VI. Stormwater Report

Storm Drainage Report Hope Village Campus Modifications

J.O. SGL 18-073B

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October 1, 2025

REGISTERED PROFESSIONAL ENGINEER
16551
Patrick A. Sisul
OREGON
MAY 18, 1993
PATRICK A. SISUL

EXPIRATION DATE: 6/30/2026

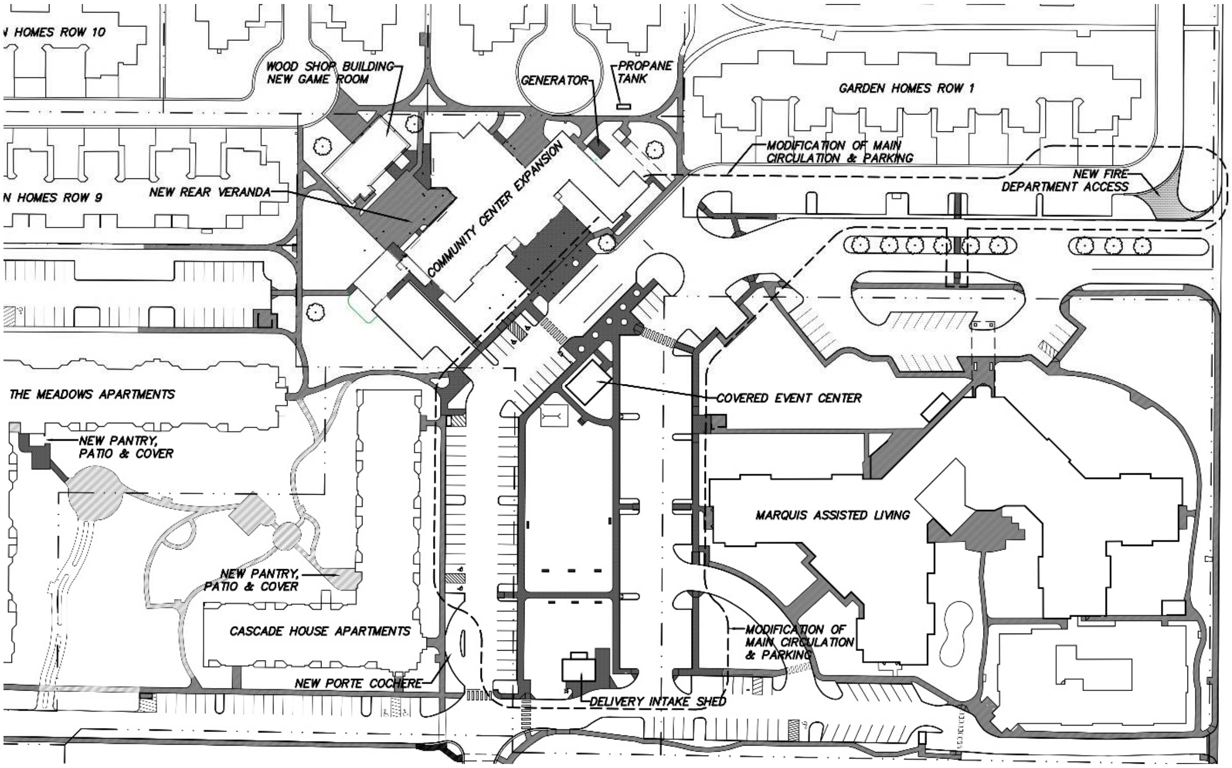
Hope Village Original Campus, Site Modifications:

Site Description:

The Hope Village campus is in south Canby and is bounded between SW 13th and SW 18th Avenues to the north and south, and S Fir and S Ivy Streets to the west and east. In the image below, the original Hope Village campus is noted in red and the South Campus portion of Hope Village is noted in white.

Proposed Project:

The proposed improvements will occur within the original portion of the Hope Village campus. Projects will include a minor covered patio addition to the rear of The Meadows apartment building, a minor covered patio addition to the rear of the Cascade House apartment building and the addition of a Porte Cochere on the front, expansion of the Community Center, creation of an Emergency Fire access near the S Ivy Street entrance, and a reimagination of the central parking area located between Cascade House, Marquis, the South Campus, and the Community Center. A map of the site improvements is on the following page:



Existing Storm Drainage System:

Storm drainage runoff from inside the Hope Village campus is collected and conveyed to several existing private drywells for underground injection. No stormwater runoff from inside Hope Village flows offsite into a public storm drainage system. The South Campus has newer drywells that do not receive any drainage from the original campus.

Eleven drywells are within the area impacted by the proposed renovations. The impervious area draining to each drywell is generally known through the review of historical plans for the campus, but there are gaps in the information, particularly with roof drain connections and with buildings that have had additions.

According to Frank Howard, Director of Development for Hope Village, who has worked at Hope Village for more than 13 years, there are no areas of concern with the private storm drainage system within the Hope Village campus. There have been occasional ponding issues through the years, but those issues were related to clogged catch basin grates or occasionally a clogged pipe within a catch basin. To his knowledge, Hope Village has not had a drywell overwhelmed by inflow. There have been issues related to the public stormwater infiltration basins along S Fir Street backing up water on S Fir Street, but those infiltration basins do not drain to drywells within Hope Village. One of the drainage issues along S Fir Street was connected to a new public drywell in 2025, so that issue has been at least partially addressed by the City of Canby.

Public Works Design Standard:

The City of Canby Public Works Design Standards, December 2019, Section 4.312 Infiltration Facilities, subsection c.3 states, “Drywells (UIC’s) shall be located to collect up to a maximum of one half of an acre of runoff. Gutter flow shall be limited to 400-500 lineal feet, provided the flow does not exceed 3” in height against the curb line. Any variation from this guideline shall be based on field infiltration tests.”

However, the City of Canby rule of thumb back in the 1990’s and 2000’s, when the original Hope Village campus was constructed, was that a drywell could accommodate runoff from one acre, not one-half acre. The one-acre rule of thumb number is likely what the original Hope Village campus was designed to meet, as from our mapping of the system, several drywells receive runoff from more than one-half acre of impervious area, with a maximum of 1.15 acres of impervious area. This south area of Canby has drywells with very high infiltration rates than can accommodate larger tributary areas to each drywell, which is demonstrated in the Performance Testing section below.

Geotechnical Reports & Drywell Performance Testing:

On October 21, 2013, at the request of Scott Investments, GeoPacific Engineering, Inc. conducted drywell performance testing of drywells located on SE 16th Avenue in the Dinsmore Estates subdivision east of Hope Village for the purpose of establishing a maximum rate of flow for a 26-foot-deep drywell in this part of Canby. Using three fire hoses connected to three separate fire hydrants, none of the drywells tested could be filled to its maximum capacity. A November 5, 2013 Report from GeoPacific Engineering states that “Drywells one through four may be assumed to infiltrate at a maximum estimated rate of 2,500 gpm.”[1] The Dinsmore Estates drywells are located approximately 1500 southeast of the Community Center. 2,500 gpm equals 5.57 cfs.

In March 2025, drywell performance testing was conducted on a 25-year-old, 28-foot drywell located between the Cascade House apartment building and The Meadows apartment building within Hope Village (Drywell #9) within this report. GeoTech Solutions, Inc. monitored the drywell performance testing[2] and reported than the drywell was filled to a depth of approximately 5 feet and then the drainage rate was monitored for how quickly the water exited the drywell. The infiltration rate was determined to be 140 in^3/hr $/in^2$. A design infiltration rate of 105 in^3/hr $/in^2$ was recommended by geotechnical engineer, Don Rondema MS, PE, GE. This rate can be applied to the bottom of the drywell, if protected by an upstream sediment capture, and to the portion of the sides of the drywell that are embedded in native gravels and cobbles. This rate will be used for the design of any new drywells.

According to the geotechnical bore pits completed for Hope Village, gravels, cobbles, and possibly boulders begin onsite at an approximate depth of 6 feet bgs. Accordingly, the design will assume that infiltration will occur along the side walls between the bottom of the drywell at 26 feet bgs to an elevation of 7 feet bgs, one foot below where the gravels and cobbles begin. No side wall infiltration is assumed to occur above 7 feet bgs. Assuming a 48-inch inside diameter and a two-foot wide granular zone around the drywell, 105 in^3/hr $/in^2$ would result in a peak infiltration rate of 1.46 cfs.

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[1] GeoPacific Engineering, Inc., James D. Imbrie PE, CEG, Infiltration Testing of As-Built Drywells, Dinsmore Estates, Canby, Oregon, November 5, 2013. See Appendix A.

[2] Geotech Solutions, Inc., Don Rondema PE, Report of Geotechnical Engineering Services, Hope Village Improvements, Canby, Oregon, August 2, 2025. See Appendix B.

Design Storm:

The table in Section 4.301.a of the City of Canby Public Works Design Standards (December 2019) identifies the minimum design storm recurrence interval for a variety of storm drainage facilities. The table identifies that the following facilities shall be designed using a design storm with the noted recurrence intervals below:

Infiltration Facilities: UIC, LID elements	10 years
Minor: Streets, curbs, gutters, inlets, catch basin & connector drains	10 years
Major: Laterals (collectors) <250 tributary acres	10 years

1973 NOAA Atlas 2, Volume X and U.S. Department of Agriculture Isolpluvials for 24-hour storms in Oregon identify the 10-year, 24-hour storm event for Canby as being between 3.0 and 3.5 inches of precipitation, while the 25-year, 24-hour storm event would have between 3.5 and 4.0 inches of precipitation.

The Oregon Department of Transportation TranGIS website identifies the 10-year, 24-hour precipitation for this area of Canby as being 3.03 inches and the 25-year, 24-hour precipitation as being 3.53 inches.

A 24-hour storm having total rainfall of 4.0 inches would exceed the 25-year event for both sources. We will use this larger design storm for conservatism.

Calculation Methodology:

Stormwater flow from the developed site will be calculated using the Santa Barbara Urban Hydrograph (SBUH) method using a Type 1A SCS storm.

Runoff Curve Numbers:

Per the Web Soil Survey, the site has Latourell Loam soils, 53A, hydrologic soil group B. The CN number for per Table 3.5.2B SCS Western Washington Runoff Curve Numbers, King County Washington Surface Water Design Manual, 11/92, is noted below:

Paved streets, Sidewalks, Driveway CN = 98

Runoff from landscaped areas will be ignored, as we are being conservative on the size of the storm, the onsite soils infiltrate well, there are not many landscaping area drains located on the campus, and Hope Village has been installing infiltration basins to address any landscaping wet zones, rather than pipe them to drywells.

Time of Concentration:

Time of concentration is made up of a combination of sheet flow, shallow concentrated flow and pipe flow and is the total travel time from the hydraulically most distance point in the drainage basin to the point of disposal. For the purposes of this study, because we are looking at a developed site, and because little of the landscaped areas drain to the onsite drywells, the time of concentration will be assumed to be 5 minutes, which is the minimum time of concentration used for stormwater runoff calculations, such as these.

King County SBUH Computations for 1.5-acre of impervious surfacing 25 year, 24-Hour Storm, with 4.0 inches of precipitation:

In order to get a sense of how much impervious area a drywell designed to the design rate can accommodate, we will calculate the peak runoff from a 1.5-acre of impervious surface and route it to a drywell. Calculations will be performed using the King County SBUH Programs.

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SBUH/SCS METHOD FOR COMPUTING RUNOFF HYDROGRAPH

STORM OPTIONS:

1 - S.C.S. TYPE-1A
2 - 7-DAY DESIGN STORM
3 - STORM DATA FILE

SPECIFY STORM OPTION: 1

S.C.S. TYPE-1A RAINFALL DISTRIBUTION
ENTER: FREQ(YEAR), DURATION(HOUR), PRECIP(INCHES)
25,24,4.0
----------------------------------------------------------------------
******************** S.C.S. TYPE-1A DISTRIBUTION ********************
*********  25-YEAR  24-HOUR STORM  ****  4.00" TOTAL PRECIP. *********
----------------------------------------------------------------------
ENTER: A(PERV), CN(PERV), A(IMPERV), CN(IMPERV), TC FOR BASIN NO.  1
0,80,1.50,98,5.0

DATA PRINT-OUT:

  AREA(ACRES)     PERVIOUS     IMPERVIOUS   TC(MINUTES)
                 A     CN      A     CN
       1.5        .0   80.0    1.5   98.0        5.0

  PEAK-Q(CFS)   T-PEAK(HRS)    VOL(CU-FT)
      1.57          7.67          20500              ← Peak Runoff Rate

ENTER [d:][path]filename[.ext] FOR STORAGE OF COMPUTED HYDROGRAPH:
HVMP.25
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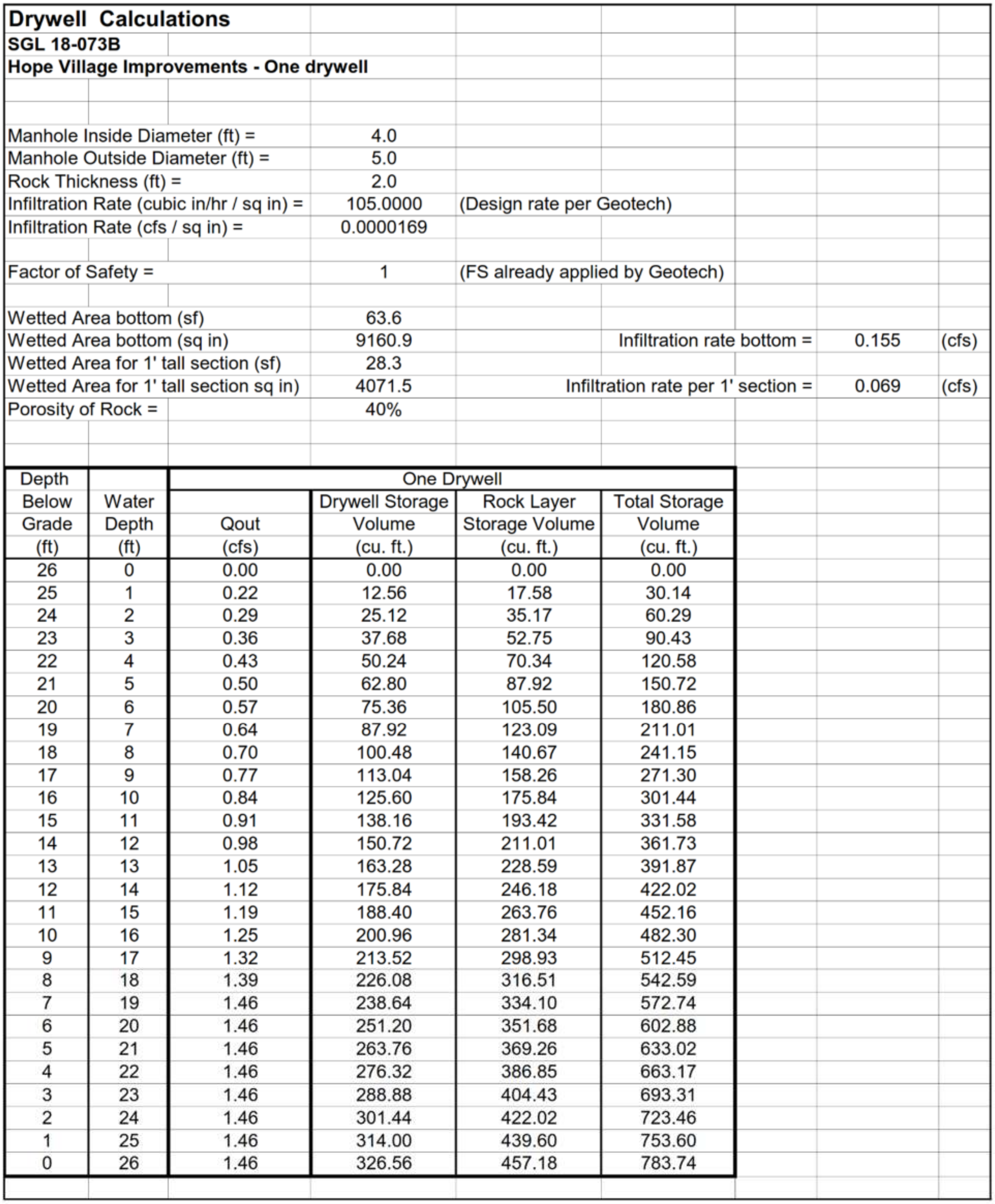
Design Storm Routing Through Drywell:

Based on the drywell design rate noted in the Drywell Performance Testing section of this report, the design will assume that infiltration will occur along the bottom of the drywell and the side walls between the bottom (at 26 feet bgs) and 7 feet bgs. No side wall infiltration will be assumed to occur above 7 feet bgs. Using a 48-inch inside diameter, six-inch walls, and a two-foot-wide granular zone around the drywell, 105 in^3/hr /in^2 would result in a peak infiltration rate of 1.46 cfs. Per the spreadsheet on the following page:



Drywell Calculations

SGL 18-073B

Hope Village Improvements - One drywell

Manhole Inside Diameter (ft) = 4.0
Manhole Outside Diameter (ft) = 5.0
Rock Thickness (ft) = 2.0
Infiltration Rate (cubic in/hr / sq in) = 105.0000 (Design rate per Geotech)
Infiltration Rate (cfs / sq in) = 0.0000169

Factor of Safety = 1 (FS already applied by Geotech)

Wetted Area bottom (sf) 63.6
Wetted Area bottom (sq in) 9160.9 Infiltration rate bottom = 0.155 (cfs)
Wetted Area for 1' tall section (sf) 28.3
Wetted Area for 1' tall section sq in) 4071.5 Infiltration rate per 1' section = 0.069 (cfs)
Porosity of Rock = 40%

		One Drywell			
Depth Below Grade (ft)	Water Depth (ft)	Qout (cfs)	Drywell Storage Volume (cu. ft.)	Rock Layer Storage Volume (cu. ft.)	Total Storage Volume (cu. ft.)
26	0	0.00	0.00	0.00	0.00
25	1	0.22	12.56	17.58	30.14
24	2	0.29	25.12	35.17	60.29
23	3	0.36	37.68	52.75	90.43
22	4	0.43	50.24	70.34	120.58
21	5	0.50	62.80	87.92	150.72
20	6	0.57	75.36	105.50	180.86
19	7	0.64	87.92	123.09	211.01
18	8	0.70	100.48	140.67	241.15
17	9	0.77	113.04	158.26	271.30
16	10	0.84	125.60	175.84	301.44
15	11	0.91	138.16	193.42	331.58
14	12	0.98	150.72	211.01	361.73
13	13	1.05	163.28	228.59	391.87
12	14	1.12	175.84	246.18	422.02
11	15	1.19	188.40	263.76	452.16
10	16	1.25	200.96	281.34	482.30
9	17	1.32	213.52	298.93	512.45
8	18	1.39	226.08	316.51	542.59
7	19	1.46	238.64	334.10	572.74
6	20	1.46	251.20	351.68	602.88
5	21	1.46	263.76	369.26	633.02
4	22	1.46	276.32	386.85	663.17
3	23	1.46	288.88	404.43	693.31
2	24	1.46	301.44	422.02	723.46
1	25	1.46	314.00	439.60	753.60
0	26	1.46	326.56	457.18	783.74

The values in the spreadsheet will be used by the routing routine (on the following page) to model flow and storage volume associated with a 26-foot deep, 4-foot diameter drywell.

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RESERVOIR ROUTING INFLOW/OUTFLOW ROUTINE

SPECIFY [d:][path]filename[.ext] OF ROUTING DATA  HopeMP.dat
 DISPLAY ROUTING DATA (Y or N)? y

ROUTING DATA:

STAGE(FT)   DISCHARGE(CFS)   STORAGE(CU-FT)   PERM-AREA(SQ-FT)
    .00            .00                .0               .0
   1.00            .22              30.1               .0
   2.00            .29              60.3               .0
   3.00            .36              90.4               .0
   4.00            .43             120.6               .0
   5.00            .50             150.7               .0
   6.00            .57             180.9               .0
   7.00            .64             211.0               .0
   8.00            .70             241.2               .0
   9.00            .77             271.3               .0
  10.00            .84             301.4               .0
  11.00            .91             331.6               .0
  12.00            .98             361.7               .0
  13.00           1.05             391.9               .0
  14.00           1.12             422.0               .0
  15.00           1.19             452.2               .0
  16.00           1.25             482.3               .0
  17.00           1.32             512.4               .0
  18.00           1.39             542.6               .0
  19.00           1.46             572.7               .0
  20.00           1.46             602.9               .0
  21.00           1.46             633.0               .0
  22.00           1.46             663.2               .0
  23.00           1.46             693.3               .0
  24.00           1.46             723.5               .0
  25.00           1.46             753.6               .0
  26.00           1.46             783.7               .0

AVERAGE PERM-RATE:    .0 MINUTES/INCH

ENTER [d:][path]filename[.ext] OF COMPUTED HYDROGRAPH:
HVMP.25

INFLOW/OUTFLOW ANALYSIS:

  PEAK-INFLOW(CFS)   PEAK-OUTFLOW(CFS)   OUTFLOW-VOL(CU-FT)li
        1.57                1.44                20402

  INITIAL-STAGE(FT)  TIME-OF-PEAK(HRS)   PEAK-STAGE-ELEV(FT)
         .00               7.83                 18.74      ← Peak Stage

  PEAK STORAGE:      560 CU-FT

ENTER [d:][path]filename[.ext] FOR STORAGE OF COMPUTED HYDROGRAPH:
HVMP25.pst
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According to the Routing Routine, a 24-hour design storm event having 4.0 inches of rainfall on a 1.5-acre impervious drainage basin, would fill a 26-foot-deep drywell to a depth of 18.74 feet and the storage would be filled to approximately 71 percent of the maximum capacity of the drywell.

One drywell is adequate to accommodate the anticipated runoff from at least 1.5 acres of impervious area for a 24-hour storm of 4.0 inches.

Existing Drywell Basin Area Changes:

Redevelopment of the site will modify the tributary drainage basins for multiple drywells within the portion of the site being altered. The table below identifies the pre-development and post-development contributing areas (within the area of alteration) for each drywell and the total drywell tributary basin area (where known).

	Pre-Development Impervious Area		Post-Development Impervious Area		
Drywell No	Estimated Basin Area (sf)	Basin Area (within “project” area) (sf)	Basin Area (within “project” area) (sf)	Change (sf)	Estimated Basin Area (Ac)
DW-1	30,400	3,147	4,868	+1,721	0.74
DW-2	14,500	4,294	5,384	+1,090	0.36
DW-3	24,000	13,686	27,315	+13,629	0.86
DW-4	50,269	24,907	18,148	-6,759	1.00
DW-5	20,850	20,850	20,966	+116	0.48
DW-6	18,400	18,400	19,268	+868	0.44
DW-7	33,300	28,721	19,897	-8,824	0.46
DW-8	31,555	17,428	18,787	+1,359	0.76
DW-9	23,050	0	695	+695	0.55
DW-10	34,750	0	301	+301	0.80
DW-11	unknown	5,019	5,612	+593	unknown

DW-1: Drywell 1 is north of the main driveway, near S Ivy Street. The impervious area draining to DW-1 will increase slightly due to the addition of the emergency fire access, but by less than 1,800 sf. With the additional area, DW-1 will have a basin are of less than 1-acre. It is anticipated that DW-1 can accommodate this minor increase in impervious area.

DW-2: Drywell 2 is north of the main driveway, between DW-1 & DW-3. The impervious area draining to DW-2 will increase by less than 1,100 sf. The tributary area to DW-2 will remain less than 0.5 acre, therefore it is anticipated that DW-2 can accommodate this minor increase in impervious area.

DW-3: Drywell 3 is north of the main driveway, between DW-2 & the Community Center. The impervious area draining to DW-3 will increase by 13,629 sf, bringing the overall impervious basin area to roughly 0.86-acre. It is anticipated that DW-3 can accommodate the additional runoff.

DW-4: Drywell 4 is located between the Community Center and Marquis. The drywell currently has an impervious basin area of 1.15 acres. The impervious area draining to DW-4 will decrease by 6,759 sf and the total impervious area draining to DW-4 will be reduced to 1.00 acre.

DW-5: Drywell 5 is in the landscaping area in front of the Community Center. The impervious area draining to DW-5 will increase by slightly more than 100 sf. The total impervious area draining to DW-5 will remain less than 0.5 acre, so it is anticipated that DW-5 will be able to accommodate the minor increase in area.

DW-6: Drywell 6 is behind the Community Center in the patio area. The impervious area draining to DW-6 will increase by 868 sf. The total impervious area will be less than 0.5 Ac and it is anticipated that DW-6 can accommodate this minor increase in impervious area.

DW-7: Drywell 7 is in the Central Parking Area and it receives some runoff from Cascade House and majority from the parking area. The impervious area draining to DW-7 will decrease by 8,824 sf.

DW-8: Drywell 8 is also in the Central Parking Area. It receives runoff from Cascade House, a portion of the parking area in front of Cascade House and the driveway to the South Campus. The impervious area draining to DW-8 will increase by 1,359 sf to 0.76-acre total impervious area. It is anticipated that DW-8 can accommodate this minor increase in impervious area.

DW-9: Drywell 9 is on the rear side of Cascade House. It receives roof drain runoff from the rear side of Cascade House and it also receives runoff from area drains in the courtyard. The new improvement to the rear of Cascade House will increase the impervious runoff area by 695 sf. It is anticipated that DW-9 can accommodate this minor increase in impervious area.

DW-10: Drywell 10 is on the rear side of The Meadows. It receives roof drain runoff from The Meadows apartment building roof drains. The new improvement to the rear of The Meadows will increase the impervious runoff area by 301 sf and will increase the tributary impervious area to 0.80-acre. It is anticipated that DW-10 can accommodate this minor increase in impervious area.

DW-11: Drywell 11 is in a landscape area near the SW entrance to the Marquis facility. It receives some roof drain runoff from the Marquis building and parking lot and driveway access area along the south and southwestern sides of Marquis. The exact limits of the drainage basin are unknown because a new drywell was added when Marquis was remodeled in 2019-2020 with the Memory Care expansion. It appears that much of the south driveway that used to drain to DW-11 has now been cut off and directed a different drywell. How the remodel may have impacted roof drain runoff, is not known. Based on what drains to DW-11 today, the impervious area draining to DW-11 will increase by 593 sf. It is anticipated that DW-11 can accommodate the runoff from this minor additional area, as it appears that the drywell used to serve a larger parking lot prior to the Marquis Memory Care Wing expansion.

All existing drywells within the area being impacted will have tributary impervious basin areas of 1.0 acre, or less, and therefore they should accommodate the tributary basin changes proposed. The drywell having the largest tributary area, DW-4 will have its current tributary drainage area decreased by 6,759 sf as compared to the basin that is currently draining to it. ✓

Appendix A: Letter from GeoPacific Engineering re: Dinsmore Estates Drywell Testing

GeoPacific
Engineering, Inc.

November 5, 2013
Project No. 07-1252

Scott Investments
130 SW 2nd Avenue
Canby, Oregon 97013
Email: Tomscott @scott-investments.com

Copy: Email PatSisul@Sisulengineering.com

SUBJECT: INFILTRATION TESTING OF AS-BUILT DRYWELLS
DINSMORE ESTATES
CANBY, OREGON

This report presents the results of recent and higher capacity infiltration testing conducted by GeoPacific Engineering, Inc. (GeoPacific) for the above referenced project. Six years ago some dry well testing was performed, but the maximum flow rate achieved was only 300 to 400 gpm and no appreciable level of water was observed in the well. The purpose of our recent testing was to determine a constant head infiltration rate of the as-built drywells with a larger flow rate. Drywells were designed by Sisul Engineering Inc. for subsurface disposal of storm water. A total of four drywells were constructed within SE 16th Avenue. We measured the depth of the dry wells **one** (westernmost) and **four** (easternmost) at 27.5 and 26.5 feet deep feet, respectively. The contractor previously indicated that clean gravels were encountered during the construction of all of the drywells at a depth of 5 to 6 feet and gravel extended to the bottom of the drywell.

INFILTRATION TESTING

On October 31, 2013, GeoPacific observed infiltration tests in Drywell one and Drywell four. Tests were performed by using three 2½ inch fire hoses drained into the drywell from the only three fire hydrants in the area. Water meter readings, the volume of water in the water truck and the start and end time of each test was recorded. The rate of infiltration did not allow a significant depth of water to accumulate in the drywells, and falling head tests were not feasible since they drained in less than 30 seconds. An observed constant head infiltration rate was obtained by calculating the total volume of water introduced into the drywell divided by the time required for the dry well to empty.

The observed, constant head infiltration test rates of Drywell one and four was 633 gpm with 2.5 feet of head pressure. Tests were performed from about one to three hours long. No change was noted after the 3 hour test. We understand there is approximately 16 vertical feet from the bottom of the drywells to the inverts of the incoming pipes. It is our opinion that the tested drywells are capable of infiltrating stormwater at a rate of at least 4 times what was delivered into the wells. Infiltration testing was limited by the ability to pump water into the drywell. For planning purposes, Drywells one through four may be assumed to infiltrate at a maximum estimated rate of 2,500 gpm.

ENVIRONMENTAL CONSIDERATIONS AND LIMITATIONS

Subsurface stormwater disposal systems have the potential to affect groundwater quality, since they provide a more direct pathway for infiltrating surface water to reach groundwater aquifers. Consequently, disposal systems should be constructed and maintained in accordance with Oregon Department of Environmental Quality requirements for groundwater protection. Systems receiving runoff from pavement areas, typically include water quality elements such as oil traps, filters or similar measures.

Infiltration test methods and procedures attempt to simulate the as-built conditions of the planned subsurface disposal system or systems. However, due to natural variations in soil properties, actual infiltration rates may vary from the measured and/or recommended design rates. Storm events in excess of the design event are inevitable. All systems should be constructed such that potential overflow is discharged in a controlled manner away from structures.

We appreciate this opportunity to be of service. Please call if you have any questions.

Sincerely,

GeoPacific Engineering, Inc.

REGISTERED PROFESSIONAL ENGINEER
14743PE
OREGON
JAN. 23, 1990
JAMES D. IMBRIE

EXPIRES: 06/30/2015

James D. Imbrie, P.E., C.E.G.
Geotechnical Engineer

Appendix B: Geotechnical Investigation for Hope Village, GeoTech Solutions, Inc.

Report of Geotechnical Engineering Services

Hope Village Improvements
Canby, Oregon

Geotech Solutions Inc.

August 2, 2025

GSI Project: hopevillage-25-1-gi

Geotech Solutions Inc.

August 2, 2025 *hopevillage-25-1-gi*

Hope Village; frank@hopevillage.org

Cc: Sisul Engineering; patsisul@sisulengineering.com

REPORT OF GEOTECHNICAL ENGINEERING SERVICES
Hope Village Improvements – Canby, Oregon

As authorized, we appreciate the opportunity to present this report for the proposed new Hope Village Improvements in Canby, Oregon. Improvements are to include expansion of the community center, canopies for the Meadows and Cascade House Apartments, a fire lane, and main driveway reconstruction with hardscaping that may include low MSE retaining walls. The structures are to be one to two-story wood framed. The purpose of our services was to provide geotechnical engineering recommendations for use in design by others, as well as testing of an existing dry well near the proposed Lodges. Specifically, our scope of work included the following:

- Provide experienced project management including client communications, management of field and subcontracted services, report writing, analyses, and invoicing.
- Review geologic maps and vicinity geotechnical information as indicators of subsurface conditions.
- Complete a site reconnaissance to observe surface features relevant to geotechnical issues, such as topography, vegetation, presence and condition of springs, exposed soils, and evidence of previous grading.
- Identify exploration locations and coordinate possible location conflicts with utilities.
- Explore subsurface conditions by drilling seven solid stem auger borings with a light duty trailer mounted drill rig (this will reduce extensive landscape disturbance from test pits) to refusal in gravels, as well as 3 pavement cores down to the subgrade.
- Classify and sample the materials encountered and maintain a detailed log of the explorations.
- Complete lab testing of select samples to aid in classification.
- Observe and analyze same day testing of the drywell, equipment and water provided by others.
- Provide recommendations for earthwork including fill materials, seasonal material usage, use of granular working pads, cut and fill slope inclinations, compaction criteria, utility trench backfill, need for subsurface drainage, and reuse of demolition materials.
- Provide recommendations for footing foundations, including embedment, bearing pressure, settlement estimates, resistance to lateral loads, a seismic site class and the need for subsurface drainage.
- Provide a report summarizing our observations and recommendations.

SITE OBSERVATIONS AND CONDITIONS

Surface Conditions

Hope Village is located immediately southeast of the intersection of SW 13th Avenue and S ivy Street in Canby, Oregon. The proposed improvement are generally in the southwest quadrant of the site, depicted in the exploration area shown on the attached ***Site Plan***. The site is bordered by residential property on all sides. The existing site topography is generally flat. Site-specific topographic survey information was not available at the time of our report. The ground surface in most proposed improvement areas is in lawn, with some sidewalk hardscaping and low gently slopes.

Subsurface Conditions

Subsurface conditions at the site were explored on March 26, 2025 by completing 7 borings (B-1 through B-7) to depths of drilling refusal which ranged from 3.6 to 7.5 feet. Approximate exploration locations are shown on the attached ***Site Plan.*** Specific subsurface conditions observed at each exploration are described in the attached ***Boring Logs.***

In general, subsurface conditions include a thin surficial topsoil zone ranging in thickness from 5 to 7 inches. The topsoil zone soils are underlain by brown silt with occasional fine roots and variable sand and gravel content. The silt was generally medium stiff at the time of our exploration in the wet season, with a few shallow samples being soft. Moisture contents ranged from 16-28%. At depths of 3.6 to 7.5 feet we generally encountered dense to very dense gravels, cobbles, and boulders, with cemented sand above gravels in B-5 and B-7. The boulder content in this area can vary from occasional to more than half the content. Boulders observed nearby were up to 2.5 to 3 feet in diameter.

Results of moisture content testing are attached.

Site Geology

We reviewed the Geologic Map of the Canby and Oregon City Quadrangles (DOGAMI, Bulletin 99) as part of our evaluation. The site is in an area of mapped Lacustrine sediments including the Willamette Silt formation (Qws) and deltaic deposits (Qdg) of sand, gravel, and boulders 'up to 8 feet in diameter'. Subsurface conditions encountered in our explorations are consistent with the mapped site geology.

Groundwater – We did not observe groundwater seepage to the depths explored. Ground water is mapped (GSI Water Solutions, 2013) near elevation 110 feet in the site area, with a depth to seasonal high ground water of roughly 60 feet. Due to the presence of silty near surface soils, perched ground water conditions could exist during extended periods of wet weather.

CONCLUSIONS AND RECOMMENDATIONS

General

Based on the results of our explorations, laboratory testing, and engineering analyses, it is our opinion that the site can be developed following the recommendations contained herein. Key geotechnical issues include soft surface conditions, possible difficult/enlarged excavation due to boulders, and moisture sensitivity of the upper silty soils to wet season grading. The proposed structures can be supported on shallow spread footings bearing as recommended herein. Specific geotechnical recommendations are provided in the following sections.

Boulders were encountered at depth nearby and boulders or cobbles resulted in drilling refusal at 3 to 8 feet. Based on available geologic information and our experience in the site vicinity, it is possible for some boulders to exceed 8 feet in diameter, though most are less than 3 feet. If utilities and excavations extend below boulder elevations, special excavation techniques and enlarged excavations may be required. Project budgets and schedules must include a contingency for rock/boulder excavation and increased backfill volumes due to boulder removal and caving.

Site Preparation

General - Prior to earthwork construction, the site should be prepared by removing any existing structures, utilities, pavement and any loose surficial or undocumented fill. Any excavation resulting from the aforementioned preparation should be brought back to grade with structural fill. Site preparation for earthwork will also require the removal of the root zone and topsoil/fill zone soils, from all pavement, building, and new fill areas. Deeper stripping depths may be required in areas near trees or shrubs.

Root balls from trees and shrubs may extend several feet and grubbing operations can cause considerable subgrade disturbance. All disturbed material should be removed to undisturbed subgrade and backfilled with structural fill. In general, roots greater than one-inch in diameter should be removed as well as areas of concentrated smaller roots.

Stabilization and Soft Areas - After stripping, we must be contacted to evaluate the exposed subgrade. This evaluation can be done by proof rolling in dry conditions or probing during wet conditions. Soft areas will require over-excavation and backfilling with well graded, angular crushed rock compacted as structural fill, overlying a separation geosynthetic such as a Propex Geotex 801 or equivalent. A geogrid may also be required in particularly soft areas, such as a Gridpro BXP-12 or equivalent punched and drawn biaxial geogrid.

Working Blankets and Haul Roads - Construction equipment must not operate directly on the subgrade, as it is susceptible to disturbance and softening. Any remaining site pavement can be used for this. Rock working blankets and haul roads placed over a geosynthetic in a thickened advancing pad can be used to protect silt subgrades. We recommend that sound, angular, pit run or crushed basalt with no more than 6 percent passing a #200 sieve be used to construct haul roads and working blankets, overlying the preceding separation geosynthetic. Working blankets must be at least 10 inches thick, and haul roads at least 18 inches thick.

As an alternative to the methods described above, reuse of till zone and native soils may be possible by soil amendment using portland cement (after topsoil /primary root zone removal) covered with 4 inches of clean well graded crushed rock. Amendment requires an experienced contractor using specialty spreading and mixing equipment. Typically, 6% cement in one or two mixing passes is used for an amendment (i.e., mix) depth of 12 inches (a soil weight of 100 pcf is typically used for the quantity calculation). However, the materials used, and quantities can vary based on moisture and organic contents, plasticity, and required amendment depth. Soils with soft wet silt under the treated zone may require deeper treatment or alternative grid and rock fill methods. Compaction and grading of amended soils must be completed within 4 hours of mixing, and the amended soil must be allowed to cure for 4 days prior to trafficking. Generally, 50 percent of mixed particles should pass a No. 4 sieve.

The permeability of amended soil is very low. The surface of amended soils in building and pavement areas must therefore be sloped at a minimum of 0.5 percent to prevent collection of surface water during construction. Amended soil must be removed from all landscape areas prior to planting.

The preceding rock and amendment thicknesses are the minimum recommended. Subgrade protection is the responsibility of the contractor and thicker sections may be required based on subgrade conditions during construction and type and frequency of construction equipment.

Earthwork

***Fill* -** The on-site fine grained inorganic silt and sand can be used for structural fill if properly moisture conditioned and if all debris and deleterious materials are removed. Use of material with more than roughly 6% silt will not be feasible during wet conditions, and generally none of the upper soils meet this criterion. Cobbles larger than 6 inches should be removed from the gravel and cobble unit if it is to be used in aerial fills, and all boulders must be removed for its use as trench backfill or areal fill. Once moisture contents are within 3 percent of optimum, the material must be compacted to at least 92 percent relative to ASTM D1557 (modified proctor) using a tamping foot type compactor. Fill must be placed in lifts no greater than 10 inches in loose thickness. In addition to meeting density specifications, fill will also need to pass a proof roll using a loaded dump truck, water truck, or similar size equipment.

In wet conditions, fill must be imported granular soil with less than 6 percent fines, such as clean crushed or pit run rock. This material must also be compacted to 95 percent relative to ASTM D1557.

***Trenches* -** Utility trenches may encounter perched ground water seepage and moderate to severe caving must be expected where seepage is present or in the gravels. Flowing soil conditions can occur in the sand unit where seepage is present. We did not encounter seepage in our test pits. Shoring of utility trenches will be required for depths greater than 4 feet and where groundwater seepage is present. We recommend that the type and design of the shoring system be the responsibility of the contractor, who is in the best position to choose a system that fits the plan of operation.

Our explorations all encountered cobbles with potential boulders, starting as depths of 3 to 8 feet. Difficult and enlarged excavations and/or special excavation techniques will be required if trenches extend below depths where boulders are present. Project budgets and schedules must include a contingency for rock/boulder excavation and increased backfill volumes.

Depending on the excavation depth and amount of groundwater seepage, dewatering may be necessary for construction of underground utilities. Flow rates for dewatering are likely to vary depending on location, soil type, and the season during which the excavation occurs. The dewatering systems, if necessary, must be capable of adapting to variable flows.

Pipe bedding must be installed in accordance with the pipe manufacturers' recommendations. If groundwater is present in the base of the utility trench excavation, we recommend over excavating the trench by 12 to 18 inches and placing trench stabilization material in the base. Trench stabilization material must consist of well-graded, crushed rock or crushed gravel with a maximum particle size of 4 inches and be free of deleterious materials. The percent passing the U.S. Standard No. 200 Sieve must be less than 5 percent by weight when tested in accordance with ASTM C 117.

Trench backfill above the pipe zone must consist of well graded, angular crushed rock or sand fill with no more than 7 percent passing a #200 sieve. Trench backfill must be compacted to 92 percent relative to ASTM D-1557, and construction of hard surfaces, such as sidewalks or pavement, must not occur within one week of backfilling.

***Slopes* -** Permanent slopes should be inclined no steeper than 2H:1V for slopes up to 5 feet high. The face of fill slopes should be cut back into compacted materials with a smooth bucket excavator. If steeper fill slopes are desired, we should be consulted to evaluate use of amended soils or grid reinforcement. Erosion control is critical to maintaining fill slopes and should be as described for cut slopes. Drainage should be routed away from slope faces.

Infiltration

dry wells embedded into clean portions of the gravel and cobble unit may be suitable for on-site disposal of storm water. Due to caving concerns, dry wells should be installed prior to building foundations, in which case dry wells can be within 5 feet of footings. Otherwise, they must be 10 feet or at least 1.5 times their depth, whichever is greater, away from footings. The following paragraphs provide geotechnical recommendations for dry wells. Actual system design will be completed by the project civil engineer based on storm water volumes and rates.

Based on the testing the 28-foot deep dry well in the Lodges expansion area, the dry wells soils had a raw infiltration rate of 140 in^3/hour per in^2. The base of this well was relatively clean of sediment and was reported as only used for downspouts. Testing was done under 5 feet of head in a falling head condition. Increased head in the drywell would increase the rate somewhat. Therefore, we recommend using a design infiltration rate of 105 in^3/hour per in^2 applied to the portion of the sides of the dry wells that are embedded within gravels and cobbles with less than 10% fines. This includes a reduction factor of 2 then a 50% increase for a higher head condiiton and can also be used for the base of drywells that are protected by upstream sediment capture. Clean gravel or cobble fill with less than 2% fines can be used for filling trenches or the perimeter of dry wells in the perforated zone. Care must be taken to design any drywell or pipe perforations and fill to avoid loss of backfill into the dry well. If geosynthetics are used over perforations, flow rates of the geosynthetic must exceed the design perforation outflow rate by a factor of 3. Clean, well graded, angular crushed rock or pit run rock should be used overlying the perforation zone fill. All backfill must be compacted until well keyed as structural fill.

We must be contacted during infiltration system construction to confirm that exposed conditions are consistent with those observed during our infiltration testing. Systems should be sized by the civil engineer according to design storm water volumes and rates. Minimum embedment should also be specified by the civil engineer.

Confirmation Testing and Maintenance - Testing of infiltration systems is required to confirm the design infiltration rate as actual subsurface conditions and infiltration rates can vary widely. Flexibility for adaptation and expansion of infiltration systems should be incorporated into the design and construction, with contingencies included in the project budget and schedule. Infiltration systems need to be maintained free of debris and silt in order to function properly.

Seismic Design

General - In accordance with the State of Oregon Structural Specialty Code (SOSSC) and based on our explorations and experience in the site vicinity, the subject project should be evaluated using the parameters associated with Site Class C.

Liquefaction - Liquefaction occurs in loose, saturated, granular soils. Strong shaking, such as that experienced during earthquakes, causes the densification and the subsequent settlement of these soils. Given the generally flat topography, unsaturated near surface conditions, and the soil type and dense conditions encountered in our explorations, the risk of liquefaction related structurally-damaging deformations in proposed building areas is low.

Shallow Foundations

Based on the provided information regarding building type and anticipated structural loads as previously stated, the proposed structure can be supported on shallow spread foundations bearing in the native medium stiff or stiffer silt or medium dense gravels or on properly constructed structural fill bearing on these units. Footings should be embedded at least 18 inches below the lowest adjacent, exterior grade. Footings can be designed for an allowable net bearing pressure of 2,500 psf when founded on medium stiff or better undisturbed native silt. The preceding bearing pressure can be increased to 5,000 psf for temporary wind and seismic loads.

Continuous footings should be no less than 18 inches wide, and pad footings should be no less than 24 inches wide. Resistance to lateral loads can be obtained by a passive equivalent fluid pressure of 350 pcf against suitable footings, ignoring the top 12 inches of embedment, and by a footing base friction coefficient of 0.35. These include a safety factor of 1.5. Properly founded footings are expected to settle less than a total of 1 inch, with less than ½ inch differentially. Footings adjacent to slopes up to 2H:1V should have a minimum horizontal setback of 5 feet from the face of the slope.

If footing construction is to occur in wet conditions, a few inches of crushed rock should be placed at the base of footings to reduce subgrade disturbance and softening during construction. Granular soils loosed by footing excavation could be "re-seated" during compaction of the crushed rock protection layer.

Slabs

Floor slab loads up to 250 psf are expected to induce less than one inch of settlement. A minimum of six inches of clean, angular crushed rock with no more than 5 percent passing a #200 sieve is recommended for underslab rock. Prior to slab rock placement the subgrade will need to be evaluated by us by probing or observing a proof rolling using a fully loaded truck. Underslab rock should be compacted to 92 percent compaction relative to ASTM D1557 and should be proof rolled as well. In addition, any areas contaminated with fines must be removed and replaced with clean rock. If the base rock is saturated or trapping water, this water must be removed prior to slab placement.

Some flooring manufacturers require specific slab moisture levels and/or vapor barriers to validate the warranties on their products. A properly installed and protected vapor flow retardant can reduce slab moisture. If moisture sensitive floor coverings or operations are planned, we recommend a vapor barrier be used. Typically, a reinforced product or thicker product (such as a 15 mil STEGO wrap) can be used. Experienced contractors using special concrete mix design and placement have been successful placing concrete directly over the vapor barrier which overlies the rock. This avoids the issue of water trapped in the rock between the slab and vapor barrier, which otherwise requires removal. In either case, slab moisture should be tested/monitored until it meets floor covering manufacturer's recommendations.

Drainage

General - Perimeter foundation drains are required around all exterior foundations. The surface around building perimeters should be sloped to drain away from the buildings. As stated previously, our retaining wall recommendations are based on drained conditions.

Foundation Drains - Foundation and retaining wall drains should consist of a two-foot-wide zone of drain rock encompassing a 4-inch diameter perforated pipe, all enclosed with a non-woven filter fabric. The drain rock should have no more than 2 percent passing a #200 sieve and should extend to within one foot of the ground surface. The geosynthetic should have an AOS of a #70 sieve, a minimum permittivity of 1.0 sec^{-1}, and a minimum puncture resistance of 80 pounds (such as Propex Geotex 601 or equivalent). Alternatively, a composite drain board such as an Amoco 500/520 could be used. In either case one foot of low permeability soil (such as the on-site silt) should be placed over the fabric at the top of the drain to isolate the drain from surface runoff.

Pavement

Existing Pavement - Pavement coring was completed at 3 locations in the planned pavement rehabilitation area at the locations C-1 through C-3 shown on the attached ***Site Plan***. Cores showed 3 to 4 inches of ½" dense graded hot mix asphalt concrete, overlying 6 to 7 inches of crushed rock base, overlying medium stiff silt subgrade. Pavement condition includes an eroded seal coat/slurry seal with evident raveling and transverse and longitudinal cracking. Alligator cracking was limited, and obvious rutting was not apparent. Extensive patching had been completed in the drive area in front of the community center. This pavement likely has less than 50% of its original capacity remaining. It may be possible to extend the life of the pavement several years with additional crack sealing and seal coating, or to grind and replace 2 inches for an additional 8-10 years. For longer life, pavement replacement in dry conditions is recommended as described in the following section.

New Asphalt Concrete - At the time of this report we did not have specific information regarding the type and frequency of expected traffic. We therefore developed asphalt concrete pavement thicknesses for areas exposed to passenger vehicles only and areas that also include up to 5 mixed trucks per day based on a 20-year design life. Traffic volumes can be revised if specific data is available.

Our pavement analyses are based on AASHTO methods and subgrade of stiff silt or structural fill as discussed in this report, and having a resilient modulus of at least 6,000 psi. We have also assumed that roadway construction will be completed during an extended period of dry weather, and that the existing base rock can be reused in place where grades allow. The results of our analyses based on these parameters are provided in the following table.

Traffic	18k ESAL's	AC (inches)	CRB (inches)
Passenger Vehicle Only	-	3	6
Up to 5 Trucks Per Day	37,000	4	6

The thicknesses listed in the preceding table are the minimum acceptable for construction during an extended period of dry summer weather. Increased rock thickness will be required for construction during wet conditions. Asphalt concrete should be ½" dense graded HMAC conforming to ODOT specifications. Crushed rock must conform to ODOT base rock standards and have less than 6 percent passing the #200 sieve. Asphalt concrete must be compacted to a minimum of 91 percent of a Rice Density.

Subgrade Preparation - The pavement subgrade must be prepared in accordance with the ***Earthwork** and **Site Preparation*** recommendations presented in this report. All pavement subgrades must pass a proof roll prior to paving. Soft areas must be repaired per the preceding ***Stabilization*** section.

LIMITATIONS AND OBSERVATION DURING CONSTRUCTION

We have prepared this report for use by Hope Village, Inc. and the design and construction teams for this project only. The information herein could be used for bidding or estimating purposes but must not be construed as a warranty of subsurface conditions. We have made observations only at the aforementioned locations and only to the stated depths. These observations do not reflect soil types, strata thicknesses, water levels or seepage that may exist between observations. We must be consulted to observe all foundation bearing surfaces, subgrade stabilization, proof rolling of slab and pavement subgrades, installation of structural fill, subsurface drainage, and cut and fill slopes. We must be consulted to review final design and specifications to see that our recommendations are suitably followed. If any changes are made to the anticipated locations, loads, configurations, or construction timing, our recommendations may not be applicable, and we must be consulted. The preceding recommendations must be considered preliminary, as actual soil conditions may vary. For our recommendations to be final, we must be retained to observe actual subsurface conditions encountered. Our observations will allow us to interpret actual conditions and adapt our recommendations if needed.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with the generally accepted practices in this area at the time this report was prepared. No warranty, expressed or implied, is given.

◆

We appreciate the opportunity to work with you on this project and look forward to our continued involvement. Please contact us if you have any questions.

Sincerely,

Don Rondema, MS, PE, GE
Principal

REGISTERED PROFESSIONAL ENGINEER
OREGON
DON RONDEMA
Expires 12/31/26

Attachments – Site Plan, Guidelines for Classification of Soil, Boring Logs, Moisture Contents

BASE PHOTO FROM ONXMAPS

Geotech Solutions Inc.

SITE PLAN

hopevillage-25-1-gi

GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

Relative Density	Standard Penetration Resistance (N-values) blows per foot
very loose	0 - 4
loose	4 - 10
medium dense	10 - 30
dense	30 - 50
very dense	over 50

Description of Consistency for Fine-Grained (Cohesive) Soils

Consistency	Standard Penetration Resistance (N-values) blows per foot	Torvane Undrained Shear Strength, tsf
very soft	0 - 2	less than 0.125
soft	2 - 4	0.125 - 0.25
medium stiff	4 - 8	0.25 - 0.50
stiff	8 - 15	0.50 - 1.0
very stiff	15 - 30	1.0 - 2.0
hard	over 30	over 2.0

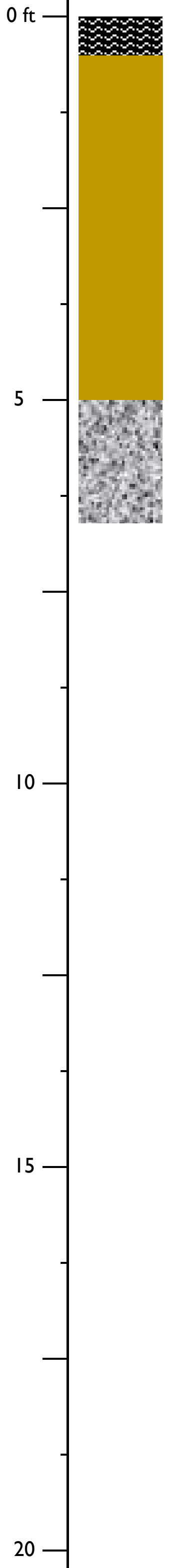
Grain-Size Classification

Description	Size
Boulders	12 - 36 in.
Cobbles	3 - 12 in.
Gravel	¼ - ¾ in. (fine)
	¾ - 3 in. (coarse)
Sand	No. 200 - No. 40 Sieve (fine)
	No. 40 - No. 10 sieve (medium)
	No. 10 - No. 4 sieve (coarse)
Silt/Clay	Pass No. 200 sieve

Modifier for Subclassification

Adjective	Percentage of Other Material In Total Sample
Clean/Occasional	0 - 2
Trace	2 - 10
Some	10 - 30
Sandy, Silty, Clayey, etc.	30 - 50

Soil and Rock Description



5” soft rooty SILT TOPSOIL/SOD

Soft to medium stiff, brown SILT with trace fine to coarse sand and occasional fine roots down to 1.5 feet; moist to wet. Non-plastic.

3.5 feet becomes stiff, gravelly, with some sand; moist.

Medium dense, brown, silty GRAVEL with some fine to coarse sand.

Dense to very dense, subrounded GRAVELS, COBBLES, and possible BOULDERS.

Drilling refusal on cobbles or boulders at 6.5 feet.

NOTES: Boring completed on March 26, 2025. Backfilled with bentonite chips or drill spoils with sod cut and placed back.

Samples and Data

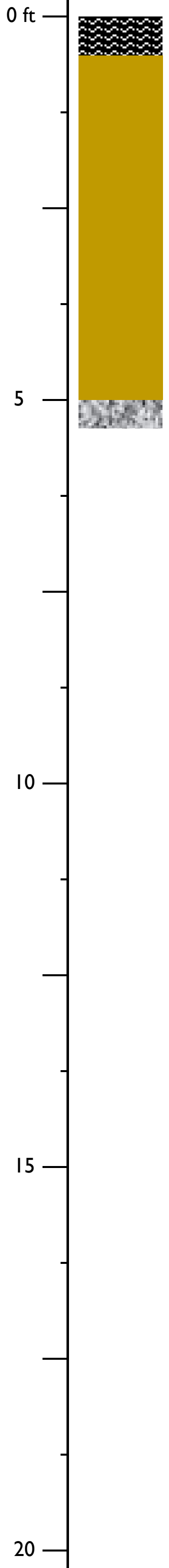
3	w = 23%
15	w = 17%
8	w = 18%
50/ 5”	

N_{60}= SPT blowcount
w = moisture content
f = percent fines
γ_d = dry unit weight

Geotech Solutions Inc.

BORING B-1

hopevillage-25-1-gi



Soil and Rock Description

6” soft rooty SILT TOPSOIL/SOD

Medium stiff, brown SILT with trace fine to coarse sand and occasional fine roots down to 1.5 feet; moist to wet. Non-plastic.

3.5 feet becomes with some sand and occasional gravel, moist.

Dense to very dense, subrounded GRAVELS, COBBLES, and possible BOULDERS.

Drilling refusal on cobbles or boulders at 5.3 feet.

NOTES: Boring completed on March 26, 2025. Backfilled with bentonite chips or drill spoils with sod cut and placed back.

Samples and Data

12	w = 17%
5	w = 25%
50/ 3”	

N_{60}= SPT blowcount
w = moisture content
f = percent fines
γ_d = dry unit weight

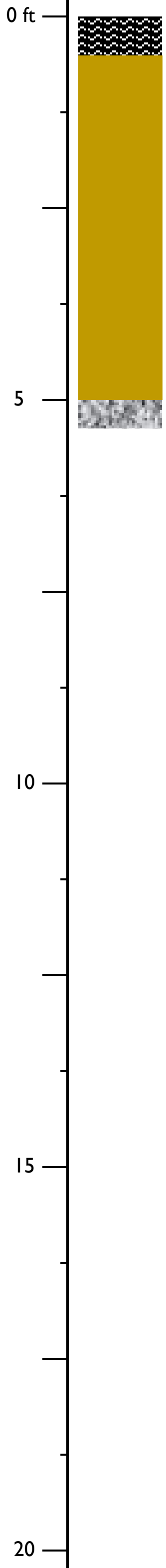
Geotech Solutions Inc.

BORING B-2

hopevillage-25-1-gi

Soil and Rock Description

Samples and Data



6” soft rooty SILT TOPSOIL/SOD

Soft to medium stiff, brown SILT with trace fine to coarse sand and occasional fine roots down to 1.5 feet; moist to wet. Non-plastic.

4 w = 28%

Becomes moist.

3 w = 22%

Dense to very dense, subrounded GRAVELS, COBBLES, and possible BOULDERS.

50/ 1”

Drilling refusal on cobbles or boulders at 5.1 feet.

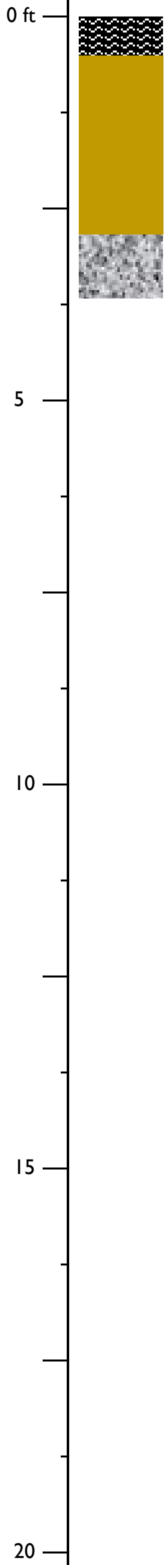
NOTES: Boring completed on March 26, 2025. Backfilled with bentonite chips or drill spoils with sod cut and placed back.

N_{60}= SPT blowcount
w = moisture content
f = percent fines
γ_d = dry unit weight

Geotech Solutions Inc.

BORING B-3

hopevillage-25-1-gi



Soil and Rock Description

6” soft rooty SILT TOPSOIL/SOD

Medium stiff, brown SILT with trace fine to coarse sand and occasional fine roots down to 1.5 feet; moist to wet. Non-plastic.

Dense to very dense, subrounded GRAVELS, COBBLES, and possible BOULDERS.

Drilling refusal on cobbles or boulders at 3.6 feet.

NOTES: Boring completed on March 26, 2025. Backfilled with bentonite chips or drill spoils with sod cut and placed back.

Samples and Data

6

w = 24%

7-15-50/1”

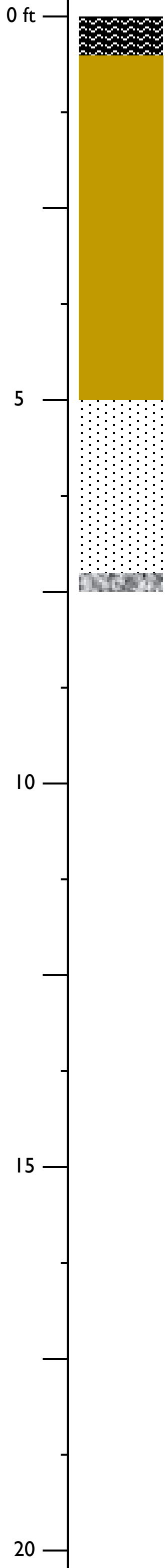
N_{60}= SPT blowcount
w = moisture content
f = percent fines
γ_d = dry unit weight

Geotech Solutions Inc.

BORING B-4

hopevillage-25-1-gi

Soil and Rock Description



5” soft rooty SILT TOPSOIL/SOD

Medium stiff, brown SILT with trace fine to coarse sand and occasional fine roots down to 1.5 feet; moist to wet. Non-plastic.

3.5 feet becomes with some sand and occasional gravel; moist.

Loose, brown, fine to medium SAND with some silt; moist.

becomes gravelly.

Dense to very dense, subrounded GRAVELS, COBBLES, and possible BOULDERS.

Drilling refusal on cobbles or boulders at 7.5 feet.

NOTES: Boring completed on March 26, 2025. Backfilled with bentonite chips or drill spoils with sod cut and placed back.

Samples and Data

4 w = 21%

6 w = 26%

7 w = 16%

N_{60}= SPT blowcount
w = moisture content
f = percent fines
γ_d = dry unit weight

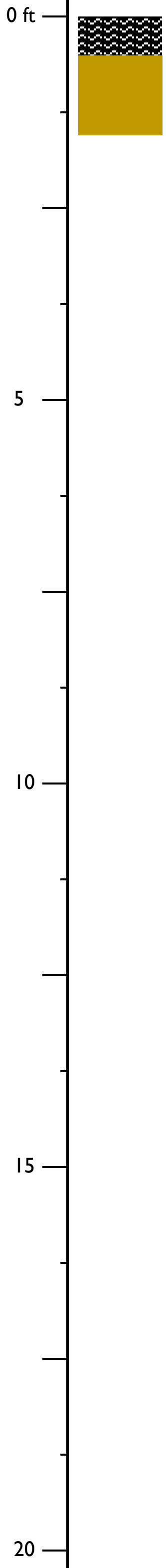
Geotech Solutions Inc.

BORING B-5

hopevillage-25-1-gi

Soil and Rock Description

Samples and Data



7" soft rooty SILT TOPSOIL/SOD

Soft to medium stiff, brown SILT with trace fine to coarse sand and occasional fine roots; moist to wet. Non-plastic.

w = 18%

No access with drill rig due to very soft surface conditions. Hand dug.

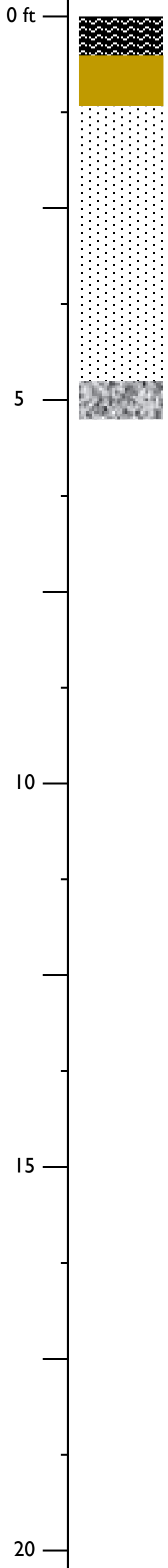
NOTES: completed by hand on March 26, 2025. Backfilled with drill spoils with sod cut and placed back.

N_{60}= SPT blowcount
w = moisture content
f = percent fines
γ_d = dry unit weight

Geotech Solutions Inc.

BORING B-6

hopevillage-25-1-gi



Soil and Rock Description

5” soft rooty SILT TOPSOIL/SOD

Medium stiff, brown SILT with trace fine to coarse sand and occasional fine roots ; moist to wet. Non-plastic.

Medium dense to dense, brown, fine to medium cemented SAND with some gravel and silt; moist.

Dense to very dense, subrounded GRAVELS, COBBLES, and possible BOULDERS.

Drilling refusal on cobbles or boulders at 5.5 feet.

NOTES: Boring completed on March 26, 2025. Backfilled with bentonite chips or drill spoils with sod cut and placed back.

Samples and Data

29	w = 18%
12	w = 22%
50/ 6”	

N_{60}= SPT blowcount
w = moisture content
f = percent fines
γ_d = dry unit weight

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Solutions Inc.

BORING B-7

hopevillage-25-1-gi

Exploration	Depth, ft	Moisture Content
B-1	1.0	23%
B-1	3.0	17%
B-1	5.0	18%
B-2	1.0	17%
B-2	2.5	25%
B-2	5.0	18%
B-3	1.0	28%
B-3	2.5	22%
B-4	1.0	24%
B-5	1.0	21%
B-5	2.5	26%
B-5	5.0	16%
B-6	1.0	18%
B-7	1.0	18%
B-7	2.5	22%

Geotech Solutions Inc.

MOISTURE CONTENTS
hopevillage-25-1-gi

Appendix C: Other Supporting Information

Soil Map—Clackamas County Area, Oregon
(Hope Village Campus)

Map Scale: 1:3,070 if printed on A portrait (8.5" x 11") sheet.

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

MAP LEGEND

Area of Interest (AOI)

☐ Area of Interest (AOI)

Soils

☐ Soil Map Unit Polygons

〰 Soil Map Unit Lines

🟧 Soil Map Unit Points

Special Point Features

◉ Blowout

⊠ Borrow Pit

✱ Clay Spot

◇ Closed Depression

✕ Gravel Pit

∴ Gravelly Spot

● Landfill

Λ Lava Flow

ψ Marsh or swamp

⚒ Mine or Quarry

◎ Miscellaneous Water

○ Perennial Water

∨ Rock Outcrop

+ Saline Spot

⁘ Sandy Spot

⊖ Severely Eroded Spot

◈ Sinkhole

≽ Slide or Slip

∅ Sodic Spot

≡ Spoil Area

◍ Stony Spot

∞ Very Stony Spot

ψ Wet Spot

△ Other

‐ ‐ Special Line Features

Water Features

〜 Streams and Canals

Transportation

┼┼┼ Rails

〰 Interstate Highways

〰 US Routes

〰 Major Roads

〰 Local Roads

Background

🟩 Aerial Photography

MAP INFORMATION

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

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

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

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

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

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

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

Soil Survey Area: Clackamas County Area, Oregon
Survey Area Data: Version 21, Aug 30, 2024

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

Date(s) aerial images were photographed: Sep 26, 2022—Oct 11, 2022

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

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
53A	Latourell loam, 0 to 3 percent slopes	49.2	100.0%
Totals for Area of Interest		**49.2**	**100.0%**

Clackamas County Area, Oregon

53A—Latourell loam, 0 to 3 percent slopes

Map Unit Setting

National map unit symbol: 225j
Elevation: 50 to 400 feet
Mean annual precipitation: 40 to 60 inches
Mean annual air temperature: 52 to 54 degrees F
Frost-free period: 165 to 210 days
Farmland classification: All areas are prime farmland

Map Unit Composition

Latourell and similar soils: 90 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Latourell

Setting

Landform: Terraces
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Stratified glaciolacustrine deposits

Typical profile

H1 - 0 to 15 inches: loam
H2 - 15 to 48 inches: loam
H3 - 48 to 60 inches: gravelly sandy loam

Properties and qualities

Slope: 0 to 3 percent
Depth to restrictive feature: More than 80 inches
Drainage class: Well drained
Capacity of the most limiting layer to transmit water (Ksat): Moderately high to high (0.57 to 1.98 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Available water supply, 0 to 60 inches: High (about 9.5 inches)

Interpretive groups

Land capability classification (irrigated): 1
Land capability classification (nonirrigated): 1
Hydrologic Soil Group: B

Hydric soil rating: No

Data Source Information

Soil Survey Area: Clackamas County Area, Oregon
Survey Area Data: Version 16, Jun 11, 2020

TABLE 3.5.2B SCS WESTERN WASHINGTON RUNOFF CURVE NUMBERS

SCS WESTERN WASHINGTON RUNOFF CURVE NUMBERS (Published by SCS in 1982)

Runoff curve numbers for selected agricultural, suburban and urban land use for Type 1A rainfall distribution, 24-hour storm duration.

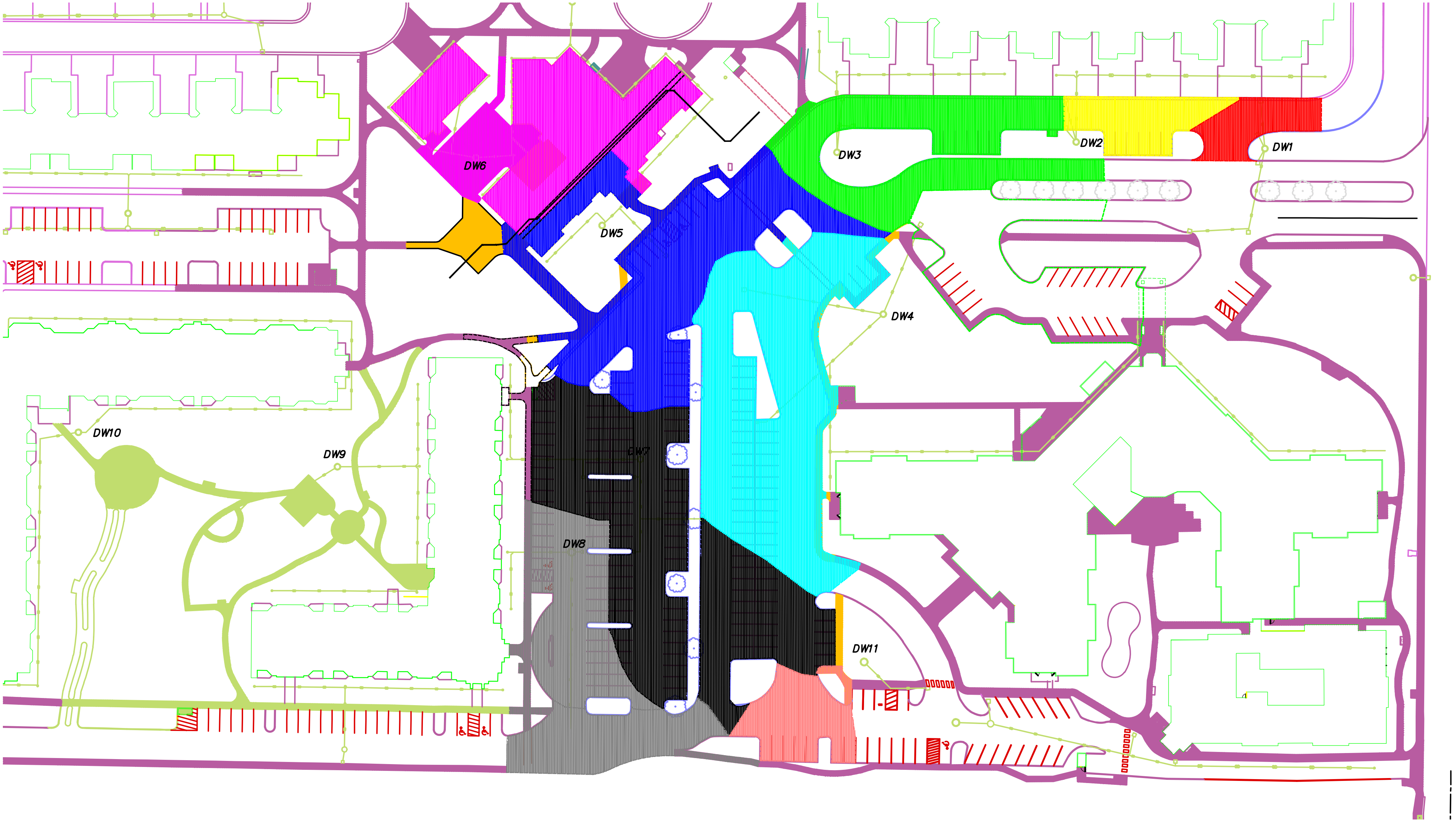
LAND USE DESCRIPTION		CURVE NUMBERS BY HYDROLOGIC SOIL GROUP			
		A	B	C	D
Cultivated land(1):	winter condition	86	91	94	95
Mountain open areas:	low growing brush and grasslands	74	82	89	92
Meadow or pasture:		65	78	85	89
Wood or forest land:	undisturbed or older second growth	42	64	76	81
Wood or forest land:	young second growth or brush	55	72	81	86
Orchard:	with cover crop	81	88	92	94
Open spaces, lawns, parks, golf courses, cemeteries, landscaping.					
good condition:	grass cover on 75% or more of the area	68	80	86	90
fair condition:	grass cover on 50% to 75% of the area	77	85	90	92
Gravel roads and parking lots		76	85	89	91
Dirt roads and parking lots		72	82	87	89
Impervious surfaces, pavement, roofs, etc.		98	98	98	98
Open water bodies:	lakes, wetlands, ponds, etc.	100	100	100	100
Single Family Residential (2)		Separate curve number shall be selected for pervious and impervious portion of the site or basin			
Dwelling Unit/Gross Acre	% Impervious (3)				
1.0 DU/GA	15				
1.5 DU/GA	20				
2.0 DU/GA	25				
2.5 DU/GA	30				
3.0 DU/GA	34				
3.5 DU/GA	38				
4.0 DU/GA	42				
4.5 DU/GA	46				
5.0 DU/GA	48				
5.5 DU/GA	50				
6.0 DU/GA	52				
6.5 DU/GA	54				
7.0 DU/GA	56				
Planned unit developments, condominiums, apartments, commercial business and industrial areas.	% impervious must be computed				

(1) For a more detailed description of agricultural land use curve numbers refer to National Engineering Handbook, Section 4, Hydrology, Chapter 9, August 1972.

(2) Assumes roof and driveway runoff is directed into street/storm system.

(3) The remaining pervious areas (lawn) are considered to be in good condition for these curve numbers.

EXISTING BASINS IN "DEVELOPMENT AREA"



REVISED BASINS IN "DEVELOPMENT AREA"