Received
Planning Division
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Wood Environment & Infrastructure Solutions, Inc.
15862 SW 72nd Ave., Suite 150
Portland, Oregon
USA 97224
T: 503-639-3400
www.woodplc.com

10/19/2021
Project No. 161M128582.01

Mr. Tripp Dietrich
Oregon Worsted Company
PO Box 82098
Portland, OR 97282

Subject: Groundwater Removal from Proposed Flood Storage Basin
Allen Boulevard Commercial Property, Beaverton, Oregon

Dear Mr. Dietrich,

Wood Environment & Infrastructure Solutions, Inc. (Wood) has prepared this report presenting an estimated groundwater infiltration rate for a proposed flood storage basin so that the size of two pumps needed to remove the groundwater under the proposed design condition at maximum expected groundwater elevation can be determined. The estimated infiltration rate was prepared using information provided by Oregon Worsted Company (including plans and reports provided by DOWL) and from other references cited below.

Background

The flood storage basin, approximately 320 by 400 feet, will be located in the southeastern part of the site, north of Fanno Creek. Fanno Creek is tributary to the Tualatin River, which drains the Tualatin Valley. The area of the basin floor will be 131,885 square feet and the basin's floor will be approximately 10 feet below the planned ground surface.

The basin's purpose is to provide temporary storage of flood water from Fanno Creek to replace flood plain storage lost because of fill needed to develop the site. Flood water will be diverted into the basin via an upper pipe whose diameter is 48 inches and whose invert elevation will be 182.5 feet (all elevations are referenced to NGVD 29).

The two pumps will be a primary pump and a secondary pump. The primary pump is expected to be the smaller one needed to control the groundwater level in the basin whenever gravity flow through the flap-gated lower pipe is not available. The larger, secondary pump will primarily be used to assist in the draining of 23,000 cubic yards of stored flood water in the basin below the 182.5 ft elevation (i.e., invert of the upper pipe) and when the creek is within the operational range noted above (i.e. 182.5 ft to ~178 ft). Both pumps could be run together providing approximately 400 gallons per minute (gpm) of pumping capacity under extreme conditions of very high groundwater.

Stored flood water will drain back to Fanno Creek via both the upper pipe and a lower flap-gated pipe whose diameter is 18 inches and whose invert elevation at the creek will be 177.56 feet. The invert elevation of the lower pipe excavation will be 178 ft. Because the water level in the creek is usually lower than 177.56 feet, the basin will be self-draining through the lower pipe much of the time and groundwater pumping will only occur when the water level in the pond is above 178 feet (i.e., standing water) and when the level of the creek is above 177.56 feet but below 182.5 feet.

According to an email from Mike Towle of DOWL, there will not be any runoff from directly connected impervious areas entering the basin. There is a 0.15-acre area of impervious pathway located east of the pond whose runoff will drain laterally to pervious areas. No runoff into the basin from pervious areas of the site is included in the pumping rate estimates.

Methodology

The groundwater inflow rate to the planned excavation was estimated using the analytical method of Marinelli and Niccoli (2000). This method pertains to unconfined aquifers and considers an excavation as a circular depression into which groundwater flows horizontally within a radius of influence (r_o) from an upper zone (zone 1) that extends from the base of the excavation basin to the water table at the position of r_o, and from a lower zone (zone 2) where groundwater flows upward through the excavation basin floor.

The method requires values for the following input parameters: r_p (excavation radius), h_o and h_p (height above the excavation floor of the water table at the location of r_o and r_p, respectively), d (depth of water in the excavation), K_{h1} (horizontal hydraulic conductivity of zone 1), K_{h2} (horizontal hydraulic conductivity of zone 2), K_{v2} (vertical hydraulic conductivity of zone 2), m (square root of K_{h2} over K_{v2}), W (groundwater recharge rate), and r_o. These parameters are based on measured or assumed values, except r_o, which must be iteratively calculated to find the r_o value that matches h_o.

To allow for uncertainty, a range of values from moderate- to worst-case were used to estimate inflow. A high inflow rate is recommended for sizing of one or more pumps because the basin needs to be maintained free of water except after overflows occur from Fanno Creek. The moderate-case inputs are summarized on page 1 of Attachment 2. The worst-case inputs are summarized on page 1 of Attachment 3. The basis for the input values is as follows:

- Basin radius is the radius of the assumed circular excavation whose floor has the same area as the planned basin.
- The water table elevation at r_o is assumed to be 187 feet, essentially the ground surface elevation and thus the maximum possible value.
- The water level in the excavation is 178 feet, the control elevation of the outlet pipe specified in information provided by DOWL.
- The depth of water in the basin is 0.3 feet, the difference between the excavation water level of 178 feet and the excavation floor elevation of 177.7 feet, as specified in information provided by DOWL.
- The horizontal hydraulic conductivity ("permeability") ties to soil type. The saturated soil immediately underlying the site is the Willamette Silt (WS), which consists of a stack of horizontal layers deposited by cataclysmic Ice Age floods (Missoula floods) from an ice-dammed lake in northwestern Montana that flooded the Columbia River and back-flooded the Tualatin Valley (see O'Connor and others, 2001).

  In Wood's experience, the WS is typically several tens of feet thick in the Tualatin Valley. Wood's review of boring logs on file at the Oregon Water Resources Department and in a geotechnical engineering report prepared for the site (GeoEngineers, 2020) indicates the base of the deposit is 30 or more feet deep beneath the site. The sediment is chiefly clayey silt to silt, but layers in the middle part of the deposit are in places sandy.

  Estimates of WS bulk horizontal hydraulic conductivity (for the entire thickness of the deposit) are available from work presented in publicly available reports completed for groundwater cleanup

projects supervised by the Oregon Department of Environmental Quality that involve the WS at an Intel facility 5 miles west of the site and at the former GAF/Mattel site 1.5 miles south of the site, to the west of Highway 217 and the Washington Square shopping center. The WS horizontal hydraulic conductivity was estimated at 1.1 feet per day (ft/day) at the Intel site (Amec Earth & Environmental, 2006) and 2.5 ft/day at the GAF/Mattel site (Landau, 2001). These estimates are consistent with an average value of 1.2 ft/day at a location near Salem, Oregon (Iverson, 2002).

Based on this information, the larger horizontal hydraulic conductivity value of 2.5 ft/day was used to develop the moderate-case groundwater inflow estimate. The worst-case estimate used a value of 9 ft/day because values of up to approximately 9 ft/day have been measured (EMCON, 1995; Iverson, 2002). For both moderate- and worst-case estimates, the same value was used for Kh_1 and K_{h2} because the excavation is shallow and does not extend to depths where the WS next to and immediately beneath the excavation becomes sandier. Site boring logs provided in a geotechnical engineering report (GeoEngineers, 2020) indicate "sandiness" beginning at depths 15 to 20 feet beneath the existing ground surface.

- Vertical hydraulic conductivity is typically orders of magnitude lower than horizontal hydraulic conductivity due to fine-grained layers that impede vertical flow; the DEQ-overseen modeling cited above used K_h:K_v ratios for the WS of 100:1 (Intel site) or 1,000:1 (GAF/Mattel site). For the moderate-case estimate, K_{v2} was assumed to be 1/100th of K_{h2}, or 0.025 ft/day because that would result in more upward groundwater flow into the excavation than a smaller K_{v2} value. The worst-case estimate assumed that the horizontal and vertical hydraulic conductivities were the same.
- Groundwater recharge is estimated at 10 inches per year (in/yr) for the moderate case and 15 in/yr for the worst case. These estimates are based on consideration of regional work by the U.S. Geological Survey (USGS) and the consultant reports described above. According to the USGS (Woodward and others, 1998), groundwater recharge based on a water-balance model for the WS is estimated as 3.6 inches per year (in/yr) for built-up areas and 14.5 in/yr for urban areas. Another USGS report (Conlon and others, 2005) used precipitation-runoff modeling to estimate groundwater recharge for the site vicinity as between 7 to 15 in/yr.

  Groundwater recharge was also estimated by means of numerical groundwater flow model calibration conducted for the DEQ-supervised groundwater cleanup projects cited above. Groundwater recharge was estimated at 10 in/yr at the Intel facility (Amec Earth & Environmental, 2006). The regional model developed for the GAF/Mattel site estimated groundwater recharge at 7.2 in/yr for the "developed" area encompassing this project site's location (Landau, 2001).

Results and Recommendation

The total estimated groundwater inflow for the moderate case is 93 gpm, or approximately 100 gpm, as detailed in the calculations provided in Attachment 2. Because this estimate is based on a high water table at an assumed elevation of 187 feet, the actual inflow will be less most of the time. If the horizontal hydraulic conductivity is larger – values of up to approximately 9 ft/day have been measured (EMCON, 1995; Iverson, 2002) - the total estimated inflow for the worst case would increase to 330 gpm (see Attachment 3). Taken together, the information presented above suggests that the groundwater inflow rate could range from approximately 100 to 330 gpm. Allowing for uncertainties, and rounding-up the worst-case estimate to 350 gpm, Wood recommends a design pumping rate of 400 gpm and adding additional pump capacify if indicated by future performance data.

Finally, the analysis considers the basin as hydraulically separate from the creek (except for inflow and outflow controls described earlier). On this basis, Wood recommends that the trenches excavated for the

inflow and outflow pipes be constructed to prevent water from flowing through their backfill and "short circuiting" the hydraulic connection between the pond and Fanno Creek.

Limitations

This analysis is based on information provided by DOWL on the proposed site configurations, and by available estimates of hydraulic conductivity from other sites in the region.

Groundwater inflow to the flood basin was estimated using a simple analytical method instead of an empirical water balance or a numerical groundwater flow model. Analytical models are based on idealized assumptions about aquifer geometry and, especially, the location of the radius of influence within which groundwater flows to the excavation. A more rigorous method might refine the estimate and support a lower design pumping capacity.

The analysis considered the excavation as a circle that extends into laterally continuous soil, with the location of h_o at a constant radial distance r_o from the center of the circle. This assumption is similar to site conditions except for the area south of the excavation, where Fanno Creek lies only 50 to 100 feet from the basin. The amount of lateral groundwater inflow from soil between the creek and the pond is expected to be minimal because of the limited recharge area. However, deeper groundwater that would ordinarily discharge to the creek might flow upward into the basin and increase the vertical inflow rate.

This report was prepared exclusively for Oregon Worsted Company by Wood Environment & Infrastructure Solutions, Inc. The quality of information, conclusions, and estimates contained herein is consistent with the level of effort involved in Wood services and based on: i) information available at the time of preparation, ii) data supplied by outside sources, and iii) the assumptions, conditions, and qualifications set forth in this report. This Groundwater Removal from Proposed Allen Blvd. Flood Storage Basin Report is intended to be used by Oregon Worsted Company for the Allen Boulevard Commercial Property, Beaverton, Oregon only, subject to the terms and conditions of its contract with Wood. Any other use of, or reliance on, this report by any third party is at that party's sole risk.

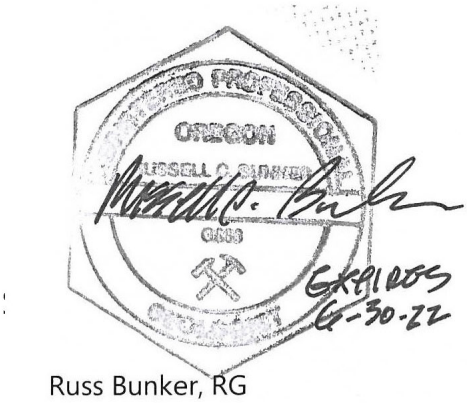
The findings contained herein are relevant to the dates of the Wood Site visit and should not be relied upon to represent conditions at later dates. In the event that changes in the nature, usage, or layout of the property or nearby properties are made, the conclusions and recommendations contained in this report may not be valid. If additional information becomes available, it should be provided to Wood so the original conclusions and recommendations can be modified as necessary.

If you have questions about this report, please contact Seth Jelen, (seth.jelen@woodplc.com), or Dan Schall (daniel.schall@woodplc.com).

Sincerely,

Wood Environment & Infrastructure Solutions, Inc.

Reviewed by:



Russ Bunker, RG
Senior Associate Geologist

Seth Jelen, PE
Principal Engineer

Attachment 1: References
Attachment 2: Moderate-Case Calculation Parameters and Results
Attachment 3: Worst-Case Calculation Parameters and Results

Attachment 1:
References

Attachment 1 - References

Amec Earth & Environmental, Inc. 2006. Remedial Action Work Plan, Intel Corporation, Aloha Campus, 3585 SW 198th Avenue, Aloha, Oregon. Report submitted to Oregon Department of Environmental Quality for Intel Corporation. July.

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Iverson, J. 2002. Investigation of the Hydraulic, Physical, and Chemical Buffering Capacity of Missoula Flood Deposits for Water Quality and Supply in the Willamette Valley of Oregon. Oregon State University, Corvallis, Oregon, M.S. thesis.

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Marinelli, F. and Niccoli, W.L. 2000. Simple Analytical Equations for Estimating Ground Inflow to a Mine Pit. Ground Water, volume 38, no. 2, pages 311-314.

O'Connor, J.E., Sarna-Wojcicki, A., Wozniak, K.C., Polette, D.J., and Fleck, R.J. 2001. Origin, Extent, and Thickness of Quaternary Geologic Units in the Willamette Valley, Oregon. U.S. Geological Survey Professional Paper 1620.

Woodward, D.G., Gannett, M.W., and Vaccaro, J.J. 1998. Hydrogeologic Framework of the Willamette Lowland Aquifer System, Oregon and Washington. U.S. Geological Survey Professional Paper 1424-B.

Attachment 2:
Moderate-Case Calculation Parameters and Results

Page 1 - Inputs - Moderate Condition

Area of pond base	131,885	ft^2
Equivalent radius, r_p	205	ft
	62	m
Water table elevation	187	ft
Pond bottom elevation	177.7	ft
h_o	9.3	ft
	2.8	m
Pond surface elevation	178.0	ft
Pond water depth, d	0.3	ft
h_p	0.3	ft
	0.09	m
$Kh_1 = Kh_2$	2.5	ft/day
	8.8E-06	m/sec
K_{h2}:K_{v2}	100:1	
K_{v2}	8.8E-08	m/sec
m	10	
Recharge, W	10	in/year
	8.1E-09	m/sec
W/K_{h1}	9.1E-04	
r_o	**144**	**m**

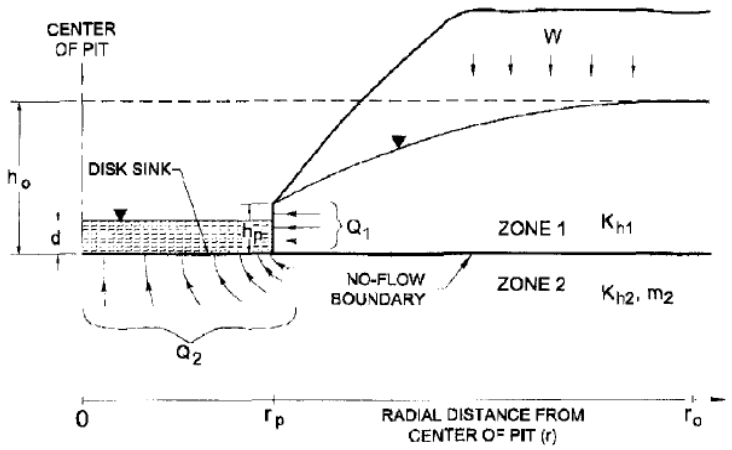

Figure 2. Pit inflow analytical model.

r_o	match solution	<< check --- zero = hp^2 - Ho^2 + (W/Kh) (ro^2 Ln(ro/rp) - (ro^2 - rp^2) / 2)
10	-6.461247362	
20	-6.847030005	
30	-7.262243939	
40	-7.630547231	
50	-7.898628324	
60	-8.025380717	
70	-7.977315069	
80	-7.726162738	
90	-7.247461097	
100	-6.519647066	
110	-5.523440853	
120	-4.241407693	
130	-2.657634944	
140	-0.757487257	
141	-0.54954171	
142	-0.338283162	
143	-0.123698705	
144	0.094224477	<< check --- zero = hp^2 - Ho^2 + (W/Kh) (ro^2 Ln(ro/rp) - (ro^2 - rp^2) / 2)
145	0.315499114	
146	0.540137846	
147	0.768153225	
148	0.99955772	
149	1.234363714	
150	1.472583505	

Page 2 - Inflow Calculations for Walls (Q1) and Bottom (Q2) - Moderate Case

$Q_1 = W*3.14*(r_o^2-r_p^2)$			$Q_2 = 4r_p(K_{h2}/m)(h_o-d)$		
W	8.10E-09	m/sec	K_{h2}	8.80E-06	m/sec
r_o	144	m	m	10	
r_o^2	20736	m^2	h_o	2.8	m
r_p	62	m	d	0.3	m
r_p^2	3844	m^2			
Q_1	4.30E-04	m^3/sec	Q_2	5.46E-03	m^3/sec
	7	gpm		86	gpm
$Q_t = Q_1+Q_2$	93	gpm			

Attachment 3:
Worst-Case Calculation Parameters and Results

Page 1 - Inputs for worst-case (conservative limit) values

Area of pond base	131,885	ft^2
Equivalent radius, r_p	205	ft
	62	m
Water table elevation	187	ft
Pond bottom elevation	177.7	ft
h_o	9.3	ft
	2.8	m
Pond surface elevation	178.0	ft
Pond water depth, d	0.3	ft
h_p	0.3	ft
	0.09	m
$Kh_1 = Kh_2$	9	ft/day
	3.2E-05	m/sec
K_{h2}:K_{v2}	1:1	
K_{v2}	3.2E-05	m/sec
m	1	
Recharge, W	15	in/year
	1.2E-08	m/sec
W/K_{h1}	3.8E-04	
r_o	**183**	**m**

r_o	match solution	<< check --- zero == hp^2 - Ho^2 + (W/Kh) (ro^2 Ln(ro/rp) - (ro^2 - rp^2) / 2)
10	-7.376897882	
20	-7.53764065	
40	-7.864106161	
60	-8.028620113	
80	-7.903945956	
100	-7.401231092	
120	-6.451964687	
140	-5.000331172	
160	-2.999216144	
179	-0.552007073	
180	-0.407848558	
181	-0.262123782	
182	-0.114828529	
183	3.4E-02	<< check --- zero == hp^2 - Ho^2 + (W/Kh) (ro^2 Ln(ro/rp) - (ro^2 - rp^2) / 2)
184	0.184490156	
200	2.809710052	
220	6.685592673	
240	11.24885725	
260	16.52602485	
280	22.54148771	
300	29.31782608	

Page 2 - Inflow Calculations for Walls (Q1) and Bottom (Q2) - Worst-Case (Conservative) Values

$Q_1 = W*3.14*(r_o^2-r_p^2)$			$Q_2 = 4r_p(K_{h2}/m)(h_o-d)$		
W	1.20E-08	m/sec	K_{h2}	3.20E-05	m/sec
r_o	183	m	m	1	
r_o^2	33489	m^2	h_o	2.8	m
r_p	62	m	d	0.3	m
r_p^2	3844	m^2			
Q_1	0.001117024	m^3/sec	Q_2	1.98E-02	m^3/sec
	18	gpm		314	gpm
$Q_t = Q_1+Q_2$	332	gpm			