MOMENTUM CIVIL
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DATE: 3/17/2022

TO: KVK Quality Construction, LLC
21411 81ST Ct. East
Bonney Lake, WA 98391

FROM: Drew Harris, P.E.

SUBJECT: Preliminary Storm Design Calculations & Civil Engineering Narrative

PROJECT: 304/312 2nd Avenue NE, Puyallup Multi-family

MC Job No: KVKQ-0001

CC: Graves + Associates

After review of the architectural site plan, project survey and geotechnical report, Momentum Civil has prepared preliminary stormwater calculations for the subject development. The proposed development is approximately 1/3 acre in total area and contains approximately 850 square feet of onsite sidewalks and 3,500 square feet of roof surfaces. Approximately 8,200 square feet of the site is proposed as parking (see attached site plan).

Existing Conditions

In the existing condition, the site is flat, with only one or two feet of grade toward 2nd Avenue NE. The site is currently vacant and vegetated with grass and a few trees. The infiltration analysis by Migizi Group determined that onsite infiltration is feasible with a long-term infiltration rate of 4.0 inches per hour.

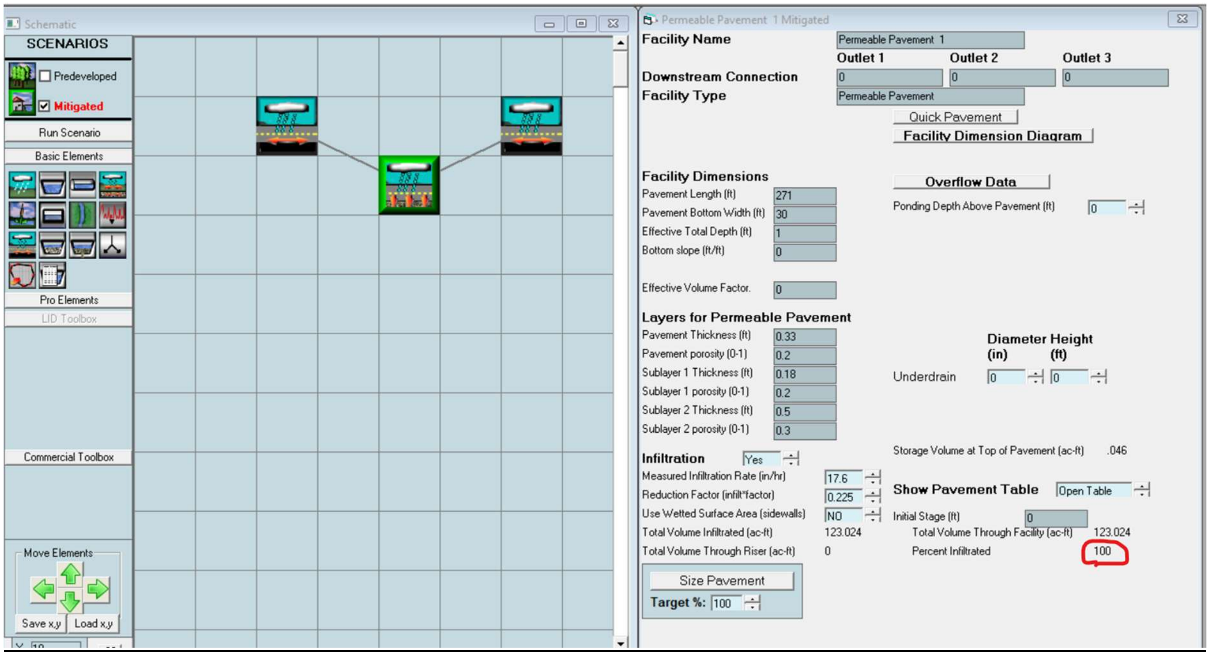
Proposed Storm Design

The impervious areas proposed with this project exceed 10,000 square feet, and the pollution generating impervious surfaces (PGIS) exceeds 5,000 square feet—these two thresholds together indicate the project must provide both stormwater quality and quantity control measures (Minimum Requirements 6 and 7 of the DOE Manual).

Based on the soil profile provided, we recommend that stormwater be managed on for this development with permeable pavement, with downspouts for both building roofs and onsite sidewalks routed to the permeable pavement ballast course.

Attached please find a WWHM report showing the calculations sizing the ballast course to provide full infiltration of the onsite impervious surfaces.

Below is a screenshot of the calculation performed, routing both onsite sidewalk and roof surfaces to the permeable pavement section. Additional detail is provided in Appendix A.



<u>Water and Sewer</u>

Both water and sewer service is proposed to connect to 2nd Avenue Northeast.

Thank you for attention on these matters and we look forward to your reply.

Attachments/Enclosures:

Attachment A – Stormwater Calculations for Permeable Pavement WWHM Report

Attachment B – Architectural Site Plan

Attachment C – Geotechnical Report by Migizi Group dated January 26, 2022

Attachment D – Infiltration Memorandum by Migizi Group dated March 11, 2022

Attachment A

Stormwater Calculations for Permeable Pavement WWHM Report

WWHM2012

PROJECT REPORT

Project Name: KVKQ-Infiltration Design
Site Name:
Site Address:
City :
Report Date: 3/17/2022
Gage : 42 IN EAST
Data Start : 10/01/1901
Data End : 09/30/2059
Precip Scale: 1.00
Version Date: 2019/09/13
Version : 4.2.17

PREDEVELOPED LAND USE

Name : Basin 1
Bypass: No

GroundWater: No

Pervious Land Use	acre
A B, Lawn, Flat	.36
Pervious Total	0.36

Impervious Land Use	acre
Impervious Total	0
Basin Total	0.36

Element Flows To:

Surface	Interflow	Groundwater

MITIGATED LAND USE

Name : Sidewalk Surfaces
Bypass: No

Impervious Land Use	acre
SIDEWALKS FLAT LAT	0.02

Element Flows To:

Outlet 1 **Outlet 2**
Permeable Pavement 1

Name : Building Roofs
Bypass: No

Impervious Land Use	**acre**
ROOF TOPS FLAT LAT	**0.08**

Element Flows To:
Outlet 1 **Outlet 2**
Permeable Pavement 1

Name : Permeable Pavement 1
Pavement Area: 0.1866 ft.
Pavement Length: 271.00 ft.
Pavement Width: 30.00 ft.
Pavement slope 1: 0 To 1
Pavement thickness: 0.33
Pour Space of Pavement: 0.2
Material thickness of second layer: 0.18
Pour Space of material for second layer: 0.2
Material thickness of third layer: 0.5
Pour Space of material for third layer: 0.3
Infiltration On
Infiltration rate: 17.6
Infiltration safety factor: 0.225
Total Volume Infiltrated (ac-ft.): 123.024
Total Volume Through Riser (ac-ft.): 0
Total Volume Through Facility (ac-ft.): 123.024
Percent Infiltrated: 100 ← 100% INFILTRATED
Total Precip Applied to Facility: 0
Total Evap From Facility: 5.898

Revise to acres [Preliminary Stormwater Calculations, Page 5/40]

Specify units. Report says 4.0 inches per hour [Preliminary Stormwater Calculations, Page 5/40]

Modeling should include the area of impervious surface entering the facility and then demonstrate the additional capacity to infiltrate the roofs and sidewalks. [Preliminary Stormwater Calculations, Page 5/40]

If roofs are to be infiltrated within pervious pavement gallery, then the gallery is a defacto infiltration facility and will be subject to infiltration gallery requirements as well as pervious pavement standards. [Preliminary Stormwater Calculations, Page 5/40]

Attachment B

Architectural Site Plan

Attachment C

Geotechnical Report by Migizi Group dated January 26, 2022

Geotechnical Engineering Report

Proposed Urban Residential Development
312 2nd Ave NE
Puyallup, WA 98372
Parcel Nos. 7940100102, 7940100103

January 26, 2022

prepared for:

KVK Quality Construction, LLC
Attention: Kon Kurkov
7427 S D Street
Tacoma, WA 98408

prepared by:

Migizi Group, Inc.
PO Box 44840
Tacoma, WA 98448
(253) 537-9400

MGI Project P2388-T21

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APPENDIX A

MIGIZI GROUP, INC.

PO Box 44840
Tacoma, Washington 98448

PHONE (253) 537-9400
FAX (253) 537-9401

January 26, 2022

KVK Quality Construction, LLC
7427 S D Street
Tacoma, WA 98408

Attention: Kon Kurkov

Subject: **Geotechnical Engineering Report**
Proposed Urban Residential Development
312 2nd Ave NE
Puyallup, WA 98372
Parcel Nos. 7940100102, 7940100103

MGI Project P2388-T21

Dear Mr. Kurkov:

Migizi Group, Inc. (MGI) is pleased to submit this report describing the results of our geotechnical engineering evaluation of the proposed development at the corner of 2nd Ave NE and 4th St NE in Puyallup, Washington. We previously prepared a *Groundwater Monitoring Results* letter dated May 19, 2021.

This report has been prepared for the exclusive use of KVK Quality Construction, LLC, and their consultants, for specific application to this project, in accordance with generally accepted geotechnical engineering practice.

1.0 SITE AND PROJECT DESCRIPTION

The project site consists of two undeveloped residential parcels located at the intersection of 2nd Ave NE and 4th St NE, just east of E Main St, in Puyallup, Washington, as shown on the enclosed Topographic and Location Map (Figure 1). The proposed development consists of two contiguous tax parcels, which encompass a total area of approximately 0.36 acres, with each individual parcel measured at 0.16 to 0.20 acres. At the time of our site visit, the parcel remains lightly vegetated with native grasses.

Improvement plans involve the construction of two multi-family apartment buildings along the north end of the project area, which will be constructed at or near existing grade. Onsite paved parking stalls, again at or near existing grade, will be located across the south end of the project area, with site access off 2nd Ave NE.

2.0 EXPLORATORY METHODS

We explored surface and subsurface conditions at the project site on March 26, 2021. Our exploration and evaluation program comprised the following elements:

- Surface reconnaissance of the site,
- One groundwater monitoring well (designated MW-1), and three auger borings (designated as B-1 to B-3) advanced on March 26, 2021, and
- A review of published geologic and seismologic maps and literature.

Table 1 (below) summarizes the approximate functional locations and termination depths of our subsurface explorations, and Figure 2 (attached) depicts their approximate relative locations. The following sections describe the procedures used for excavation of the auger borings.

TABLE 1
APPROXIMATE LOCATIONS AND DEPTHS OF EXPLORATIONS

Exploration	Functional Location	Termination Depth (feet)
B-1	Northwest project corner	16.5
B-2	Northeast project corner	16.5
B-3	Along west side of site, near lot line	16.5
MW-1	Center of project area	31.5

The specific numbers and locations of our explorations were selected in relation to the existing site features, under the constraints of surface access, underground utility conflicts, and budget considerations.

It should be realized that the explorations performed and utilized for this evaluation reveal subsurface conditions only at discrete locations across the project site and that actual conditions in other areas could vary. Furthermore, the nature and extent of any such variations would not become evident until additional explorations are performed or until construction activities have begun. If significant variations are observed at that time, we may need to modify our conclusions and recommendations contained in this report to reflect the actual site conditions.

2.1 Auger Boring Procedures

Our exploratory borings were advanced through the soil with a hollow-stem auger, using a track-mounted drill rig operated by an independent drilling firm working under subcontract to MGI. An engineering technician from our firm continuously observed the borings, logged the subsurface conditions, and collected representative soil samples. All samples were stored in watertight containers and later transported to a laboratory for further visual examination. After each boring was completed, the borehole was backfilled with bentonite chips.

Throughout the drilling operation, soil samples were obtained at intervals of 2½ to 5 feet by means of the Standard Penetration Test (SPT) per American Society for Testing and Materials (ASTM:D-1586). This testing and sampling procedure consists of driving a standard 2-inch-diameter steel split-spoon sampler 18 inches into the soil with a 140-pound hammer free-falling 30 inches. The number of blows required to drive the sampler through each 6-inch interval is counted, and the total number of blows struck during the final 12 inches is recorded as the Standard Penetration Resistance, or "SPT blow count". If a total of 50 blows are struck within any 6-inch interval, the driving is stopped, and the blow count is recorded as 50 blows for the actual penetration distance. The resulting Standard Penetration Resistance values indicate the relative density of granular soils and the relative consistency of cohesive soils.

The enclosed boring logs (Appendix A) describe the vertical sequence of soils and materials encountered in each boring, based primarily on our field classifications and supported by our subsequent laboratory examination and testing. Where a soil contact was observed to be gradational, our logs indicate the average contact depth. Where a soil type changed between sample intervals, we inferred the contact depth. Our logs also graphically indicate the blow count, sample type, sample number, and approximate depth of each soil sample obtained from the borings, as well as any laboratory tests performed on these soil samples.

If any groundwater was encountered in a borehole, the approximate groundwater depth is depicted on the boring log. Groundwater depth estimates are typically based on the moisture content of soil samples, the wetted height on the drilling rods, and the water level measured in the borehole after the auger has been extracted.

The soils were classified visually in general accordance with the system described in Figure A-1, which includes a key to the exploration logs. Summary logs of the explorations are included as Figures A-2 through A-5.

3.0 SITE CONDITIONS

The following sections present our observations, measurements, findings, and interpretations regarding surface, soil, groundwater, seismic and infiltration conditions, and liquefaction potential.

3.1 Surface Conditions

As previously indicated, the project site consists of two undeveloped residential parcels, situated along the south side of 2nd Ave NE in Puyallup, Washington. The proposed development consists of two contiguous tax parcels, which encompass a total area of approximately 0.36 acres.

Topographically, the project area is level, with minimal grade change being observed over its extent. While the project area has not yet been graded, only several trees have been removed in the recent past. Otherwise, vegetation consists of native grasses.

No hydrologic features were observed on site such as seeps, springs, ponds, or streams, nor was there evidence of surface hydrology present. During our previous exploratory visit, groundwater was observed during auger drilling at approximately 12 feet deep. Groundwater was measured between 12 and 13 feet during the months of March and April, 2021 during monitoring well observations.

3.2 Soil Conditions

We observed subsurface conditions through the advancement of four borings across the project area. As noted above, three borings were drilled to 16.5 feet deep, and one temporary groundwater monitoring was installed to a depth of 31.5 feet deep. In general, explorations revealed relatively consistent subgrade conditions, consisting of a thin surface cap of topsoil, underlain by native, wet silty sands and sand with silt. Soils at shallow depths were observed during drilling operations to be in a very loose to loose condition, based on recorded blow counts during soil sampling. As drilling was conducted on March 26, 2021, groundwater was observed in each boring, at approximately 12 feet deep. As such, the soils encountered were moist to very saturated.

The native sands and silty sands were found to be graded at depth between fine to coarse grained within each layer of strata. These soils are directly associated with geologically recent meandering of the Puyallup River channel as well as archaic flood plain deposits. Holocene alluvium is typically described as being poorly consolidated beds of silts, sands, and some gravel, which can range from fine to coarse grained silts and sands. Intermittent beds of peat have been observed elsewhere in the valley, but not on this site directly. Some of the observed beds of silt and sand had some degree of mottling, indicating the area's propensity to hold water rather than transmit vertically.

In *the Geologic Map of the Tacoma 1:100,000-scale Quadrangle, Washington*, as prepared by the Washington State Department of Natural Resources (WSDNR) (2015), the project site is mapped as containing Qa, or Holocene-aged alluvium. The National Cooperative Soil Survey (NCSS) for the Pierce County Area, classifies soils onsite as 31A – Puyallup fine sandy loam, 0 to 3 percent slopes. These soils are directly associated with deposits from the Puyallup River. Our field observations generally correspond with the site classifications prepared by both the WSDNR and NCSS. An excerpt from the mapping is presented as Figure 3 (below).

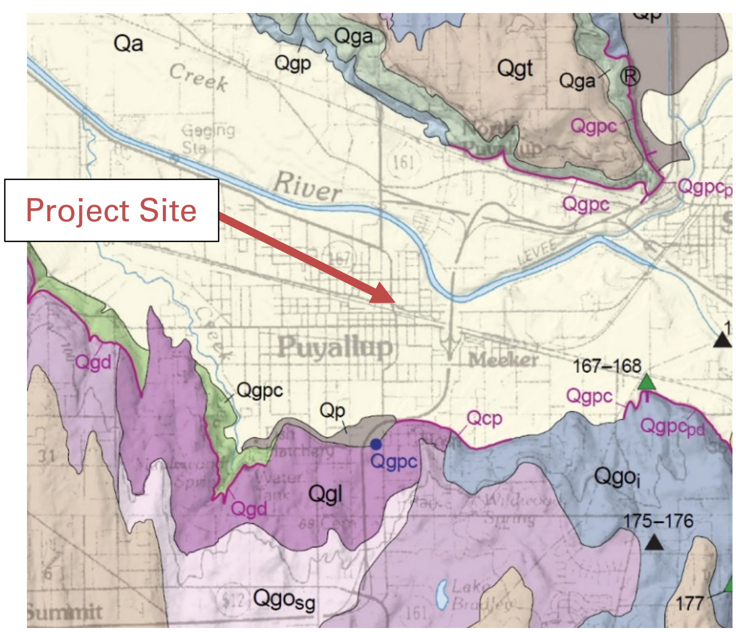


Figure 3. Excerpt from the Geologic Map of the Tacoma 1:100,000-scale Quadrangle, WA (2015)

The enclosed exploration logs (Appendix A) provide a detailed description of the soil strata encountered in our subsurface explorations.

3.3 Groundwater Conditions

At the time of our initial reconnaissance and subsurface explorations (March 26, 2021), we encountered groundwater in all drilling borings, which extended a maximum depth of 31.5 feet below existing grade. Given the fact that our explorations were conducted within what is considered the rainy season across Western Washington (November 1 to March 31), after what was considered a relatively wet fall and winter, groundwater levels will mostly likely not rise much higher than that which we observed. We anticipate moderately shallow groundwater around 10 feet deep will be present across the site during periods of extended precipitation. Groundwater levels will fluctuate with localized geology and precipitation levels. In addition, due to the loose and saturated nature of the alluvial sands observed in borings, shallow groundwater may result in slow seepage, which can be observed as "flowing sands", in deeper, open trench line excavations for utilities.

3.4 Seismic Conditions

The site is in the Puget Sound basin, which has experienced several earthquakes. A detailed description of the regional seismicity is beyond the scope of this report; however, previous regional earthquakes can be split into two general categories: 1) large earthquakes with a moment magnitude greater than 8.0 ($M_W > 8.0$) and 2) modest size earthquakes with a moment magnitude generally less than 7.25 ($M_W < 7.25$). In all cases, the thickness of the soil between the bedrock and the ground surface can change (usually amplify) the seismically induced ground motions and therefore the inertial loads acting on surface structures.

"Site Class" is a classification system used by the International Building Code (IBC) and ASCE 7 to provide some insight to the potential for ground motion amplification. The site class is based on the properties of the upper 100 feet of the soil and rock materials at the site. MGI used a combination of onsite explorations and our review of the geologic mapping of the site to derive a site class for the site. Based on evaluation and the definitions of Site Class as provided in Table 20.3-1 of ASCE 7-16 (as required by the 2018 IBC), the soil conditions on this site satisfy the definition of Site Class D – Default. Our evaluation assumes the soil conditions encountered in the bottom of our explorations, and those from nearby properties, is similar to or increasing in density/consistency down to 100 feet below ground surface.

The 2018 IBC considers earthquake shaking having a 2 percent probability of exceedance in 50 years (i.e., a 2475-year return period) as the code-based design requirement. Using the third-party graphical user interface tools made available by the USGS at https://seismicmaps.org, MGI derived the design ground motions to be used for design of the structures. Our evaluation used ASCE 7-16 as the code reference, Risk Category III, and Site Class D – Default. The results of our evaluation are provided in Table 2 (page 6).

TABLE 2 SEISMIC DESIGN PARAMETERS		
Parameter	**Value**	**Basis**
Site Class	D – Default	Site specific data
S_S	1.271	seismicmaps.org
F_a	1.2[A]	seismicmaps.org
S_{MS}	1.525	$= F_a \cdot S_S$
S_{DS}	1.017	$= {}^2/_3\ S_{MS}$
S_1	0.437	seismicmaps.org
F_V	1.563 [B, C]	2018 IBC
S_{M1}	0.683 [B, C]	$= F_V \cdot S_1$
S_{D1}	0.455 [B, C]	$= {}^2/_3\ S_{M1}$
PGA	0.500g	seismicmaps.org
PGA_M	0.600g	seismicmaps.org
T_0	-- [C]	Not applicable
T_S	-- [C]	Not applicable
T_L	6 sec	seismicmaps.org

Notes:

A. Use the value provided unless the simplified design procedure of ASCE 7 Section 12.14 is used. If this occurs, please contact our office for more information.

B. Based on Table 1613.2.3(2) of the 2018 IBC – An ASCE 7-16 Chapter 21 analysis has not been performed.

C. More detailed seismic design criteria are available upon request. Please contact MGI for more information.

<u>3.5 Liquefaction Potential</u>

Liquefaction is a sudden increase in pore water pressure and a sudden loss of soil shear strength caused by shear strains, as could result from an earthquake. Research has shown that saturated, loose, fine to medium sands with a fines (silt and clay) content less than about 20 percent are most susceptible to liquefaction below the water table. The fine-grained alluvial soils encountered beneath the project area, when saturated, should be considered a moderate to high risk for liquefaction and would likely liquefy during a large-scale seismic event, resulting in post-construction settlement. Foundation preparation recommendations contained within this report will help mitigate the risk for post-construction settlement.

<u>3.6 Infiltration Conditions and Infiltration Rate</u>

As indicated in the *Soil Conditions* section of the report, the site is underlain by poorly drained Puyallup River flood plain alluvial deposits. This material is comprised of saturated, poorly consolidated silts and sands. Given the variability of groundwater depths and localized consistency of native soils observed across the project area, it would be extremely difficult to accurately design any infiltration system. Considering the geologic and hydrogeologic conditions presents onsite, we do not interpret infiltration as being feasible for this project, and any stormwater generated onsite should be directed to an existing storm system along 2nd Ave E, or managed through other appropriate means.

4.0 CONCLUSIONS AND RECOMMENDATIONS

Improvement plans involve the construction of two multi-family apartment buildings along the north side of the project area. Site produced stormwater should be directed off site, as design of any infiltration system would be infeasible. We offer the following recommendations:

- Feasibility: Based on our field explorations, research and evaluations, the proposed structures and pavements appear feasible from a geotechnical standpoint.

- Foundation Options: Over-excavation of spread footing subgrades to a depth of 3 to 5 feet and construction of structural fill bearing pads will be necessary for foundation support of the proposed multi-family structures. If foundation construction occurs during wet conditions, it is likely that a geotextile fabric and/or a packed layer of quarry spall rock, placed between bearing pads and native soils, will also be necessary. Recommendations for spread footings are provided in Section 4.2.

- Floor Options: Based on explorations across the site, we recommend that floor sections be over-excavated to a minimum depth of 2 feet, then placement of suitable and properly compacted structural fill as a floor subbase. We do not anticipate that adequate bearing soils will be encountered within the top 5 feet, and we foresee the need for imported and compacted granular fill subbase. Recommendations for slab-on-grade floors are included in Section 4.3. If floor construction occurs during wet conditions, it is likely that a geotextile fabric and/or a packed layer of quarry spall rock, placed between bearing pads and native soils, will also be necessary. Fill underlying floor slabs should be compacted to 95 percent (ASTM:D-1557).

- Pavement Sections: After removal of any organic-rich materials underlying pavement areas, we recommend a conventional pavement section comprised of an asphalt concrete pavement over a crushed rock base course over properly prepared (compacted) subgrade or granular subbase. Given the relative loose/soft condition of native onsite soils, we recommend an over-excavation in proposed concrete and asphalt areas of 2 feet, with the placement and compaction of a suitable structural fill subbase.

  All soil subgrades below 2 feet should be thoroughly compacted then proof-rolled with a loaded dump truck or heavy compactor. Any localized zones of yielding subgrade disclosed during this proof-rolling operation should be over-excavated to a depth of 12 inches and replaced with a suitable structural fill material.

- Geologic Hazards: During our site reconnaissance, advancement of subsurface explorations, and general evaluation of the proposed development, we did not observe any erosional, landslide, seismic, settlement, or other forms of geologic hazards within the subject property. Given this fact, we recommend that no buffers, setbacks, or other forms of site restraints be implemented to address these potential hazards.

The following sections present our specific geotechnical conclusions and recommendations concerning site preparation, spread footings, slab-on-grade floors, drainage systems, asphalt pavement, and structural fill. The Washington State Department of Transportation (WSDOT) Standard Specifications and Standard Plans cited herein refer to WSDOT publications M41-10, *Standard Specifications for Road, Bridge, and Municipal Construction*, and M21-01, *Standard Plans for Road, Bridge, and Municipal Construction*, respectively.

4.1 Site Preparation

Preparation of the project site should involve erosion control, temporary drainage, clearing, stripping, excavations, cutting, subgrade compaction, and filling.

Erosion Control: Before new construction begins, an appropriate erosion control system should be installed. This system should collect and filter all surface water runoff through silt fencing. We anticipate a system of berms and drainage ditches around construction areas will provide an adequate collection system. Silt fencing fabric should meet the requirements of WSDOT Standard Specification 9-33.2 Table 6. In addition, silt fencing should embed a minimum of 6 inches below existing grade. An erosion control system requires occasional observation and maintenance. Specifically, holes in the filter and areas where the filter has shifted above ground surface should be replaced or repaired as soon as they are identified.

Temporary Drainage: We recommend intercepting and diverting any potential sources of surface or near-surface water within the construction zones before stripping begins. Because the selection of an appropriate drainage system will depend on the water quantity, season, weather conditions, construction sequence, and contractor's methods, final decisions regarding drainage systems are best made in the field at the time of construction. Based on our current understanding of the construction plans, surface, and subsurface conditions, we anticipate that curbs, berms, or ditches placed around the work areas will adequately intercept surface water runoff.

Clearing and Stripping: After surface and near-surface water sources have been controlled, sod, topsoil, and root-rich soil should be stripped from the site. Our explorations and field observations indicate that preliminary grading has been conducted across the bulk of the proposed residential lots, and stripping should be minimal. Stripping is best performed during an extended period of dry weather.

Site Excavations: Based on our field explorations, we anticipate that excavations will encounter loose/soft to medium dense/stiff silty, sandy alluvial soils. This material can be easily excavated utilizing standard excavation equipment.

Dewatering: We anticipate that site excavations will encounter groundwater at shallow depths during periods of extended precipitation. In addition, if excavations of trenchlines are left open for an extended period of time, slow seepage of groundwater may be observed. If groundwater is encountered during the course of regular earthwork activities, we anticipate that an internal system of ditches, sumpholes, and pumps will be adequate to temporarily dewater shallow excavations.

Temporary Cut Slopes: At this time, final designs and construction sequencing have not been completed. To facilitate project planning we provide the following general comments regarding temporary slopes:

- All temporary soil slopes associated with site cutting or excavations should be adequately inclined and covered in plastic sheeting to prevent sloughing and collapse,
- Temporary cut slopes in site soils should be no steeper than 1½H:1V, and
- Temporary slopes should conform to Washington Industrial Safety and Health Act (WISHA) regulations.

These general guidelines are necessarily somewhat conservative (steeper temporary slopes may be possible). As the project progresses, temporary grading plans are developed, final site features are better defined, and a contractor is engaged, MGI may modify these general guidelines to allow steeper slopes.

Subgrade Compaction: Exposed subgrades for the foundations of the planned structures should be compacted to a firm, unyielding state before new concrete or fill soils are placed. Any localized zones of looser granular soils observed within a subgrade should be compacted to a density commensurate with the surrounding soils. In contrast, any organic, soft, or pumping soils observed within a subgrade should be over-excavated and replaced with a suitable structural fill material.

Site Filling: Our conclusions regarding the reuse of onsite soils and our comments regarding wet-weather filling are presented subsequently. Regardless of soil type, all fill should be placed and compacted according to our recommendations presented in the *Structural Fill* section of this report. Specifically, building pad fill soil should be compacted to a uniform density of at least 95 percent (based on ASTM:D-1557).

Onsite Soils: We offer the following evaluation of these onsite soils in relation to potential use as structural fill:

- *Surficial Organic Soil and Organic-Rich Topsoil:* Where encountered, surficial organic soils, like duff, topsoil, root-rich soil, and organic-rich fill soils, are *not* suitable for use as structural fill under any circumstances, due to high organic content. Consequently, this material can be used only for non-structural purposes, such as in landscaping areas
- *Alluvial Silt and Silty Sand*: The alluvial silty sand that underlies the site is very moisture sensitive and will be difficult to impossible to reuse during most weather conditions. The majority of this soil type is currently above the optimum moisture content and will not compact adequately unless extensively aerated or treated with cement.
- *Alluvial Fine Sand*: Where encountered, and if properly segregated from its siltier counterpart, the native fine sands are a possible source of structural fill. This material type is relatively impervious to moisture content variations, and can be reused in most weather conditions.

Permanent Slopes: All permanent cut slopes and fill slopes should be adequately inclined to reduce long-term raveling, sloughing, and erosion. We generally recommend that no permanent slopes be steeper than 2H:1V. For all soil types, the use of flatter slopes (such as 2½H:1V) would further reduce long-term erosion and facilitate revegetation.

Slope Protection: We recommend that a permanent berm, swale, or curb be constructed along the top edge of all permanent slopes to intercept surface flow. Also, a hardy vegetative groundcover should be established as soon as feasible, to further protect the slopes from runoff water erosion. Alternatively, permanent slopes could be armored with quarry spalls or a geosynthetic erosion mat.

4.2 Spread Footings

In our opinion, conventional spread footings will provide adequate support for the proposed buildings, if the subgrades are properly prepared. Due to the soft soils that underlie the site, over-excavation of spread footing subgrades, to a depth of 3 to 5 feet, and the construction of structural fill bearing pads, will be necessary for foundational support of the new structures. We offer the following comments and recommendations for spread footing design.

Footing Depths and Widths: For frost and erosion protection, the bases of all exterior footings should bear at least 18 inches below adjacent outside grades, whereas the bases of interior footings need bear only 12 inches below the surrounding slab surface level. To reduce post-construction settlements, continuous (wall) and isolated (column) footings should be at least 18 and 24 inches wide, respectively.

Bearing Subgrades: Footings should bear on medium dense or denser, undisturbed native soils or properly compacted structural fill which bears on undisturbed medium dense to very dense native soils. Structural fill bearing pads, 3 to 5 feet thick and compacted to a density of at least 95 percent (based on ASTM: D-1557), should underlie spread footings for the proposed construction. If foundation work occurs during wet conditions, it is possible that a geotextile fabric, placed between the bearing pad and native soil, will be necessary. Refer to the *Structural Fill* section of this report.

In general, before footing concrete is placed, any localized zones of loose soils exposed across the footing subgrades should be compacted to a firm, unyielding condition, and any localized zones of soft, organic, or debris-laden soils should be over-excavated and replaced with suitable structural fill. Structural fill bearing pads should be compacted to a density of at least 95 percent (based on ASTM: D-1557).

Lateral Over-excavations: Because foundation stresses are transferred outward as well as downward into the bearing soils, all structural fill placed under footings, should extend horizontally outward from the edge of each footing. This horizontal distance should be equal to the depth of placed fill. Therefore, placed fill that extends 3 feet below the footing base should also extend 3 feet outward from the footing edges.

<u>Subgrade Observation</u>: All footing subgrades should consist of firm, unyielding, native soils or structural fill materials that have been compacted to a density of at least 95 percent (based on ASTM:D-1557). Footings should never be cast atop loose, soft, or frozen soil, slough, debris, existing uncontrolled fill, or surfaces covered by standing water.

<u>Bearing Pressures</u>: In our opinion, for static loading, footings that bear on dense, properly prepared bearing pads can be designed for maximum allowable soil bearing pressures listed in the following table:

Bearing Pad Thickness (feet)	Allowable Bearing Pressure (psf)
3	1,500
4	2,000
5	2,500

A one-third increase in allowable soil bearing capacity may be used for short-term loads created by seismic or wind related activities.

<u>Footing Settlements</u>: Assuming that structural fill soils are compacted to a dense or denser state, we estimate that total post-construction settlements of properly designed footings bearing on properly prepared subgrades will not exceed 1 inch, under static conditions. Differential settlements for comparably loaded elements may approach one-half of the actual total settlement over horizontal distances of approximately 50 feet.

<u>Footing Backfill</u>: To provide erosion protection and lateral load resistance, we recommend that all footing excavations be backfilled on both sides of the footings and stem walls after the concrete has cured. Either imported structural fill or non-organic onsite soils can be used for this purpose, contingent on suitable moisture content at the time of placement. Regardless of soil type, all footing backfill soil should be compacted to a density of at least 90 percent (based on ASTM:D-1557).

<u>Lateral Resistance</u>: Footings that have been properly backfilled as recommended above will resist lateral movements by means of passive earth pressure and base friction. We recommend using an allowable passive earth pressure of 250 psf and an allowable base friction coefficient of 0.35 for both soil types.

<u>4.3 Slab-On-Grade Floors</u>

In our opinion, soil-supported slab-on-grade floors can be used in structures if the subgrades are properly prepared. We offer the following comments and recommendations concerning slab-on-grade floors.

<u>Floor Subbase</u>: For the proposed multi-family residential structures, we recommend over-excavation of slab-on-grade floor subgrades to a minimum depth of 1.5 feet, then placement of properly compacted structural fill as a floor subbase. If floor construction is to occur during wet conditions, it is likely that a geotextile fabric and/or compacted layer of quarry spall rock, placed

between the structural fill floor subbase and native soils, will be necessary. All subbase should be compacted to a density of at least 95 percent (based on ASTM:D-1557).

Capillary Break and Vapor Barrier: To retard the upward wicking of moisture beneath the floor slab, we recommend that a capillary break be placed over the subgrade. Ideally, this capillary break would consist of a 4-inch-thick layer of pea gravel or other clean, uniform, well-rounded gravel, such as "Gravel Backfill for Drains" per WSDOT Standard Specification 9-03.12(4). Alternatively, angular gravel or crushed rock can be used if it is sufficiently clean and uniform to prevent capillary wicking. In addition, a layer of plastic sheeting (such as Crosstuff, Moistop, or Visqueen) be placed directly between the capillary break and the floor slab to prevent ground moisture vapors from migrating upward through the slab. During subsequent casting of the concrete slab, the contractor should exercise care to avoid puncturing the vapor barrier

4.4 Drainage Systems

We offer the following recommendations and comments for drainage design for construction purposes.

Perimeter Drains: We recommend that the proposed residential structural, where applicable, be encircled with a perimeter drain system to collect seepage water. This drain should consist of a 4-inch-diameter perforated pipe within an envelope of pea gravel or washed rock, extending at least 6 inches on all sides of the pipe, and the gravel envelope should be wrapped with filter fabric to reduce the migration of fines from the surrounding soils. Ideally, the drain invert would be installed no more than 8 inches above the base of the perimeter of the foundation.

Discharge Considerations: If possible, all perimeter drains should discharge to a municipal storm drain, or other suitable location by gravity flow. Check valves should be installed along any drainpipes that discharge to a sewer system, to prevent sewage backflow into the drain system.

Runoff Water: Roof-runoff and surface-runoff water should *not* discharge into the perimeter drain system. Instead, these sources should discharge into separate tightline pipes and be routed away from the buildings to a storm drain or other appropriate location.

Grading and Capping: Final site grades should slope downward away from the building so that runoff water will flow by gravity to suitable collection points, rather than ponding near the building. Ideally, the area surrounding the building would be capped with concrete, asphalt, or low-permeability (silty) soils to minimize or preclude surface-water infiltration.

4.5 Asphalt Pavement

To assist in the completion of the primary access road for this project off 2nd Ave NE, we offer the following comments and recommendations for pavement design and construction.

Subgrade Preparation: After removal of any organics underlying proposed areas of pavement, we recommend a conventional pavement section comprised of an asphalt concrete pavement over a crushed rock base course over a properly prepared (compacted) subgrade or a granular subbase. Given the relative loose/soft soil conditions observed across the site, we recommend the over-excavation of 24 inches of the existing subgrade material underlying the proposed pavement sections and replacement with a suitable structural fill subbase. We recommend limiting the subgrade preparation to times of dry weather. If this is not feasible, an alternative would be the cement-treatment of the proposed pavement subgrade to a minimum depth of 24 inches. Recommendations for Soil Cement Pavement Subbase are found in Section 4.7.

All soil subgrades below 24 inches should be thoroughly compacted, then proof-rolled with a loaded dump truck or heavy compactor. Any localized zones of yielding subgrade disclosed during this proof-rolling operation should be over excavated to an additional maximum depth of 12 inches and replaced with a suitable structural fill material. All structural fills should be compacted according to our recommendations given in the *Structural Fill* section. Specifically, the upper 2 feet of soils underlying pavement section should be compacted to at least 95 percent (based on ASTM D-1557), and all soils below 2 feet should be compacted to at least 90 percent.

Pavement Materials: For the base course, we recommend using imported crushed rock, such as "Crushed Surfacing Top Course" per WSDOT Standard Specification 9-03.9(3). If a subbase course is needed, we recommend using imported, clean, well-graded sand and gravel such as "Ballast" or "Gravel Borrow" per WSDOT Standard Specifications 9-03.9(1) and 9-03.14, respectively.

Conventional Asphalt Sections: A conventional pavement section typically comprises an asphalt concrete pavement over a crushed rock base course. We recommend using the following conventional pavement sections:

	Minimum Thickness		
Pavement Course	**Automobile Parking Areas**	**Driveways**	**Areas subjected to Heavy Traffic**
Asphalt Concrete Pavement	2 inches	3 inches	4 inches
Crushed Rock Base	4 inches	6 inches	8 inches
Granular Fill Subbase (if needed)	8 inches	12 inches	16 inches

Compaction and Observation: All subbase and base course material should be compacted to at least 95 percent of the Modified Proctor maximum dry density (ASTM D-1557), and all asphalt concrete should be compacted to at least 92 percent of the Rice value (ASTM D-2041). We recommend that an MGI representative be retained to observe the compaction of each course before any overlying layer is placed. For the subbase and pavement course, compaction is best observed by means of frequent density testing. For the base course, methodology observations and hand-probing are more appropriate than density testing.

Pavement Life and Maintenance: No asphalt pavement is maintenance-free. The above-described pavement sections present our minimum recommendations for an average level of performance during a 20-year design life; therefore, an average level of maintenance will likely be required. Furthermore, a 20-year pavement life typically assumes that an overlay will be placed after about 10 years. Thicker asphalt and/or thicker base and subbase courses would offer better long-term performance but would cost more initially; thinner courses would be more susceptible to "alligator" cracking and other failure modes. As such, pavement design can be considered a compromise between a high initial cost and low maintenance costs versus a low initial cost and higher maintenance costs.

4.6 Structural Fill

The term "structural fill" refers to any material placed under foundations, retaining walls, slab-on-grade floors, sidewalks, pavements, and other structures. Our comments, conclusions, and recommendations concerning structural fill are presented in the following paragraphs.

Materials: Typical structural fill materials include clean sand, gravel, pea gravel, washed rock, crushed rock, well-graded mixtures of sand and gravel (commonly called "gravel borrow" or "pit-run"), and miscellaneous mixtures of silt, sand, and gravel. Recycled asphalt, concrete, and glass, which are derived from pulverizing the parent materials, are also potentially useful as structural fill in certain applications. Import soils used for structural fill should not contain any organic matter or debris, nor any individual particles greater than 4 inches in diameter.

Fill Placement: Clean sand, gravel, crushed rock, soil mixtures, and recycled materials should be placed in horizontal lifts not exceeding 8 inches in loose thickness, and each lift should be thoroughly compacted with a mechanical compactor.

Compaction Criteria: Using the Modified Proctor test (ASTM:D-1557) as a standard, we recommend that structural fill used for various onsite applications be compacted to the following minimum densities:

Fill Application	Minimum Compaction
Footing subgrade and bearing pad	95 percent
Foundation backfill	90 percent
Slab-on-grade floor subgrade and subbase	95 percent
Asphalt pavement base and subbase	95 percent
Asphalt pavement subgrade (upper 2 feet)	95 percent
Asphalt pavement subgrade (below 2 feet)	90 percent

Subgrade Observation and Compaction Testing: Regardless of material or location, all structural fills should be placed over firm, unyielding subgrades prepared in accordance with the *Site Preparation* section of this report. The condition of all subgrades should be observed by geotechnical personnel before filling or construction begins. Also, fill soil compaction should be verified by means of in-place density tests performed during fill placement so that adequacy of soil compaction efforts may be evaluated as earthwork progresses.

Soil Moisture Considerations: The suitability of soils used for structural fill depends primarily on their grain-size distribution and moisture content when they are placed. As the "fines" content (that soil fraction passing the U.S. No. 200 Sieve) increases, soils become more sensitive to small changes in moisture content. Soils containing more than about 5 percent fines (by weight) cannot be consistently compacted to a firm, unyielding condition when the moisture content is more than 2 percentage points above or below optimum. For fill placement during wet-weather site work, we recommend using "clean" fill, which refers to soils that have a fines content of 5 percent or less (by weight) based on the soil fraction passing the U.S. No. 4 Sieve.

5.0 RECOMMENDED ADDITIONAL SERVICES

Because the future performance and integrity of the structural elements will depend largely on proper site preparation, drainage, fill placement, and construction procedures, monitoring and testing by experienced geotechnical personnel should be considered an integral part of the construction process. Subsequently, we recommend that MGI be retained to provide the following post-report services:

- Review all construction plans and specifications to verify that our design criteria presented in this report have been properly integrated into the design,
- Prepare a letter summarizing all review comments (if required),
- Check all completed subgrades for footings and slab-on-grade floors before concrete is poured, in order to verify their bearing capacity, and
- Prepare a post-construction letter summarizing all field observations, inspections, and test results (if required).

6.0 CLOSURE

The conclusions and recommendations presented in this report are based, in part, on the explorations that we observed for this study; therefore, if variations in the subgrade conditions are observed at a later time, we may need to modify this report to reflect those changes. Also, because the future performance and integrity of the project elements depend largely on proper initial site preparation, drainage, and construction procedures, monitoring and testing by experienced geotechnical personnel should be considered an integral part of the construction process. MGI is available to provide geotechnical monitoring of soils throughout construction.

We appreciate the opportunity to be of service on this project. If you have any questions regarding this report or any aspects of the project, please feel free to contact our office.

Respectfully submitted,

MIGIZI GROUP, INC.

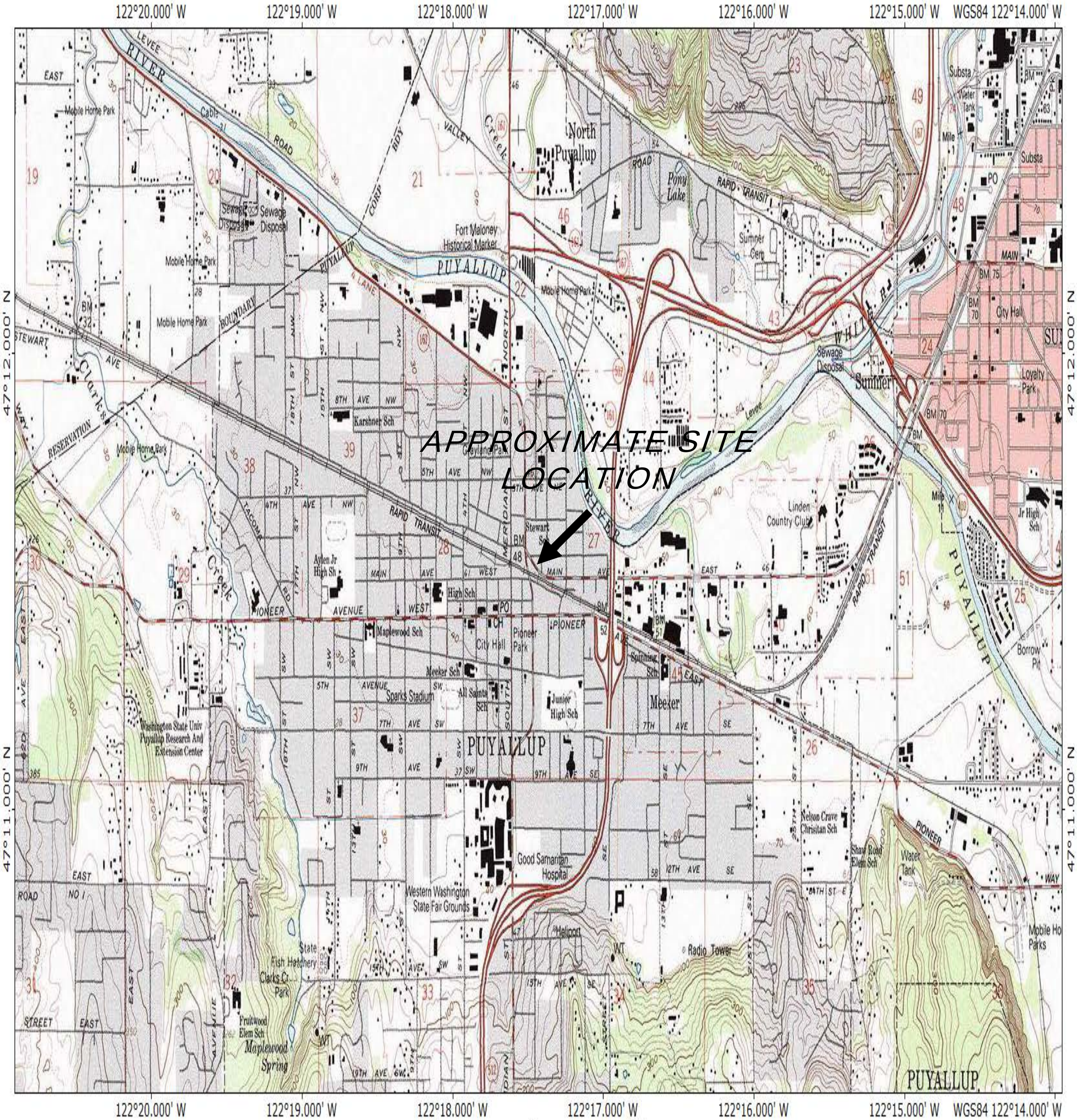
JAMES E. BRIGHAM
STATE OF WASHINGTON
20540
REGISTERED
PROFESSIONAL ENGINEER

01/26/22

Randall V. Conger-Best, G.I.T.
Staff Geologist

James E. Brigham, P.E.
Senior Principal Engineer

TOPO! map printed on 01/13/22 from "Untitled.tpo"



TN MN 16°

0 .5 1 MILE

0 1000 FEET 0 500 1000 METERS

Map created with TOPO!® ©2003 National Geographic (www.nationalgeographic.com/topo)

	Location	Job Number	Figure
MIGIZI GROUP	312 2nd Ave NE Puyallup, WA	P2388-T21	1
P.O. Box 44840 Tacoma, WA 98448	Title: Topographic and Location Map		Date: 01/13/22

N
W E

BORING LOCATION
B-1

NOTE:
BOUNDARY AND TOPOGRAPHY ARE BASED ON MAPPING PROVIDED TO MIGIZI OBSERVATIONS MADE IN THE FIELD. THE INFORMATION SHOWN DOES NOT CONSTITUTE A FIELD SURVEY BY MIGIZI.

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PROJECT:	2nd Ave NE near Stewart Ave Puyallup, Washington		
SHEET TITLE:	Site and Exploration Plan		
DESIGNER:	JHB	JOB NO.	P2388-T21
DRAWN BY:	JHB	SCALE:	NTS
CHECKED BY:	JEB	FIGURE:	2
DATE:	Jan. 10, 2022	FILE:	Fig2.dwg

APPENDIX A
SOIL CLASSIFICATION CHART AND KEY TO TEST DATA

LOG OF BORINGS AND MONITORING WELL

MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS More than Half > #200 sieve	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES
		GRAVELS WITH OVER 15% FINES	GM	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS
			SP	POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 15% FINES	SM	SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES
			SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS More than Half < #200 sieve	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS

	Modified California	RV	R-Value
	Split Spoon	SA	Sieve Analysis
	Pushed Shelby Tube	SW	Swell Test
	Auger Cuttings	TC	Cyclic Triaxial
	Grab Sample	TX	Unconsolidated Undrained Triaxial
	Sample Attempt with No Recovery	TV	Torvane Shear
CA	Chemical Analysis	UC	Unconfined Compression
CN	Consolidation	(1.2)	(Shear Strength, ksf)
CP	Compaction	WA	Wash Analysis
DS	Direct Shear	(20)	(with % Passing No. 200 Sieve)
PM	Permeability	∇	Water Level at Time of Drilling
PP	Pocket Penetrometer	$\blacktriangledown$	Water Level after Drilling(with date measured)

SOIL CLASSIFICATION CHART AND KEY TO TEST DATA

Figure A-1

MIGIZI GROUP

Migizi Group, Inc.
PO Box 44840
Tacoma, WA 98448
Telephone: 253-537-9400

BORING NUMBER B-1

Figure A-2

CLIENT KVK Quality Construction LLC
PROJECT NAME Proposed Urban Residential Development
PROJECT NUMBER P2388-T21
PROJECT LOCATION 2nd Ave NE near Stewart Ave, Puyallup, WA
DATE STARTED 3/26/21 **COMPLETED** 3/26/21
GROUND ELEVATION **HOLE SIZE** 4.25" HSA
DRILLING CONTRACTOR Holocene Drilling Inc.
GROUND WATER LEVELS:
DRILLING METHOD D50 Track Rig
AT TIME OF DRILLING 12.00 ft
LOGGED BY DSZ **CHECKED BY** JEB
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	BLOW COUNTS (N VALUE)	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0				GP-GM	2.0	(GP-GM) Dark brown gravel with silt and fine to medium sand (medium dense, moist) (Fill)
	SS S-1	18	3-2-1 (3)	SM		(SM) Gray/brown mottled fine silty sand (very loose, moist)
5	SS S-2	0	1-1-1 (2)			
	SS S-3	12	2-1-1 (2)			
10	SS S-4	12	1-1-1 (2)		12.0	
				SP		(SP) Dark gray fine to medium sand (loose, wet)
15	SS S-5	12	1-2-4 (6)		16.5	

Bottom of borehole at 16.5 feet.

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BORING NUMBER B-2

CLIENT KVK Quality Construction LLC
PROJECT NAME Proposed Urban Residential Development
PROJECT NUMBER P2388-T21
PROJECT LOCATION 2nd Ave NE near Stewart Ave, Puyallup, WA
DATE STARTED 3/26/21 **COMPLETED** 3/26/21
GROUND ELEVATION **HOLE SIZE** 4.25" HSA
DRILLING CONTRACTOR Holocene Drilling Inc.
GROUND WATER LEVELS:
DRILLING METHOD D50 Track Rig
AT TIME OF DRILLING 12.50 ft
LOGGED BY DSZ **CHECKED BY** JEB
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	BLOW COUNTS (N VALUE)	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0					0.5	Sodded topsoil
				SM		(SM) Gray/brown mottled silty sand (very loose, moist)
	SS S-1	12	2-2-2 (4)			
5	SS S-2	12	1-3-2 (5)			Grades to loose
	SS S-3	18	1-0-1 (1)		8.5	
				SP-SM		(SP-SM) Brown fine sand with silt (very loose, moist)
10	SS S-4	18	2-2-3 (5)			Grades to dark brown and loose
					15.0	
15	SS S-5	18	2-4-4 (8)	SP	16.5	(SP) Dark gray fine to medium sand (loose, wet)

Bottom of borehole at 16.5 feet.

BORING NUMBER B-3

PAGE 1 OF 1
Figure A-4

Migizi Group, Inc.
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CLIENT KVK Quality Construction LLC
PROJECT NAME Proposed Urban Residential Development
PROJECT NUMBER P2388-T21
PROJECT LOCATION 2nd Ave NE near Stewart Ave, Puyallup, WA
DATE STARTED 3/26/21 **COMPLETED** 3/26/21
GROUND ELEVATION **HOLE SIZE** 4.25" HSA
DRILLING CONTRACTOR Holocene Drilling Inc.
GROUND WATER LEVELS:
DRILLING METHOD D50 Track Rig
AT TIME OF DRILLING 12.50 ft
LOGGED BY DSZ **CHECKED BY** JEB
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	BLOW COUNTS (N VALUE)	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0					0.5	Sodded topsoil
				SP		(SP) Dark gray fine to medium sand (very loose, moist)
	SS S-1	6	2-2-1 (3)	SP		
5	SS S-2	12	2-2-2 (4)	SP	7.5	
	SS S-3	18	2-2-1 (3)	SP	8.5	(SP) Light gray sand (very loose, moist)
10				ML	10.0	(ML) Light gray silt (soft, moist)
	SS S-4	18	2-2-1 (3)	ML	11.0	(ML) Light gray/brown sandy silt (soft, moist)
				SM		(SM) Light brown silty sand (very loose, moist)
				SM	15.0	Mottling observed at 12 feet
15	SS S-5	0	2-2-1 (3)	SP	16.5	(SP) Dark gray fine to medium sand (loose, wet)

Bottom of borehole at 16.5 feet.

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BORING NUMBER MW-1

PAGE 1 OF 1
Figure A-5

CLIENT KVK Quality Construction LLC
PROJECT NAME Proposed Urban Residential Development
PROJECT NUMBER P2388-T21
PROJECT LOCATION 2nd Ave NE near Stewart Ave, Puyallup, WA
DATE STARTED 3/26/21 **COMPLETED** 3/26/21
GROUND ELEVATION **HOLE SIZE** 4.25" HSA
DRILLING CONTRACTOR Holocene Drilling Inc.
GROUND WATER LEVELS:
DRILLING METHOD D50 Track Rig
▽ **AT TIME OF DRILLING** 14.00 ft
LOGGED BY DSZ **CHECKED BY** JEB
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY (in) (RQD)	BLOW COUNTS (N VALUE)	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0					0.5	Sodded topsoil
	SS S-1	12	2-2-2 (4)	SP-SM	3.5	(SP-SM) Gray sand with silt (very loose, moist)
5				SM	5.0	(SM) Dark brown silty sand (very loose, moist)
	SS S-2	18	2-1-2 (3)	SM	7.5	(SM) Light gray silty sand with some mottling (very loose, moist)
10	SS S-3	12	2-2-3 (5)	SP-SM		(SP-SM) Light gray/brown sand with silt (loose, moist)
	SS S-4	18	2-3-3 (6)		▽ 15.0	
15	SS S-5	6	4-5-5 (10)	SP	20.0	(SP) Dark brown medium to coarse sand (loose, wet)
20	SS S-6	6	2-6-9 (15)	SP	25.0	(SP) Dark gray/brown fine to coarse sand with some gravel (medium dense, wet)
25	SS S-7	12	6-4-5 (9)	SP-SM	30.0	(SP-SM) Dark brownish-gray fine sand with silt and some gravel (loose, wet)
30	SS S-8	18	5-7-17 (24)	SP-SM	31.0	(SP-SM) Dark brown fine sand with silt and some gravel (medium dense, wet)
				ML	31.5	(ML) Light gray sandy silt (very stiff, wet)

Bottom of borehole at 31.5 feet.

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March 11, 2022

KVK Quality Construction LLC
7427 S D Street
Tacoma, WA 98408

Attention: Kon Kurkov

Subject: **Infiltration Evaluation Letter**
Proposed Urban Residential Development
312 2nd Ave NE
Puyallup, WA 98372
Parcel Nos. 7940100102 and 7940100103

MGI Project P2388-T21

Dear Mr. Kurkov:

Migizi Group, Inc. (MGI) is pleased to submit this infiltration evaluation letter for the proposed multi-family residential structure in Puyallup, Washington. We previously prepared a *Groundwater Monitoring Results* letter, dated May 19, 2021, and a *Geotechnical Engineering Report* for this project, dated January 26, 2022.

SITE AND PROJECT DESCRIPTION

The project site consists of two contiguous tax parcels, situated along the south side of 2nd Ave NE, just east of E Main Street, in downtown Puyallup, Washington as shown on the enclosed Topographic and Location Map (Figure 1). The project area is square-shaped, spanning 131 feet east to west and 120 feet north to south; encompassing a total area of approximately 0.36 acres. The entirety of the site is currently undeveloped and covered in native grasses with no shrubberies or trees on-site.

Improvement plans involve the construction of two multi-family apartment buildings with on-site paved parking stalls for tenants. A drive lane is proposed traveling north-south between these structures, and parking facilities are proposed towards the southern margin of the site. Preliminary layouts have the bulk of the improvement area being built close to current grade, which will require minimal grading. Site produced stormwater will be retained onsite if feasible. It is our understanding that an evaluation of infiltration conditions is requested prior to the evaluation of the proposed structures. As such, the purpose of this evaluation is to address infiltration feasibility.

EXPLORATORY METHODS

We explored subsurface conditions to calculate infiltration rates at the project site on March 2, 2022. Our infiltration evaluation program included the following elements:

- One Small-Scale Pilot Infiltration Test (PIT) (designated INF-1) performed towards the central portion of the improvement area.

Table 1 (below) summarizes the approximate functional locations and termination depths of our subsurface explorations, and Figure 2 (attached) depicts their approximate relative locations. The following sections describe the procedures used for excavation of the test pits.

TABLE 1
APPROXIMATE LOCATIONS AND DEPTHS OF EXPLORATIONS

Exploration	Functional Location	Termination Depth (feet)
INF-1	North-center of the proposed project area	2

The specific numbers and locations of our explorations were selected in relation to the existing site features, under the constraints of surface access, underground utility conflicts, and budget considerations.

<u>Infiltration Test Procedures</u>

In-situ field infiltration testing was performed for determination of a Design Infiltration Rate in general accordance with the Small-Scale Pilot Infiltration Test (PIT) procedures, as described in Vol. III-3.3.6 Design Saturated Hydraulic Conductivity of the 2014 Stormwater Management Manual for Western Washington, as adopted by the City of Puyallup Stormwater Management Department.

The first step of this test procedure was to identify a suitable soil stratum for stormwater retention, and once completed, perform an excavation within this soil group with a minimum surface area of 12 square feet (sf). Once the excavation was completed, a stabilized tape measure was installed towards the center of the test area. Water was then introduced into the test area, being conveyed through a 4-inch corrugated pipe to a splash block at the bottom of the excavation. Once 12 inches of water was developed at the bottom of the excavation, the test surface was saturated prior to testing. After the saturation period was completed, a steady state flow rate was developed in order to maintain 12 inches of head at the bottom of the test surface. This steady state rate was maintained for one hour. After completion of the steady state period, water was no longer introduced into the excavation, and infiltration of the existing water was allowed. As the test area drained in faster than one hour, we recorded the entirety of the falling head rate for 38 minutes, for comparison with the steady state rate.

SITE CONDITIONS

Groundwater Conditions

At the time of our subsurface infiltration testing (March 2, 2021), we measured groundwater in the on-site monitoring well at 11.56 feet bgs. Given the fact that our field explorations were conducted within what is generally considered the rainy season in Western Washington (November 1 to March 31), we anticipate that groundwater levels will not rise significantly higher than that which we observed. Given the hydrogeologic and/or topographic setting of the project, we anticipate that the bulk of the project excavations will encounter perched groundwater if conducted during the rainy season. Groundwater levels will fluctuate with localized precipitation and geology.

Infiltration Conditions

As indicated in the *Soil Conditions* section of this letter, the site is largely underlain by alluvium associated with the flood plain of the adjacent Puyallup River. This material ranges in composition from a fine to medium sand with silt to a deeper massive medium coarse sand deposit. Deposits observed in multiple borings indicate that the shallow soils consist of silty sand. Based on subsurface soil sampling, with soil mottling observed in B-1, B-2, and B-4 above a depth of 7.5 feet bgs (see *Geotechnical Engineering Report,* Jan. 26, 2022), and wet season groundwater monitoring observations, we do not believe that shallow groundwater will be a limiting factor when designing onsite infiltration systems. Based upon our observations, we believe that infiltration **will** be feasible using the shallow native soils as described above.

On March 2, 2022, a geologist from MGI performed field infiltration testing using the procedures noted at the onset of this letter. The field test (INF-1) was performed towards the north-central portion of the proposed work area, where pervious pavement would be installed as the main site entry. As described in the *Infiltration Test Procedures* section of this letter, there are two complementary portions of the Small PIT procedure used to determine a field infiltration rate: the steady-state period and the falling head period. In our experience, the falling head period is generally more conservative and provides a more accurate evaluation of infiltration conditions. The result of the falling head portion of our Small PIT procedure is recorded in Table 2 (below).

TABLE 2
FALLING HEAD PERIOD TEST RESULTS

Test Exploration	Depth of Test Surface (feet)	Field Infiltration Rate (in/hr)
INF-1	2	17.6

A design infiltration rate is determined by applying an appropriate correction factor to the measured infiltration rate. As described in the SWMMWW, this total correction factor (CF_T) should be equal to:

$$CF_T = CF_v \times CF_t \times CF_m$$

Where CF_v accounts for site variability and number of locations tested, CF_t accounts for uncertainty with the test method, and CF_m accounts for siltation and biofouling. The SWMMWW recommends using a value between 0.33 and 1 for CF_v, a value of 0.5 for CF_t, and a value of 0.9 for CF_m. For this evaluation we used a value of 0.5 for CF_v, giving us a $CF_T = 0.225$. Applying this value to our measured infiltration rate, we believe that the alluvial sands could support a design infiltration rate of **4.0 inches per hour**. As such, we believe that infiltration is feasible for this project, and site produced stormwater may be managed through methods such as pervious pavement, rain gardens, or an onsite infiltration gallery.

CONCLUSIONS AND RECOMMENDATIONS

Improvement plans involve the stripping and minimal grading of the site and the construction of two new multi-family apartment structures in the northeast and northwest corners of the project area. A drive lane is proposed traveling north-south between these structures, and paved parking stalls are proposed across the southern portion of the site. Site produced stormwater will be retained onsite if feasible. It is our understanding that an evaluation of infiltration conditions is requested prior to the evaluation of the proposed structures. As such, the purpose of this evaluation is to address infiltration feasibility. We offer the following conclusions and recommendations:

- <u>Feasibility</u>: Based on our field explorations, research, and evaluation, the utilization of bioretention and permeable pavements appears feasible for this project from a geotechnical standpoint.
- <u>Infiltration Conditions</u>: We believe that the shallow silty sand layers of native alluvium underlying the site could support a design infiltration rate of **6.0 inches per hour**. Based on observations of soil conditions and groundwater monitoring well water levels, we do not believe that groundwater levels will be a hindrance to proposed infiltration systems.
- <u>Site Preparation</u>: Before new construction begins, an appropriate erosion control system should be installed. This system should collect and filter all surface water runoff through silt fencing. We anticipate a system of berms and drainage ditches around construction areas will provide an adequate collection system. Silt fencing fabric should meet the requirements of WSDOT Standard Specification 9-33.2 Table 6. In addition, silt fencing should embed a minimum of 6 inches below existing grade. An erosion control system requires occasional observation and maintenance. Specifically, holes in the filter and areas where the filter has shifted above ground surface should be replaced or repaired as soon as they are identified.

RECOMMENDED ADDITIONAL SERVICES

Because the future performance and integrity of the structural elements will depend largely on proper site preparation, drainage, fill placement, and construction procedures, monitoring and testing by experienced geotechnical personnel should be considered an integral part of the construction process. Subsequently, we recommend that MGI be retained to provide the following post-letter services:

- Review all construction plans and specifications to verify that our design criteria presented in this letter have been properly integrated into the design,
- Prepare a letter summarizing all review comments (if required), and
- Prepare a post-construction letter summarizing all field observations, inspections, and test results (if required).

CLOSURE

The conclusions and recommendations presented in this letter are based, in part, on the explorations that we observed for this study; therefore, if variations in the subgrade conditions are observed at a later time, we may need to modify this letter to reflect those changes. Also, because the future performance and integrity of the project elements depend largely on proper initial site preparation, drainage, and construction procedures, monitoring and testing by experienced geotechnical personnel should be considered an integral part of the construction process. MGI is available to provide geotechnical monitoring of soils throughout construction.

We appreciate the opportunity to be of service on this project. If you have any questions regarding this letter or any aspects of the project, please feel free to contact our office.

Respectfully submitted,

MIGIZI GROUP, INC.

JAMES E. BRIGHAM
STATE OF WASHINGTON
20540
REGISTERED
PROFESSIONAL ENGINEER

03/11/22

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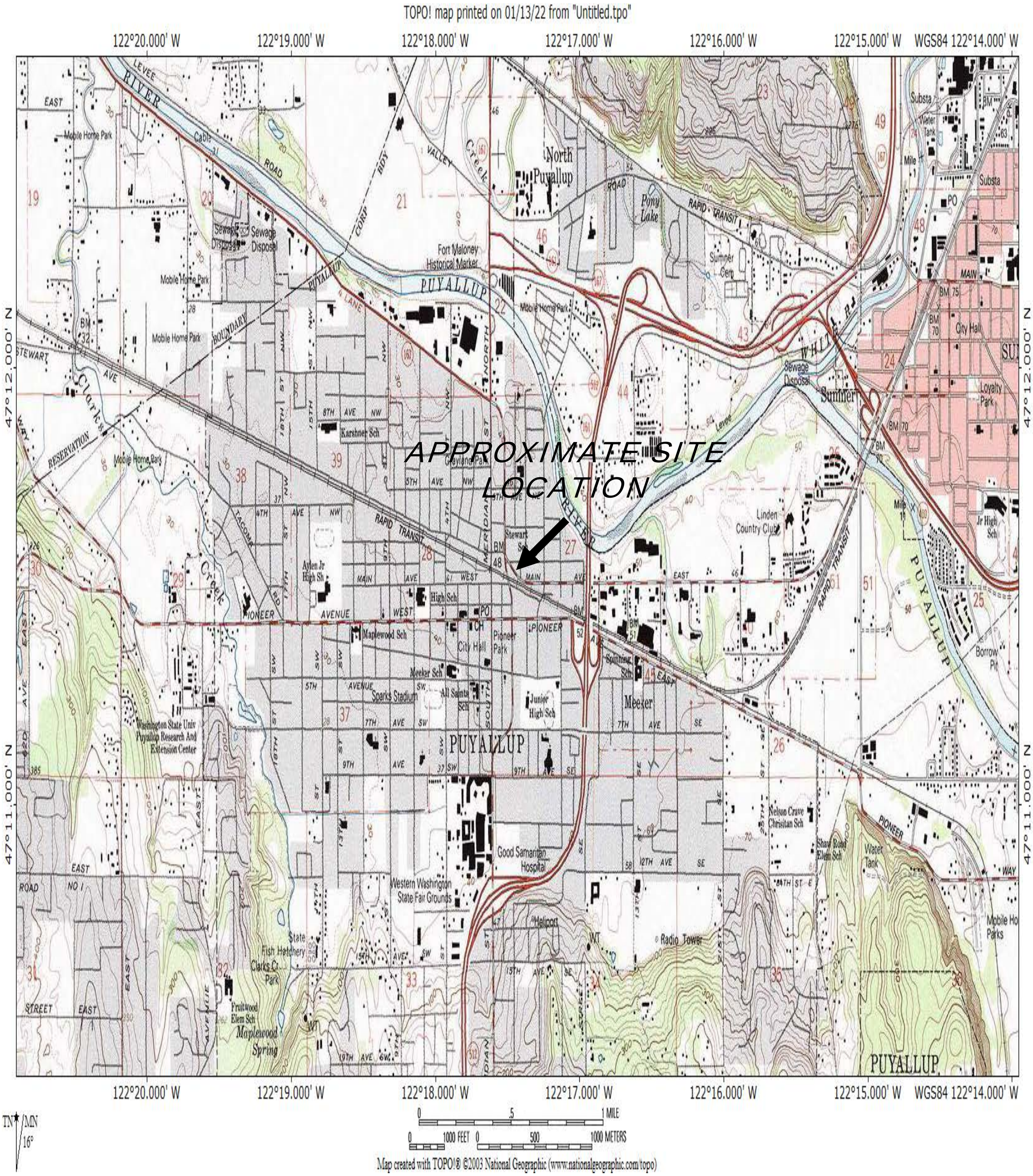
Staff Geologist

James E. Brigham, P.E.
Senior Principal Engineer

Attachments: *Figure 1. Topographic and Location Map*
Figure 2. Site and Exploration Plan



MIGIZI GROUP

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Location	Job Number	Figure
312 2nd Ave NE Puyallup, WA	P2388-T21	1

Title	Date
Topographic and Location Map	03/05/22

N
W E

BORING LOCATION
B-1

INFILTRATION TEST LOCATION
INF-1

NOTE:
BOUNDARY AND TOPOGRAPHY ARE BASED ON MAPPING PROVIDED TO MIGIZI OBSERVATIONS MADE IN THE FIELD. THE INFORMATION SHOWN DOES NOT CONSTITUTE A FIELD SURVEY BY MIGIZI.

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PROJECT:	2nd Ave NE near Stewart Ave Puyallup, Washington		
SHEET TITLE:	Site and Exploration Plan		
DESIGNER:	RVCB	JOB NO.	P2388-T21
DRAWN BY:	RVCB	SCALE:	NTS
CHECKED BY:	JEB	FIGURE:	2
DATE:	Mar. 03, 2022	FILE:	Fig2.dwg

Attachment D

Infiltration Memorandum by Migizi Group dated March 11, 2022