

Approximate Infiltration Test Location

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

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|---|
| **FIGURE A-3 - TEST PIT/BORING LOCATION MAP** |
| PROPOSED EXPERIENCE SENIOR LIVING FACILITY |
| NW OF 55TH AVENUE AND SOUTH FISKE STREET |
| SPOKANE, WASHINGTON |
| Client Name: NexCore Group |
| Project No.: 220-252G |
| Date: NOVEMBER 2020 |

# APPENDIX B
# LOGS OF TEST PITS/BORINGS,
# UNITED SOIL CLASSIFICATION SYSTEM

# LOG OF BORING

| PROJECT: | Proposed Experience Senior Living Facility<br>NW of East 55th Avenue and South Fiske Street<br>Spokane, Washington<br>ALLWEST Project No. 220-252G | BORING: **B-01** |
|---|---|---|
| | | LOCATION: **See Test Pit and Borehole Location Map** |
| | | DATE: **10/23/2020** SCALE: **1" = 5'** |

| Depth 0.0 | ASTM D2487 Symbol | Description of Materials | N | WL | Tests or Notes |
|---|---|---|---|---|---|
| 1.0 | FILL | Silty GRAVEL with Sand, poorly-graded, fine to coarse grained, angular clasts, brown, damp. (Undocumented Fill) | NA | | |
| 4.0 | Rx | Basalt, slightly weathered, jointed with minor staining on fracture faces, dark gray, fine grained, hard. | | | Significant sand infill from 4.0 to 7.0 feet below ground surface. |
| 13.0 | Rx | Basalt, slightly weathered, jointed with trace vesicles, fine sand, and trace clay infill, dark brown, fine grained, medium hard. | | | |
| 14.0 | Rx | Basalt, slightly weathered, jointed, dark brown, fine grained, hard to very hard. | | | |
| 16.0 | Rx | Basalt, slightly weathered, jointed with trace vesicles, fine sand, and trace clay infill, dark brown, fine grained, medium hard. | | ▼ | Groundwater encountered at 16.5 feet below ground surface. |
| 20.0 | Rx | Basalt, slightly weathered, jointed and vesicular with fine sand and trace clay infill, black, fine grained, medium hard. | | | |
| | | Boring terminated at 20.0 feet.<br><br>Groundwater encountered at approximately 16.5 feet below ground surface.<br><br>Boring backfilled with bentonite upon completion. | | | |

(See Report and Standard Plates for elevation and descriptive terminology.)

# LOG OF BORING

| | |
|---|---|
| PROJECT: **Proposed Experience Senior Living Facility**<br>**NW of East 55th Avenue and South Fiske Street**<br>**Spokane, Washington**<br>**ALLWEST Project No. 220-252G** | BORING: **B-02**<br>LOCATION: **See Test Pit and Borehole Location Map**<br>DATE: **10/23/2020** SCALE: **1" = 5'** |

| Depth 0.0 | ASTM D2487 Symbol | Description of Materials | N | WL | Tests or Notes |
|---|---|---|---|---|---|
| 3.0 | FILL | Silty GRAVEL with Sand, poorly-graded, fine to coarse grained, angular clasts, brown, damp. (Undocumented Fill) | NA | | |
| 5.0 | Rx | Slightly weathered, jointed with minor staining on fracture faces, dark gray, fine grained, strong (R4), BASALT. | | | |
| 9.0 | Rx | Basalt, slightly weathered, jointed with trace vesicles, fine sand and clay infill, dark brown, fine grained, medium hard. | | | Significant fine sand infill from 5.0 to 8.0 feet below ground surface. |
| 14.0 | Rx | Basalt, slightly weathered, vesicular, broken with clay infill, black, fine grained, medium hard. | | | Rock coring performed at 9.9 to 11.4 feet below ground surface.<br><br>Sample returned consisted of rubbled basalt with heavy clay infill.<br>Recovery: 1.5 ft / 1.5 ft RQD = 0 |
| 15.0 | Rx | Basalt, slightly weathered, jointed, dark brown, fine grained, hard to very hard. | | | |
| 20.0 | Rx | Basalt, slightly weathered, jointed and vesicular with fine sand and trace clay infill, black, fine grained, medium hard. | | ▽ | Due to condition of rock air rotary drilling resumed from 11.4 to 20.0 feet (end of hole). |
| | | Boring terminated at 20.0 feet.<br><br>Groundwater level not measured due to drilling with water.<br><br>Boring backfilled with bentonite upon completion. | | | |

(See Report and Standard Plates for elevation and descriptive terminology.)

# LOG OF BORING

| | | | |
|---|---|---|---|
| PROJECT: | **Proposed Experience Senior Living Facility**<br>**NW of East 55th Avenue and South Fiske Street**<br>**Spokane, Washington**<br>**ALLWEST Project No. 220-252G** | BORING: **B-03**<br>LOCATION: **See Test Pit and Borehole Location Map** | |
| | | DATE: **10/23/2020** | SCALE: **1" = 5'** |

| Depth 0.0 | ASTM D2487 Symbol | Description of Materials | N | WL | Tests or Notes |
|---|---|---|---|---|---|
| 0.5 | FILL | Silty GRAVEL with Sand, poorly-graded, fine to coarse grained, angular clasts, brown, damp. (Undocumented Fill) | NA | | |
| 3.5 | Rx | Basalt, slightly weathered, jointed with minor staining on fracture faces, dark gray, fine grained, hard. | | | Significant fine sand infill from 3.5 to 8.0 feet below ground surface. |
| 12.0 | Rx | Basalt, slightly weathered, jointed with trace vesicles, fine sand and trace clay infill, dark brown, fine grained, medium hard. | | | |
| 14.0 | Rx | Basalt, slightly weathered, jointed, dark brown, fine grained, hard to very hard. | | | |
| 16.0 | Rx | Basalt, slightly weathered, jointed with trace vesicles, fine sand, and trace clay infill, dark brown, fine grained, medium hard. | | ▼ | Groundwater encountered at 15.0 feet below ground surface. |
| 20.0 | Rx | Basalt, slightly weathered, jointed and vesicular with fine sand and trace clay infill, black, fine grained, medium hard. | | | |

Boring terminated at 20.0 feet.

Groundwater encountered at approximately 15.0 feet below ground surface upon completion of drilling.

Boring backfilled with bentonite upon completion.

(See Report and Standard Plates for elevation and descriptive terminology.)

# LOG OF TEST PIT

| | |
|---|---|
| PROJECT: **Proposed Experience Senior Living Facility**<br>**NW of East 55th Avenue and South Fiske Street**<br>**Spokane, Washington**<br>**ALLWEST Project No. 220-252G** | TEST PIT: **TP-01**<br>LOCATION: **See Test Pit and Borehole Location Map**<br>DATE: **10/15/2020** SCALE: **1" = 2.5'** |

| Depth 0.0 | ASTM D2487 Symbol | Description of Materials | WL | Tests or Notes |
|---|---|---|---|---|
| 0.8 | FILL | Well-graded GRAVEL and SAND, fine gravel, fine to coarse sand, angular clasts, dark gray, damp.<br>(Undocumented Fill - 5/8-inch minus Crushed Surfacing Top Course) | | Test Pit located at the toe of stockpiled crushed rock.<br>Organic content testing performed on sample collected from 0.8 to 2.0 feet.<br>Organic content = 4.4%. |
| 2.0 | ML | SILT with Sand and Organics, dark brown, damp.<br>(Buried Topsoil) | | |
| 2.7 | ML | SILT with sand, low plasticity, fine sand, brown, damp.<br>(Loess) | | |
| 4.5 | SP | Poorly-graded SAND with Silt, fine to coarse sand, trace gravel, gray brown, dry.<br>(Alluvium) | | |
| | | Test Pit terminated at 4.5 feet on suspected basalt bedrock.<br><br>No groundwater or caving observed during excavation.<br><br>Test Pit backfilled upon completion. | | |

(See Report and Standard Plates for elevation and descriptive terminology.)

# LOG OF TEST PIT

| | |
|---|---|
| PROJECT: **Proposed Experience Senior Living Facility**<br>**NW of East 55th Avenue and South Fiske Street**<br>**Spokane, Washington**<br>**ALLWEST Project No. 220-252G** | TEST PIT: **TP-02**<br>LOCATION: **See Test Pit and Borehole Location Map**<br>DATE: **10/15/2020** SCALE: **1" = 2.5'** |

| Depth 0.0 | ASTM D2487 Symbol | Description of Materials | WL | Tests or Notes |
|---|---|---|---|---|
| 0.6 | ML | SILT with Sand and Organics, dark brown, damp. (Topsoil) | | |
| 2.5 | SM | Silty SAND, low plasticity, fine sand, brown, damp. (Loess) | | Soil classification testing performed on sample collected from a depth of 0.6 to 2.5 feet. |
| 4.1 | SM | Silty SAND with GRAVEL and COBBLES, fine to coarse sand, trace gravel, gray brown, dry. (Alluvium) | | Soil classification testing performed on sample collected from a depth of 2.5 to 4.1 feet. |
| | | Test Pit terminated at 4.1 feet on suspected basalt bedrock.<br><br>No groundwater or caving observed during excavation.<br><br>Test Pit backfilled upon completion. | | |

(See Report and Standard Plates for elevation and descriptive terminology.)

# LOG OF TEST PIT

| PROJECT: | Proposed Experience Senior Living Facility<br>NW of East 55th Avenue and South Fiske Street<br>Spokane, Washington<br>ALLWEST Project No. 220-252G | TEST PIT: **TP-03** |
|---|---|---|
| | | LOCATION: **See Test Pit and Borehole Location Map** |
| | | DATE: **10/15/2020** SCALE: **1" = 2.5'** |

| Depth 0.0 | ASTM D2487 Symbol | Description of Materials | WL | Tests or Notes |
|---|---|---|---|---|
| 4.0 | FILL | Silty GRAVEL with Sand and Debris,poorly-graded, fine to coarse grained, angular clasts, brown, damp.<br>(Uncontrolled Fill)<br>(Debris includes romex and metal pipe.) | | |
| | | Test Pit terminated at 4.0 feet on suspected basalt bedrock.<br><br>No groundwater or caving observed during excavation.<br><br>Test Pit backfilled upon completion. | | |

(See Report and Standard Plates for elevation and descriptive terminology.)

# LOG OF TEST PIT

ALLWEST Testing & Engineering

| PROJECT: | Proposed Experience Senior Living Facility<br>NW of East 55th Avenue and South Fiske Street<br>Spokane, Washington<br>ALLWEST Project No. 220-252G | TEST PIT: **TP-04**<br>LOCATION: **See Test Pit and Borehole Location Map**<br>DATE: **10/15/2020** SCALE: **1" = 2.5'** |
|---|---|---|

| Depth 0.0 | ASTM D2487 Symbol | Description of Materials | WL | Tests or Notes |
|---|---|---|---|---|
| 0.5 | FILL | Silty GRAVEL with Sand and Debris,poorly-graded, fine to coarse grained, angular clasts, brown, damp.<br>(Uncontrolled Fill)<br>(Debris includes bricks and metal pipe.) | | |
| | | Test Pit terminated at 0.5 feet on suspected basalt bedrock.<br><br>No groundwater or caving observed during excavation.<br><br>Test Pit backfilled upon completion. | | |

(See Report and Standard Plates for elevation and descriptive terminology.)

# LOG OF TEST PIT

| PROJECT: | **Proposed Experience Senior Living Facility**<br>**NW of East 55th Avenue and South Fiske Street**<br>**Spokane, Washington**<br>**ALLWEST Project No. 220-252G** | TEST PIT: **TP-05**<br>LOCATION: **See Test Pit and Borehole Location Map** |
|---|---|---|
| | | DATE: **10/15/2020** SCALE: **1" = 2.5'** |

| Depth 0.0 | ASTM D2487 Symbol | Description of Materials | WL | Tests or Notes |
|---|---|---|---|---|
| 1.5 | FILL | Well-graded GRAVEL with Sand, Cobbles and Boulders, fine to coarse sand, angular clasts, dark gray, damp.<br>(Undocumented Fill - Recycled Asphalt) | | Test Pit located near stockpile. |
| | | Test Pit terminated at 1.5 feet on suspected basalt bedrock.<br><br>No groundwater or caving observed during excavation.<br><br>Test Pit backfilled upon completion. | | |

(See Report and Standard Plates for elevation and descriptive terminology.)

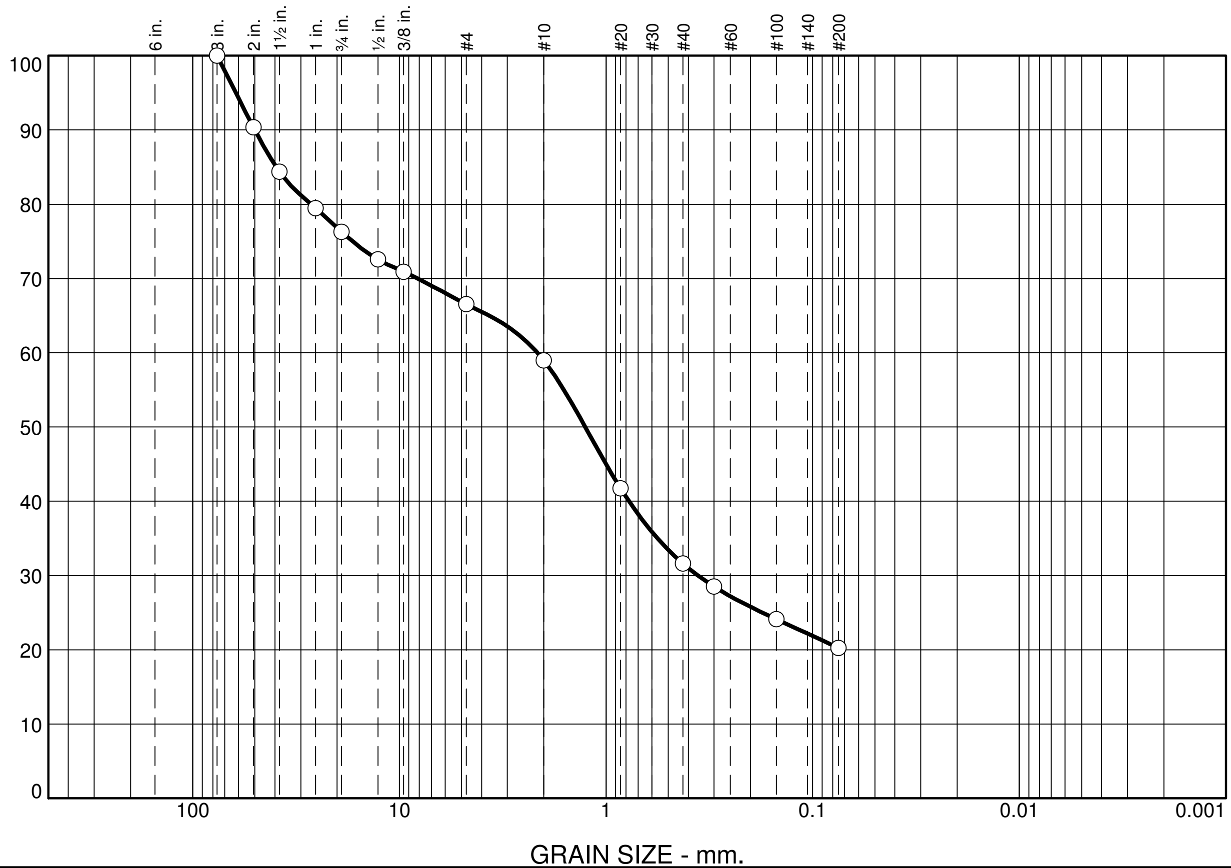
# Unified Soil Classification System

| MAJOR DIVISIONS | | | SYMBOL | TYPICAL NAMES |
|---|---|---|---|---|
| COARSE GRAINED SOILS | GRAVELS | CLEAN GRAVELS | GW | Well-Graded Gravel, Gravel-Sand Mixtures. |
| | | | GP | Poorly-Graded Gravel, Gravel-Sand Mixtures. |
| | | GRAVELS WITH FINES | GM | Silty Gravel, Gravel-Sand-Silt Mixtures. |
| | | | GC | Clayey Gravel, Gravel-Sand-Clay Mixtures. |
| | SANDS | CLEAN SANDS | SW | Well-Graded Sand, Gravelly Sand. |
| | | | SP | Poorly-Graded Sand, Gravelly Sand. |
| | | SANDS WITH FINES | SM | Silty Sand, Sand-Silt Mixtures. |
| | | | SC | Clayey Sand, Sand-Clay Mixtures. |
| FINE GRAINED SOILS | SILTS AND CLAYS<br><br>LIQUID LIMIT LESS THAN 50% | | ML | Inorganic Silt, Silty or Clayey Fine Sand. |
| | | | CL | Inorganic Clay of Low to Medium Plasticity, Sandy or Silty Clay. |
| | | | OL | Organic Silt and Clay of Low Plasticity. |
| | SILTS AND CLAYS<br><br>LIQUID LIMIT GREATER THAN 50% | | MH | Inorganic Silt, Elastic Silt, Micaceous Silt, Fine Sand or Silt. |
| | | | CH | Inorganic Clay of High Plasticity, Fat Clay. |
| | | | OH | Organic Clay of Medium to High Plasticity. |
| Highly Organic Soils | | | PT | Peat, Muck and Other Highly Organic Soils. |

ALLWEST

# APPENDIX C
# FIELD INFILTRATION/SOILS LABORATORY TEST RESULTS

# Particle Size Distribution Report




| % Cobbles | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0 | 24 | 9 | 8 | 27 | 12 | 20 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 3" | 100 | | |
| 2" | 90 | | |
| 1 1/2" | 84 | | |
| 1" | 79 | | |
| 3/4" | 76 | | |
| 1/2" | 73 | | |
| 3/8" | 71 | | |
| #4 | 67 | | |
| #10 | 59 | | |
| #20 | 42 | | |
| #40 | 32 | | |
| #50 | 29 | | |
| #100 | 24 | | |
| #200 | 20 | | |

* (no specification provided)

**Soil Description**

Silty sand with gravel and cobbles

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

$D_{90}$= 50.0326 $D_{85}$= 39.4493 $D_{60}$= 2.1483
$D_{50}$= 1.2588 $D_{30}$= 0.3590 $D_{15}$=
$D_{10}$= $C_u$= $C_c$=

**Classification**

USCS= SM AASHTO=

**Remarks**

B.Borer sampled 11/6/20

**Location:** TP-2
**Sample Number:** S220-0556 **Depth:** @ 2.5'-4.1' **Date:** 11/16/20

ALLWEST

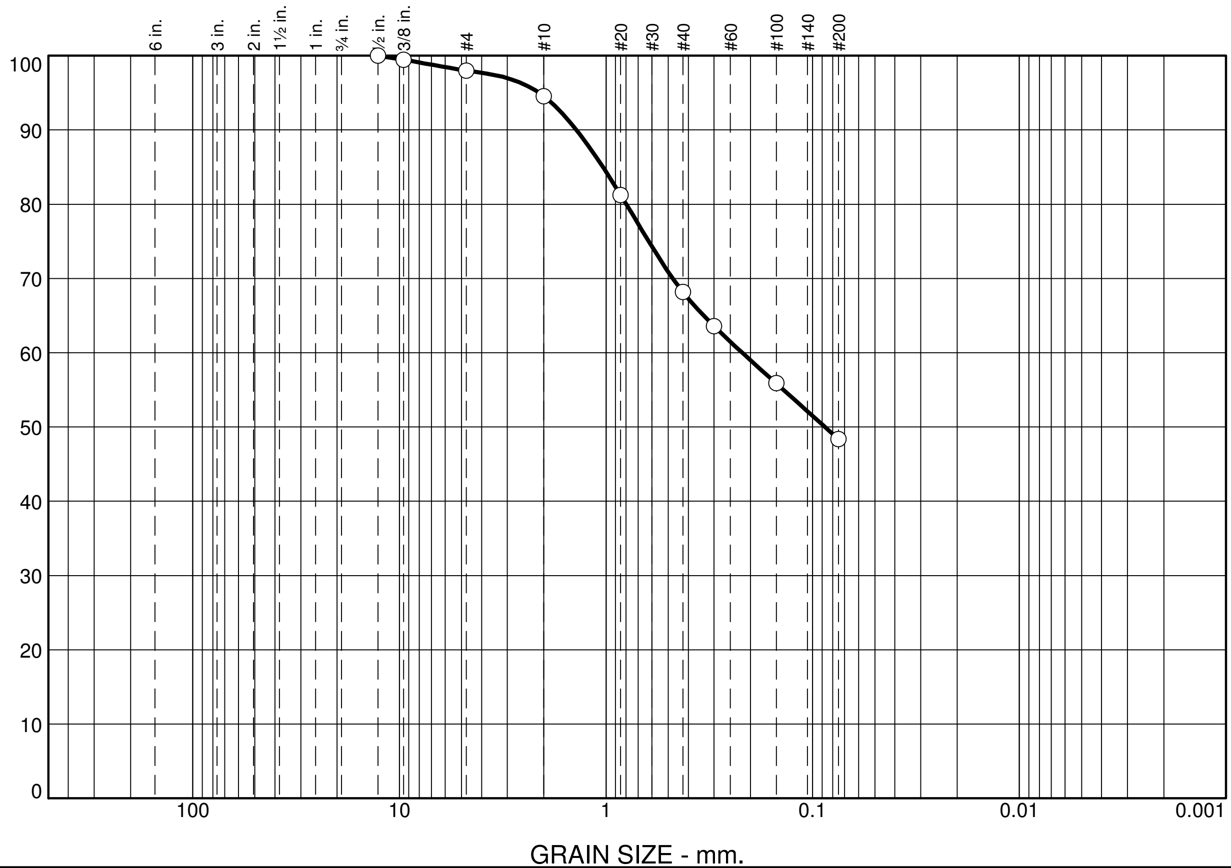
**Client:** NexCore Group, LP
**Project:** Experiance Senior Living

**Project No:** 220-252G **Reviewed by:** TBD



**Tested By:** C.McDonald **Checked By:** D.Schmitz

# Particle Size Distribution Report




| % Cobbles | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0 | 0 | 2 | 3 | 27 | 20 | 48 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 1/2" | 100 | | |
| 3/8" | 99 | | |
| #4 | 98 | | |
| #10 | 95 | | |
| #20 | 81 | | |
| #40 | 68 | | |
| #50 | 64 | | |
| #100 | 56 | | |
| #200 | 48 | | |

* (no specification provided)

**Soil Description**

Silty sand

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

$D_{90}$= 1.3903 $D_{85}$= 1.0346 $D_{60}$= 0.2190
$D_{50}$= 0.0871 $D_{30}$= $D_{15}$=
$D_{10}$= $C_u$= $C_c$=

**Classification**

USCS= SM AASHTO=

**Remarks**

B.Borer sampled 11/6/20

**Location:** TP-2
**Sample Number:** S220-0558 **Depth:** @ 0.6'-2.5' **Date:** 11/16/20

ALLWEST

**Client:** NexCore Group, LP
**Project:** Experiance Senior Living

**Project No:** 220-252G **Reviewed by:** TBD

**Tested By:** C.McDonald **Checked By:** D.Schmitz

ALLWEST Testing & Engineering

# LABORATORY SUMMARY

| | | | |
|---|---|---|---|
| **Project Name:** | Experiance Snior Living | **Source:** | On-site |
| **Client Name:** | NexCore Group, LP | **Project No.:** | 220-252G |
| **Location:** | Fiske Road and E. 53rd Avenue Spokane, WA 99206 | **Date:** | 11/16/2020 |

| | | | |
|---|---|---|---|
| LOCATION: | TP-1 @ 0.8'-2.0' | | |
| LAB SAMPLE NUMBER: | S220-0557 | | |
| SAMPLED BY: | B.Borer | | |
| DATE SAMPLED: | 11/6/2020 | | |
| MATERIAL: | Silty sand | | |
| | | | |

| TEST PERFORMED | SPECS | RESULTS | SPECS | RESULTS | SPECS | RESULTS |
|---|---|---|---|---|---|---|
| Organic Content in Soils | | | | | | |
| AASHTO T267 / ASTM D2974 | | 4.4% | | | | |



**Reviewed By:** TBD **Date:** 11/17/2020

690 W. Capstone Court • Hayden, ID 83835 • (208) 762-4721 • Fax (208) 762-0942
3005 N. Industrial Lane, 5th Street Spokane Valley, Wa 99216 • (509) 534-4411 • Fax (509) 534-9326
2127 2nd Avenue North • Lewiston, ID 83501 • (208) 743-5710 • Fax (208) 743-8270

## SINGLE RING INFILTROMATER TEST RESULTS

Testing performed in general conformance with the *Spokane Regional Stormwater Manual Appendix 4D - Single Ring Infiltrometer Test Method (April 2008)*

**Proposed Experience Senior Living Facility**

**Project No. 220-252G**

Test Location: INF-1

10/28/2020

Flow Meter Used: Master Meter

| ELAPSED TIME [MIN] | METER READING [FT³] | ELAPSED TIME BETWEEN READINGS [MIN] | DEPTH TO WATER [FT] | INCREMENTAL FLOW [FT³] | FLOW RATE - Q [CFS] | TEST PHASE |
|---|---|---|---|---|---|---|
| 0 | 4258.3 | 0 | 0.00 | --- | 0 | PRESOAK |
| 10 | 4259.6 | 10.0 | 0.17 | 1.3 | 2.25E-03 | BEGIN TEST |
| 20 | 4260.0 | 10.0 | 0.15 | 0.4 | 6.67E-04 | |
| 30 | 4260.4 | 10.0 | 0.17 | 0.4 | 7.00E-04 | |
| 40 | 4260.9 | 10.0 | 0.17 | 0.4 | 6.83E-04 | |
| 50 | 4261.2 | 10.0 | 0.17 | 0.4 | 6.17E-04 | |
| 60 | 4261.6 | 10.0 | 0.15 | 0.4 | 7.00E-04 | |
| 70 | 4262.0 | 10.0 | 0.17 | 0.3 | 5.67E-04 | |
| 80 | 4262.3 | 10.0 | 0.17 | 0.4 | 6.00E-04 | |
| 90 | 4262.6 | 10.0 | 0.17 | 0.3 | 4.83E-04 | |
| 100 | 4263.0 | 10.0 | 0.17 | 0.3 | 5.33E-04 | |
| 110 | 4263.2 | 10.0 | 0.17 | 0.3 | 4.67E-04 | |
| 120 | 4263.6 | 10.0 | 0.17 | 0.3 | 5.33E-04 | END TEST |
| 130 | | 10.0 | 0.67 | | | FALLING HEAD |
| 140 | | 10.0 | 1.08 | | | |
| 143 | | 3.0 | DRY | | | |

**AVERAGE "Q" (CFS) = 5.31E-04**

**Infiltration Rate, I ($cfs/ft^2$) = 6.76E-04**

**Infiltration Rate, I (Inch/hr) = 29.2**

**Permeability Rate, K (feet/second)**

K = (Q*L/A*H)

where: Q = Stabilized Flow Rate Near the End of the Constant Head Portion of the Test

H = Constant level of water within the ring (inches)

L = Depth of soil contained within the ring (inches)

A = Area of soil inside the ring (square feet)

**K (ft/sec)= 3.86E-04**