WILLIAM & MARGARET KANG
9706 55th Ave NE
Marysville, WA 98270

Attn: Benny Kim
Re: 9706 55th Ave NE Marysville, WA
Parcel #s: 30051500301800

At your request, we have conducted a soils exploration and foundation evaluation for the above mentioned project. The results of this investigation, and our recommendations, are to be found in the following report.

During our exploration, two borings were advanced and soil samples submitted for laboratory testing from the project site. Published literature and previous soil explorations have been carefully analyzed to determine soil bearing capacity. The proposed construction is well away from the 25 foot setback from the property line. The estimated bearing capacity is 2,000 psf on a built up soil section or mat foundation. Drainage is expected to be collected and tight lined to the city maintained storm system or to the existing (wetland) natural drainage location on the west side.

Infiltration of site soils averaged 5.17 inches/hour between 3 and 5 feet, however the water table is located about 4 feet below the surface. The site is classified as hydrologic class C.
The Cation Exchange capacity and the organic content tests are provided below.

Test	Result
Cation Exchange Capacity	1.6 meq/100g
Organic Content	0.4 %

We appreciate this opportunity to be of service to you and we look forward to working with you in the future. If you have any questions concerning this report, the procedures used, or if we can be of any further assistance please call us at **(206) 786-8645**.

Respectfully Submitted,
JASON ENGINEERING, INC.
Jason EC Bell
Geotechnical Engineer

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Investigation Information

Introduction

This report presents the results of a soils exploration and foundation analysis for the proposed addition located at 9706 55th Ave NE, Marysville WA. The soils exploration and analysis determines soil components, the engineering characteristics of the foundation materials and provides criteria for the design engineers and architects to prepare or verify the suitability of the foundation design. Written authorization to perform this exploration and analysis was provided by the owner, William Kang.

Parcel information, size and legal description are summarized below:

Address:	9706 55th Ave NE, Marysville, WA 98270-5205
Parcel:	30051500301800
Legal:	SEC 15 TWP 30 RGE 05RT-27C-1) NE1/4 S1/2 NW1/4 SW1/4 LESS CO RD
Lot size:	211,266 SF (4.85 AC)

Scope:

The scope of this geotechnical report and analysis included; a review of geological maps of the area, review of geologic and related literature, a reconnaissance of the immediate site, subsurface exploration, field and laboratory testing, and an engineering analysis and evaluation of the foundation materials to provide allowable bearing capacity, estimates of settlement, subgrade modulus, lateral earth pressure design values, geotechnical recommendations for the site including drainage and erosion control measures, as well as an evaluation of landslide and erosion hazards at the site per the Critical Areas regulations.

We were not requested to provide an Environmental Site Assessment for this property. Any comments concerning onsite conditions and/or observations, including soil appearances and odors, are provided as general information. Information in this report is not intended to describe, quantify or evaluate any environmental concern or situation.

The exploration and analysis of the site conditions reported herein are considered sufficient in detail and scope to form a reasonable basis for design. Any revision in the plans for the proposed structure from those enumerated in this report should be brought to the attention of the soils engineer so that he may determine if changes in the foundation recommendations are required. If deviations from the noted subsurface conditions are encountered during construction, they should also be brought to the attention of the soils engineer.

The soils engineer warrants that the findings, recommendations, specifications, or professional advice contained herein, have been promulgated after being prepared according to generally accepted professional engineering practice in the fields of foundation engineering, soil mechanics and engineering geology. No other warranties are implied or expressed.

This investigative report has been prepared for the exclusive use of William Kang and retained design consultants thereof. Findings and recommendations within this report are for specific application to the proposed project. All recommendations are in accordance with generally accepted soils and foundation engineering practices.

Drilling & Sampling Procedures:

Test pits were advanced to 8 feet below the existing ground surface using a track excavator. A site plan was obtained from the city GIS and used to locate test pits and borings used in the investigation. Measurements are presumed to be accurate to within a few feet. Samples were taken during our investigation. Soil logs are provided in the Appendix.

Site Information

Project Description

The purpose of this section is to describe details of the proposed structure. The following information was provided by the owner. The new construction will be a wood framed, 3 story apartment building with 58 units. Slab-on-grade floors are contemplated. Differential settlements are limited to ¾ inch. A pavement section has not been requested but has been provided within this report. The site of the proposed building addition upon which this soils exploration has been made is located at 7020 55th Ave NE in Seattle WA.

Access to the subject site is from the west side off 55th Ave NE or the south side off 97th Street NE. The parcel is bordered on the north, south and east sides by residential homes.

Location and Surface Conditions

The site topography consists of flat land. The site drainage consists of surface runoff and natural seepage. The site vegetation is grass. Prior grading has not occurred. NO hazardous areas for the site are shown per the County GIS. The total vertical relief of the subject site is about 2 feet over 690 feet with an average slope of less than 2% across the subject site.

Landslide hazard area categories

Landslide hazard areas are classified into categories which reflect each landslide hazard areas past landslide activity and the potential for future landslide activity based on an analysis of slope instability. There are no slopes near or around the site. No other hazard areas are indicated for the site per the county GIS. No landslides have been recorded on the subject site or surrounding parcels. No indications of landslides or soil disturbances were noted during the site investigation.

Geology of Area: Custer fine sandy loam

The geology of the site and surrounding area as taken from the USDA Soil Conservation Service Survey consists of Custer fine sandy loam. This very deep, poorly drained soil is in basins on outwash plains. It formed in glacial outwash. The native vegetation is mainly conifers and hardwoods. Elevation is near sea level to 150 feet. Slope is 0 to 2 percent. Included in this unit are small areas of Indianola soils on terraces, Norma soils in upland drainage ways, and Custer soils that have been partially drained. Included areas make up about 15 percent of the total acreage.

Typically, the surface layer is very dark grayish brown fine sandy loam about 9 inches thick. The upper part of the subsoil is loamy fine sand about 7 inches thick. The lower part is gray and olive sand about 19 inches thick and has iron-cemented concretions that form a discontinuous hardpan. The substratum is gray sand about 14 inches thick over gravelly coarse sand that extends to a depth of 60 inches or more. In some areas a hardpan is not present in the subsoil.

Permeability of this Custer soil is moderately slow in the discontinuous hardpan and very rapid below it. Available water capacity is low. Runoff is very slow. Ponding occurs from November to March. The main limitation for home sites is the seasonal high water table. Open ditches and tile drains around footings help to remove excess water. The main limitations for septic tank absorption fields are ponding, wetness, and moderately slow permeability. If effluent penetrates below the discontinuous hardpan, seepage into the water table is also a limitation. Cutbanks on this unit are subject to caving in.

Use of wheeled and tracked equipment when the soil is wet produces ruts, compacts the soil, and damages the roots of trees. Unsurfaced roads and skid trails are soft when wet, and they may be impassable during rainy periods.

Based on site visits and subsurface test pits, we are in agreement with the USGS classification for the site.

Soil Site Class and Geoseismic Setting:

Foundation soils on this site are designated Site Class D per the Washington State Department of Natural Resources map. All building structures on this project should be designed per Code Requirements for such a seismic classification. These types of soils have a shear wave velocity in the range of 600 to 1,200 ft/sec. The undrained shear strength is typically 1,000-2,000 psf with blow counts less than 30 blows per foot. Site specific coefficients were obtained from (https://hazards.atcouncil.org/). USGS Seismic Design Summary report is provided in the Appendix.

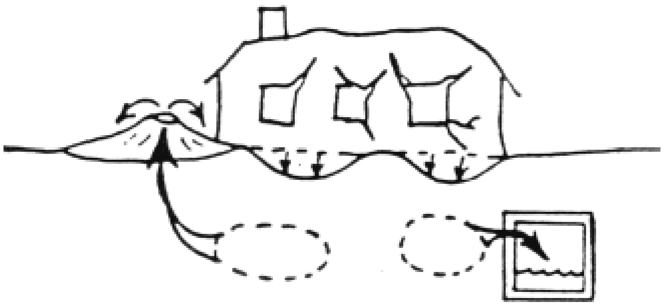
Liquefaction Potential:

Liquefaction is when saturated, cohesionless soils are temporarily turned in to a liquid state usually from a seismic event. If ground motion lasts for extended amounts of time, the grain to grain contact shifts and the grain structure can collapse. If the water within the soil cannot flow easily between the grain and out of a collapsing area, the water pressure increases. When pore pressures build up within the soil and exceed the effective contact pressure of the soil, the water can push the soil particles apart. When the particles lose contact with each other, the soil mass can behave like a liquid. If pore pressures are great enough, water may discharge out of the ground like a geyser leaving characteristic signs, such as sand boils. Liquefaction is generally related to; soil characteristics, water table depths and the degree of seismic activity. The results are lower bearing capacities, increased settlement issues, landslides, and lateral spreading to name a few things. Liquefaction potential for this site is provided within the boundaries of the site. Seismic events which affect land masses on a greater scale are beyond the scope of this report.

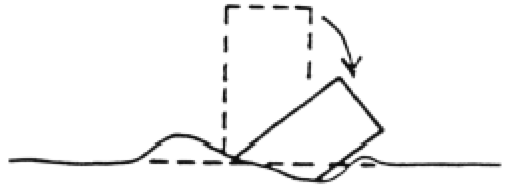
In our review we found no evidence of liquefaction of the soils in the immediate area from the 1949, 1965 and 2001 earthquakes. Information on the site has been reviewed on Liquefaction Susceptibility Map provided by the Department of Natural Resources which rates the site as having a LOW to MODERATE susceptibility to liquefaction.

The largest earthquakes in recent history in the Puget Sound Region are the 1949 surface wave (magnitude 7.1) in Olympia, the 1965 Seattle-Tacoma earthquake (magnitude 6.5) and the 2001 (magnitude 6.8). All of the historic liquefaction sites are located in the Duwamish valley in Holocene alluvium (Category I deposits). Liquefaction during the 1949 and 1965 earthquakes were mostly in the form of sand blows and surface cracking which was substantiated with many eyewitness observers living in the Pacific/Algona area. Broken water lines were reported in Auburn during the 1949 event suggests lateral spreading. Vertical ground water seepage around sewer manholes was also observed in Auburn, but no broken sewer lines were reported. From well records, Osceola deposits are 265 ft below sea level at a site 4 miles north of Auburn. The deposit is found at this depth because the Duwamish valley was an arm of the Puget Sound at that time. An important surface exposure of the Osceola Mudflow in a cut bank of the Puyallup River at Sumner suggests that the mudflow extended in the subsurface to Puyallup.

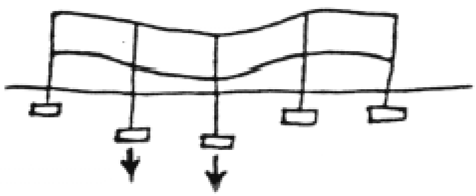
Sandy soils, and silty soils of very low plasticity, tend to experience "triggering" of cyclically induced soil liquefaction at relatively low shear strains (typically on the order of 3% to 6%), and the loss of strength can be severe. In other words, smaller displacements and stresses may result in liquefaction. Soils of higher plasticity, on the other hand, may also experience the same loss of strength and stiffness, and increased pore pressures. But the pore pressure ratios may be somewhat lower than those associated with more "classically" liquefiable soils, and the loss of strength and stiffness becomes pronounced at somewhat larger shear strains. The in-situ soils are non-plastic but also contain some cohesive properties. Non-plastic soils would typically liquefy quicker than plastic soils. The fact that these soils have cohesion, which is characteristic of a plastic soil, will give an additional safety factor against liquefaction. These soils are less likely to be "triggered" by small stresses and displacements. Larger stresses and pore pressures will need to build up in order to influence liquefaction. However, if the pressures do build up, in the case of a large seismic event, the effects could be severe. If liquefaction should occur, soil movements are likely to be one of the following instances:



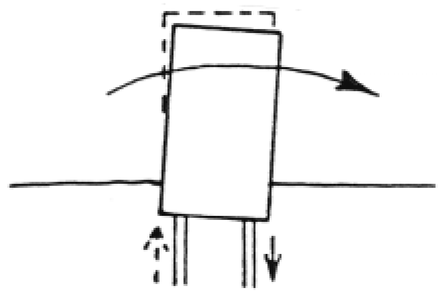
"Boil" ejecta from underground pools of free water.



Bearing failure by localized lateral soil movement.



Partial bearing failure by "punching" shear.



Differential settlement due to ground softening and inertial rocking.

Groundwater was noted during the site visit. Groundwater may be present during the wet season, or if excavations are left open for long periods of time.

- ✓ No hazardous areas for the site are shown per the County GIS
- ✓ There are no steep slopes on the property.
- ✓ We do not anticipate flooding in this area.
- ✓ The liquefaction susceptibility is LOW to MODERATE per the DNR maps.
- ✓ The erosion hazard is slight.

We utilized several methods to analyze the potential for liquefaction in this area. The most preferred method and currently used for the state of Washington is provided in 'WSDOT Evaluation of Liquefaction Hazards in Washington State Report': WA-RD 668.1 (December 2008) by Dr. Steven Kramer, Univ of Wash. It is very extensive and accounts for many factors including; groundwater elevation, geology, history, past seismic events, soil quality, and current compositional factors of the soil such as water content, particle shape, fines content, plasticity, and layers of impermeability. The results of the analysis provide a Susceptibility Rating Factor (SRF) to characterize the overall potential for liquefaction hazard. Included here are the results of our analysis using the WSDOT method which indicates a SRF = 1. According to the research, it matches the rating provided by the Department of Natural Resources as LOW to MODERATE. We are in agreement with this rating.

LIQUEFACTION HISTORY FACTOR				F_{hist} =	6.00
	C_{obs} =	6.0	Low to Moderate		
	C_{seis} =	1.0	PHA = 0.451g		
GEOLOGY FACTOR				$F_{geology}$ =	7.20
	C_{class} =	6.0	Outwash plains		
	$C_{quality}$ =	1.2	Engineer site visit, site maps		
COMPOSITIONAL FACTOR				F_{comp} =	0.95
	$C_{gradation}$ =	1.0	Cu = unknown		
	C_{shape} =	1.0	rounded		
	C_{fines} =	1.0	low fine content		
	$C_{plasticity}$ =	1.0	nonplastic		
	C_{wc} =	1.0	high water table		
	C_{cap} =	1.0	No cap		
GROUNDWATER FACTOR				F_{gw} =	1.00
Susceptibility Rating Factor				**SRF =**	**41**

Table 4.1 Characterization of overall site susceptibility to liquefaction hazards.

SRF	Site Susceptibility
0 – 5	Very Low
5 – 10	Low
10 – 25	Moderate
25 – 50	High
> 50	Very High

The most important factors for this site that reduce the liquefaction potential are:

- Proximity to the Puget Sound
- Low permeability soils
- Fines content

Percolation Rate:

At your request, we have performed a site specific PIT infiltration tests for the subject site. The tests were performed in accordance with KCSWDM standard Section 5.2.1 for a Pilot Infiltration test (PIT) in the test pit. The depth of the test was 4 feet beneath the existing surface. We compared the test results to mathematical models for estimating infiltration rates. Infiltration test results are summarized in the table below:

Pit ID#	Hazen Equation (in/hr)	PIT Infiltration Falling Head (in/hr)	PIT Infiltration Steady state (in/hr)	PIT Infiltration Steady state w/FS (in/hr)
1	xxx	14.1	11.5	5.17

The apparent infiltration rate is 5.17 inches/hour from the PIT test method.

HAZEN FORMULA

Foundation Discussion & Recommendations

General Notes:

Two requirements must be fulfilled for a successful foundation. First, the load must be less than the ultimate bearing capacity of the foundation soils to maintain stability; and secondly, the differential settlement must not exceed an amount that will produce adverse behavior of the superstructure. The allowable settlement is usually exceeded before bearing capacity considerations become important, thus the allowable bearing pressure is normally controlled by settlement considerations. Settlements are not expected to exceed tolerable limits.

Foundation:

On the basis of the data obtained from the site and the test results from the various laboratory tests performed, we estimate a net allowable soils bearing capacity of 2,000 psf on a 1.5 foot built up soil section of compacted crushed rock placed on the existing native soils. A conventional spread/column footing foundation is a suitable type of foundation for the support of the proposed structures. Footings shall be a minimum of 4.0 feet wide.

An alternative foundation may be a suitably reinforced mat foundation with 18 inch thickened edges under bearing walls. A capillary break system shall be used along with a perimeter drain system. A wider foundation is more stable during a seismic event and is more likely to retain structural integrity in case of liquefaction.

- Footings are required to be a minimum of 18 inches below grade for freeze thaw purposes. The excavation should be a minimum of 1 foot out from the side of the footings.

- Excavate footings areas to the native, undisturbed soils. These soils shall be confirmed for bearing capacity and verified by the soils engineer after excavation. Remove all organics below footing areas. Any excessively loose or soft spots shall be removed and replaced with at least 1 foot of additional suitable structural fill material and compacted to least 95% of the maximum dry density as determined by ASTM D-1557.

- Place at least 1.5 foot of crushed, structural fill material and compact to least 95% of the maximum dry density as determined by ASTM D-1557.

- Considering the subsurface conditions and the proposed construction, it is recommended that the structure be founded upon conventional spread footing foundations. We recommend a footing width of 4.0 foot (minimum) square footings and continuous strip footings.

The footings should be proportioned to meet the stated bearing capacity and/or the current minimum requirements of the current International Building Code. Total settlement should be limited to 1 inch total with differential settlement of ¾ inch. In order to minimize the effects of any slight differential movement that may occur due to variations in the characters of the supporting soils and any variations in seasonal moisture contents, it is recommended that all continuous footings be suitably reinforced to make them as rigid as possible.

Temporary Shoring & Excavation

We do not anticipate excavations deeper than 4 feet for this site. Shallow excavations required for construction of foundations that do not exceed four feet in depth may be constructed as needed. For deep excavations, side slopes are likely to naturally slough. The soils present cannot be expected to remain in position for extended periods and should be expected to fail, or collapse into any excavation thereby undermining the upper soils materials. This is especially true when working at depths near any groundwater or runoff. Temporary shoring should be implemented for cuts steeper than 1H:1V and greater than 4 feet in height. All excavations made for the foundations should be properly backfilled with suitable material compacted according to the procedures outlined in this report. Before the backfill is placed, all water and loose debris should be removed from these excavations.

Lateral earth pressures are dependent upon the backfill materials and their configuration and moisture content. There are no below grade retaining walls, or walls designed for retaining earthen fills on this project. Earth pressure coefficients are provided for completeness. Values were obtained based on a unit weight of 115 pcf, and a phi angle of 25 degrees for the native soil.

Earth Pressure Coefficients		**Earth Pressure**		
Active, K_a:	0.406	Active:	47	lbs./ft³
At Rest, K_o:	0.577	At Rest:	66	lbs./ft³
Passive, K_p:	2.464	Passive:	283	lbs./ft³

Coefficient of Friction: 0.35

It is our opinion that maintaining safe working conditions is the responsibility of the contractor. Proper care must be taken to protect personnel and equipment. Jobsite conditions such as soil moisture content, weather condition, earth movements and equipment type and operation can all affect slope stability. All excavations should be sloped or braced as required by applicable local, state and federal requirements.

Utilities

There are no existing utilities expected within the building area. There may be utilities along the sides of the parcel adjacent to existing houses. Care should be taken to avoid disruption or breakage of water, power, sewer, gas, cable, phone and any other utility that may exist. Call 811 prior to excavation to have utilities marked.

Groundwater Control:

Groundwater was encountered at the time the field exploration was conducted. Groundwater is expected to be present during construction or cause an issue for the proposed addition. The depth will vary throughout the year and correspond to rainfall amounts. If construction is performed during the dry season, groundwater may not be visible. With proper site drainage procedures, groundwater is not expected to cause difficulties during construction of this project. It is recommended that any runoff caused by wet weather be directed away from the construction area.

A perimeter footing drain is recommended around the building to prevent water from accumulating around the foundation. A sump pump just below the structural footing grade would also help to remove water from the around the foundation. All runoff will stay on site and slowly permeate in to the soil. This method will be slow and may result in marshy yards. A direct connection to the city maintained system through a tight-line system is feasible. Roof and surface drains should NOT be connected to any footing drain. Any and all roof drains should be rigid, solid PVC pipe and placed with positive gradient to allow gravity discharge away from the foundation to the drainage system. All runoff shall be collected and directed away from any open excavations. A closed tight line to 55th Ave NE is also feasible.

Structural Fill:

No structural fill is anticipated for the proposed construction. Structural fill should consist of a 3 inch minus select, clean, granular soil with no more than 5% fines (-#200). Suitable structural fill should consist of material that meets one of the following specifications, WSDOT Section 9-03.9(3) Crushed Surfacing (Base Course Specs), WSDOT Section 9-03.9(3) Crushed Surfacing (Top Course Specs), quarry spalls, or railroad ballast. Material that does not meet one of the specifications should be submitted with sieve analysis results for approval prior to placement.

Any fill should be placed in lifts not to exceed 12 inches in loose thickness. Each layer of structural fill should be compacted to a minimum density of 95% of the maximum dry density as determined by ASTM designation D-1557 or to the satisfaction of the geotechnical engineer. For structural fill below footings, the area of the compacted backfill must extend outside the perimeter of the foundation for a distance at least equal to the thickness of the fill between the bottom of the foundation and the underlying soils. If it is elected to utilize a compacted backfill for the support of foundations, the subgrade preparation and the placing of the backfill should be monitored continuously by a qualified engineer or his representative so that the work is performed according to these recommendations.

The use of on-site native soils as structural fill is not recommended.

Settlement

Organic material can compress and result in differential settlement that is detrimental the life and integrity of any foundation. Excessively organic top soils be removed and wasted or stockpiled for later use prior to the start of any construction. It is recommended that the final exposed subgrade be inspected by a representative of the soils engineer. This inspection should verify that all organic material has been removed. Any soft spots or deflecting areas should be removed and replaced with structural fill at least 1 foot below bottom of concrete footing.

Estimates were made for the total settlement over the lifespan of the structures based on the allowable bearing capacity. The majority of the settlement (primary settlement) will occur within in the first year, if not during construction. Larger footing loads will create larger settlement. Spreading the load out over a larger base will reduce the amount of total settlement.

The post construction settlement will be comprised of immediate settlement, primary settlement, and secondary (or long term) settlement. The rapidly occurring immediate and primary settlement will contribute to some of the settlement that occurs on the site. Approximately 60% of the settlement will occur during construction and the first month after construction. Settlement calculations are included in the Appendix.

Pavement Design Recommendations

Based on the soil conditions and the assumed traffic counts of the proposed project, the pavement profile should consist of the following recommendations:

<u>Existing Native:</u>

1) All subgrade preparation work to be performed should be monitored by a representative of our firm.
2) Excavate the existing soils to the native undisturbed soil with no organic material.
3) Over-excavate any areas that exhibit pumping of the subgrade soils at least 12 inches (or as directed by a representative from our firm). Replace with gravel base material. If subgrade is saturated or pumping excessively after over-excavating then it may be necessary to place quarry spalls on the subgrade prior to placement of any gravel base material.

4) Compact the existing soils to 95% of the maximum dry density as determined by ASTM D-1557 (Modified Proctor).
5) Under certain site conditions the existing subgrade can be accepted by proof-rolling the subgrade using a fully loaded dump truck. This procedure (if used) should be witnessed and accepted by a representative of our firm.

Subgrade:

1) The gravel base material should consist of 6.0 inches of material that is placed and compacted to 95% of the maximum dry density as determined by ASTM D-1557 (Modified Proctor).
2) The gravel base material should consist of a clean free draining granular material that has less than 10% passing the #200 sieve. This material should meet one of the following specifications, WSDOT Section 9-03.10 Aggregate for Gravel Base, WSDOT Section 9-03.14(1) Gravel Borrow, WSDOT Section 9-03.14(2) Select Borrow, APWA Class A Pit Run, or APWA class B Pit Run. Material that does not meet one of the specifications should be submitted for approval.
3) The material should be placed in lifts not to exceed 6 inches, with each lift being compacted and verified.

Crushed Aggregate Base:

1) The layer of crushed surfacing material should consist of 8.0 inches of WSDOT Section 9-03.9(3) Crushed Surfacing (Base Course Specs) that is placed and compacted to 95% of the maximum dry density as determined by ASTM D-1557 (Modified Proctor).
2) All of the gravel base and crushed surfacing material could, at the contractor's option, consist of WSDOT Section 9-03.9(3) Crushed Surfacing (Top Course Specs) that is placed and compacted to 95% of the maximum dry density as determined by ASTM D-1557 (Modified Proctor).
3) The crushed surfacing material should be placed to provide the proper grade and drainage for the asphalt pavement.

Asphalt Concrete Pavement:

1) The asphalt pavement should consist of at least 3.0 inches of WSDOT Class B asphalt that is placed and compacted to at least 91% of the theoretical maximum density as determined by ASTM D-2041 (Rice Method).
2) Provide a tack coat on all concrete surfaces that the pavement will be placed against, and for multiple lifts that are not placed within an hour time period.

AASHTO Pavement Section Design

Project Location: Margaret Estates, 9706 55th Ave NE Marysville

Average Daily Traffic Count: 500 Parking Lot, 58 units

Pavement Design Life: 20 Years

% of Traffic in Design Lane: 100%

Terminal Seviceability Index (P_t): 2.0

Level of Reliability: 95

Expected Growth Rate: 2.0%

Subgrade CBR Value: 10 **Subgrade M_r:** 15,000

Calculation of Design 18 kip ESALs

	Daily Traffic Breakdown	Load Factors	Design ESAL's
Passenger Cars:	153	0.0008	1,085
Buses:	3	0.6806	15,090
Panel & Pickup Trucks:	38	0.0122	4,057
2 Axle, 6 Tire Trucks:	10	0.1890	16,762
Concrete Trucks:	0	4.4800	2,980
Dump Trucks:	20	3.6300	643,856
Tractor Semi Trailer Trucks:	25	2.3719	525,882
Double Trailer Trucks	1	2.3187	20,563
Heavy Tractor Trailer Combo Trucks:	1	2.9760	26,393
Average Daily Traffic in Design Lane:	250		

Total Design Life 18 kip ESAL's:	1,256,668	
Actual Log (ESAL's):	6.099	
Trial Log (ESAL's):	6.241	
Trial SN:	3.00	**OK**

	Design Depth Inches	Structural Coefficient	Drainage Coefficient
Asphalt Concrete:	3.00	0.42	n/a
Asphalt Treated Base:	0.00	0.25	n/a
Cement Treated Base:	0.00	0.17	n/a
Crushed Aggregate:	8.00	0.14	1.0
Gravel Base:	6.00	0.11	1.0

Pavement Section Design SN: 3.04 **OK**

Conclusion

The results of the exploration and analysis indicate that conventional spread/column footing foundation is a suitable type of foundation for the support of the proposed structures. On site soils are suitable for a 2,000 psf foundation after 1.5 feet of compacted crushed rock placed on the existing native soils. Footings shall be a minimum of 4.0 feet wide.

An alternative foundation may be a suitably reinforced mat foundation with 18 inch thickened edges under bearing walls. A capillary break system shall be used along with a perimeter drain system.

There are no slopes of any significance on or around the site. The runoff shall be directed away from the building at least 10 feet. Infiltration is feasible in the upper layers of the soil, howver the groundwater table was noted at 4 feet below the surface. A perimeter drain pipe is recommended to prevent water from accumulating around the foundation. A direct connection to the city maintained system through a tight-line system is also feasible.

When the plans and specifications are complete, or if significant changes are made in the character or location of the proposed structures, a consultation should be arranged to review them regarding the prevailing soil conditions. Then, it may be necessary to submit supplementary recommendations. While the recommendations made herein are considered sufficient in detail for the construction of the proposed project, there are many alternative methods of construction which are available. We can discuss various other options for construction at your request.

Construction Considerations

Earthwork:

Excessively organic top soils generally undergo high volume changes when subjected to loads. This is detrimental to the behavior of pavements, floor slabs, structural fills and foundations placed upon them. Excavation equipment may disturb the bearing soils and loose pockets can occur at bearing levels that were not disclosed by any soil investigations. For this reason, it is recommended that the bottoms of any excavations be compacted in-place to achieve an in-place density of not less than 95% of the maximum dry density as determined by ASTM D-1557.

Excavations:

Excavation is anticipated for the project. The native soils can be expected to slough at depths greater than 4 feet, especially if the water is high. It is our opinion that maintaining safe working conditions is the responsibility of the contractor. Jobsite conditions such as soil moisture content, weather condition, earth movements and equipment type and operation can all affect slope stability. All excavations should be sloped or braced as required by applicable local, state and federal requirements.

Floor Slab-On-Grade:

Before the placing of concrete floors or pavements on the site, or before any floor supporting fill is placed, the organic, loose or obviously compressive materials must be removed. The subgrade should then be verified by the geotechnical engineer or his representative that all soft or deflecting areas have been removed. Areas of excessive yielding should be excavated and backfilled with structural fill.

Any additional fill used to increase the elevation of the floor slab should meet the requirement for structural fill. Structural fill should be placed in layers of not more than 12 inches in thickness, at moisture contents at or above optimum, and compacted to a minimum density of 95% of the maximum dry density as determined by ASTM designation D-1557.

A granular mat should be provided below the floor slabs. This should be a minimum of four inches in thickness and properly compacted. The mat should consist of sand or sand and gravel mixture with non-plastic fines. All material should pass a ¾ inch sieve and contain less than 10% passing the #200 sieve. Groundwater may be present at shallower depths during the winter months. A moisture barrier, such as visqueen or plastic sheeting, should be placed beneath all floor slabs that are within a foot of the water table, as determined during excavation.

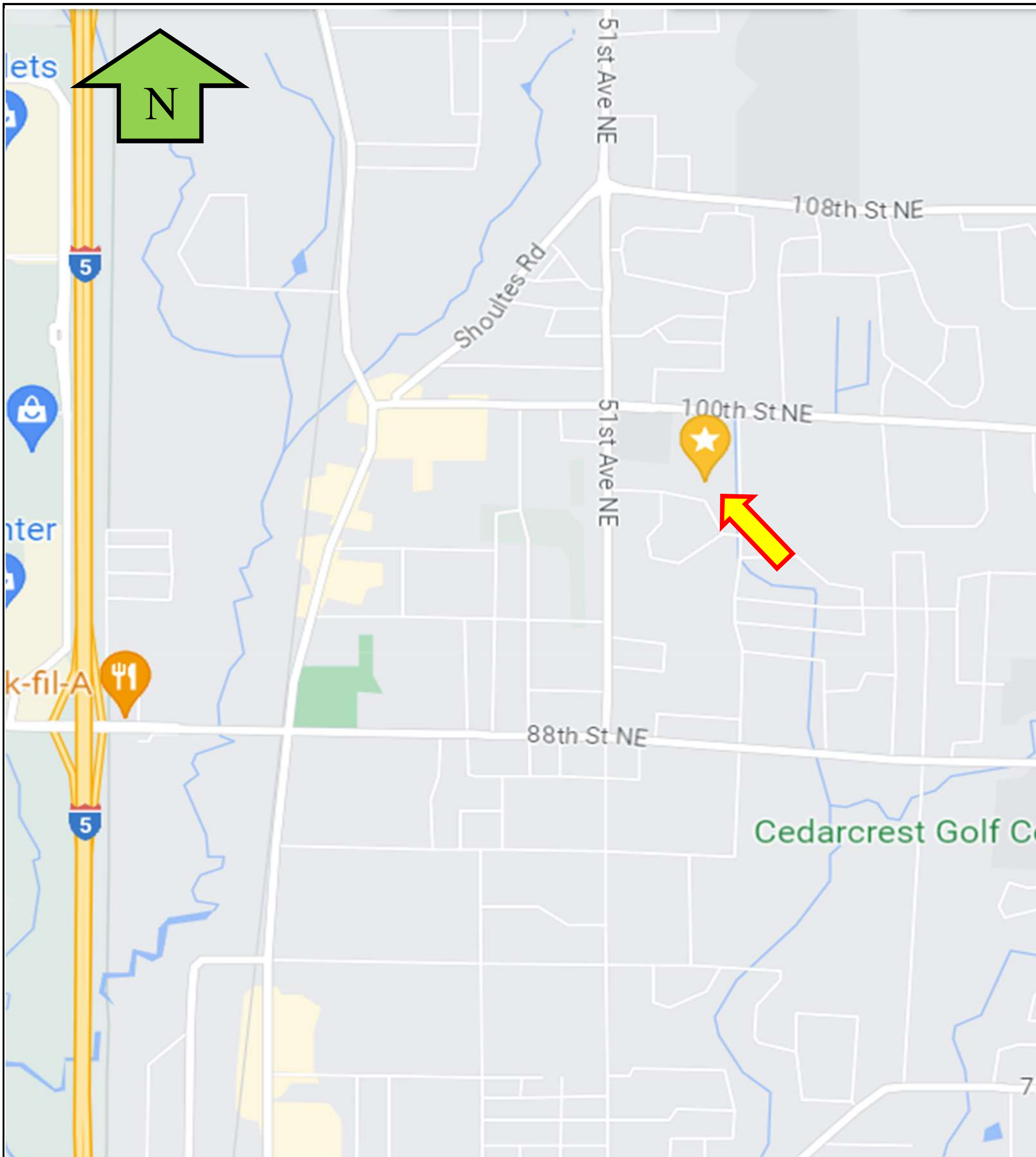
Erosion Control (typical)

1. The implementation of these ESC plans and the construction, maintenance, replacement, and upgrading of these ESC facilities is the responsibility of the owner/ESC supervisor until all construction is approved.
2. During the construction period, ESC facilities shall be upgraded as needed for unexpected storm events and modified to account for changing site conditions (e.g., additional sump pumps, relocation of ditches and silt fences, etc.).
3. The ESC facilities shall be inspected daily by the applicant/ESC supervisor and maintained to ensure continued proper functioning. Written records shall be kept of weekly reviews of the ESC facilities during the wet season (Oct. 1 to April 30) and of monthly reviews during the dry season (May 1 to Sept. 30).
4. Any areas of exposed soils, including roadway embankments, that will not be disturbed for two days during the wet season or seven days during the dry season shall be immediately stabilized with the approved ESC methods (e.g., seeding, mulching, plastic covering, etc.).
5. Any area needing ESC measures not requiring immediate attention shall be addressed within fifteen (15) days.
6. The ESC facilities on inactive sites shall be inspected and maintained a minimum of once a month or within forty-eight (48) hours following a storm event.
7. At no time shall more than one (1) foot of sediment be allowed to accumulate within a catch basin. All catch basins and conveyance lines shall be cleaned prior to paving. The cleaning operation shall not flush sediment-laden water into the downstream system.
8. Stabilized construction entrances and roads shall be installed at the beginning of construction and maintained for the duration of the project. Additional measures, such as

wash pads, may be required to ensure that all paved areas are kept clean for the duration of the project.

9. Any permanent flow control facility used as a temporary settling basin shall be modified with the necessary erosion control measures and shall provide adequate storage capacity. If the facility is to function ultimately as an infiltration system, the temporary facility must be graded so that the bottom and sides are at least three feet above the final grade of the permanent facility.

Appendix : Figures



Not to Scale

Margaret Estates Apartments
9706 55th Ave NE,
Marysville, WA 98270-5205
Parcel: 30051500301800

SITE LOCATION

Date: 2023.06.10

Figure A.1.0

File#: 23026

Jason Engineering - (206) 786-8645 - Jason@Jasonengineering.com

View looking west

View looking south east

Silty materials held together in top 3 ft until further excavation caused caving.

Mottled soils at 4 feet

Water level began at 6 feet below surface then rose to 4 ft during excavation

Coarse gray sand below 5 ft

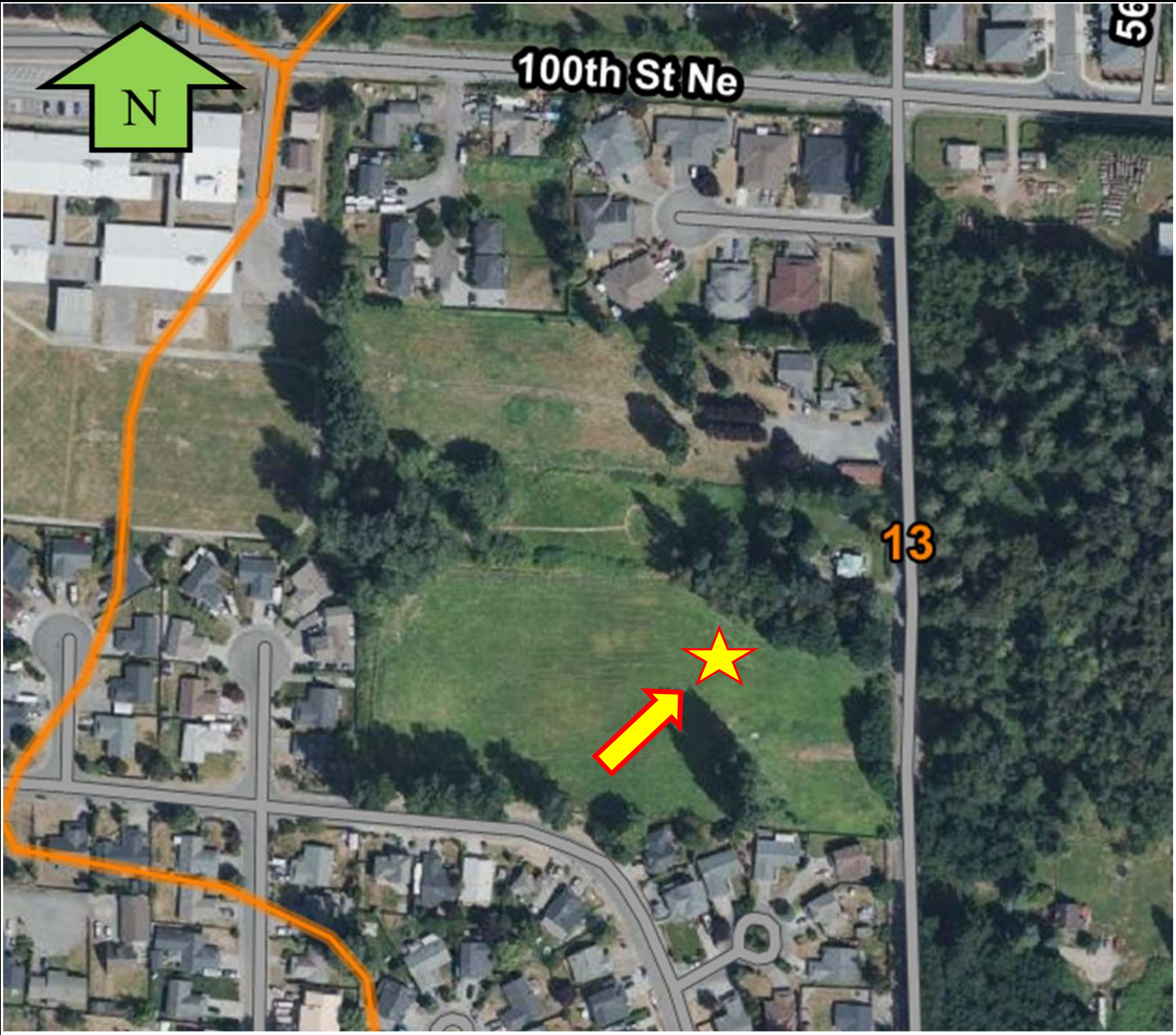
Margaret Estates Apartments 9706 55th Ave NE, Marysville, WA 98270-5205 Parcel: 30051500301800	Not to Scale SITE PHOTOS Date: 2023.06.10 Figure A.1.1

File#: 23026 Jason Engineering - (206) 786-8645 - Jason@Jasonengineering.com

NO CRITICAL AREAS PER THE COUNTY GIS

Margaret Estates Apartments 9706 55th Ave NE, Marysville, WA 98270-5205 Parcel: 30051500301800	Not to Scale GIS MAP Date: 2023.06.10 Figure A.2.0

File#: 23026 Jason Engineering - (206) 786-8645 - Jason@Jasonengineering.com



The geology of the site and surrounding area as taken from the USDA Soil Conservation Service Survey consists of Custer fine sandy loam. This very deep, poorly drained soil is in basins on outwash plains. It formed in glacial outwash. The native vegetation is mainly conifers and hardwoods. Typically, the surface layer is very dark grayish brown fine sandy loam about 9 inches thick. The upper part of the subsoil is loamy fine sand about 7 inches thick. The lower part is gray and olive sand about 19 inches thick and has iron-cemented concretions that form a discontinuous hardpan. The substratum is gray sand about 14 inches thick over gravelly coarse sand that extends to a depth of 60 inches or more. In some areas a hardpan is not present in the subsoil.

Margaret Estates Apartments 9706 55th Ave NE, Marysville, WA 98270-5205 Parcel: 30051500301800	Not to Scale USGS SOIL TYPE Date: 2023.06.10 Figure A.3.0
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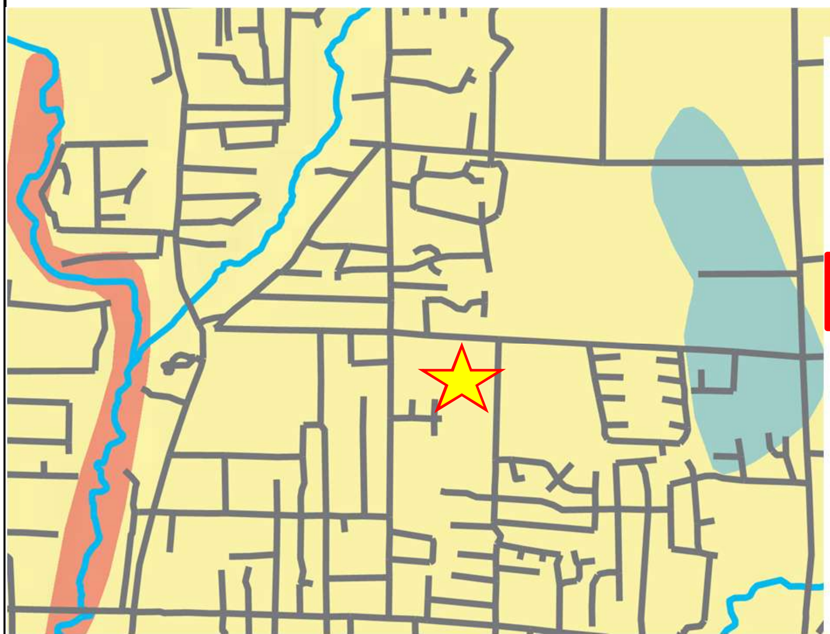
LIQUEFACTION SUSECEPTIBILITY



EXPLANATION

- Liquefaction susceptibility: HIGH
- Liquefaction susceptibility: MODERATE to HIGH
- Liquefaction susceptibility: MODERATE
- Liquefaction susceptibility: LOW to MODERATE
- Liquefaction susceptibility: LOW
- Liquefaction susceptibility: VERY LOW to LOW
- Liquefaction susceptibility: VERY LOW
- Bedrock
- Peat deposit — Peat is not susceptible to liquefaction but may undergo permanent displacement or loss of strength as a result of earthquake shaking.
- Water

SITE CLASS

EXPLANATION

- Site class F — Requires site-specific investigation
- Site class E
- Site class D to E
- Site class D
- Site class C to D
- Site class C
- Site class B to C
- Site class B
- Water

Increasing amplification of ground shaking

Not to Scale

Margaret Estates Apartments
9706 55th Ave NE,
Marysville, WA 98270-5205
Parcel: 30051500301800

LIQUEFACTION SUSECEPTIBILITY

SITE CLASS

Date: 2023.06.10

Figure A.4.0

File#: 23026

Jason Engineering - (206) 786-8645 - Jason@Jasonengineering.com

Excavation Date:	2023.06.10	Boring ID:	TP-1
Project Name:	Margaret Estates	Technician:	JB
Sample Method:	Track Exc	SPT:	NA
Total depth (ft):	8	Surface Elevation (ft):	75

Depth, ft	Sample: Moist %	Sample: type	Sample: USCS	SPT (N) blows per 6"	Description / Notes
1					Topsoil, brown, roots
2			SM-GP		SAND, tan, mottled orange
3					
4			SM		Silty SAND, orange
5					
6			SM		Silty SAND, gray, mottled orange
7					Groundwater found at 6 ft during excavation, rose to 4 ft by end of day
8					End test pit

Excavation Date:	2023.06.10	Boring ID:	TP-2
Project Name:	Margaret Estates	Technician:	JB
Sample Method:	Track Exc	SPT:	NA
Total depth (ft):	8	Surface Elevation (ft):	75

Depth, ft	Sample: Moist %	Sample: type	Sample: USCS	SPT (N) blows per 6"	Description / Notes
1					Topsoil, brown, roots
2			SM-GP		SAND, tan, mottled orange
3					
4			SM		Silty SAND, orange
5					
6			SM		Silty SAND, gray, mottled orange
7					Groundwater found at 6 ft during excavation, rose to 4 ft by end of day
8					End test pit

Margaret Estates Apartments
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Marysville, WA 98270-5205
Parcel: 30051500301800

SOIL LOGS

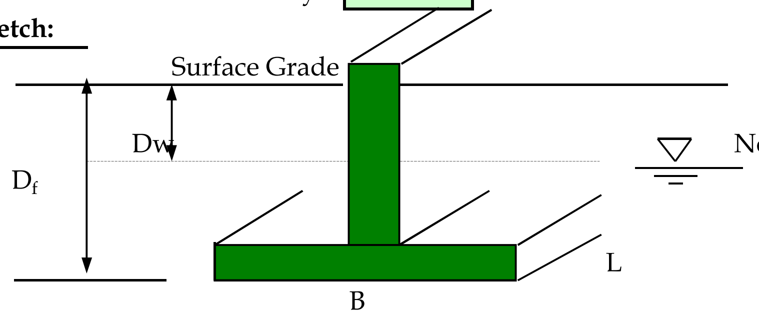
Date: 2023.06.10 Figure A.5.0

File#: 23026 Jason Engineering - (206) 786-8645 - Jason@Jasonengineering.com

Given: Strip Footing

In-situ density, γ_{sat} =	**115**	pcf	Cohesion, C =	**0**	psf
dry density, γ =	115	pcf	Width, B =	**1.50**	ft
footing depth, D_f =	**1.5**	ft	Length, L =	**15.00**	ft
depth of water table, Dw =	**10**	ft	Phi angle, ϕ =	**25**	degrees
Factor of Safety =	**3**		β =	**0**	degrees

Sketch:



No groundwater within 2x footing depth

DeterminAllowable Bearing Capacity (psf) for Footing size, B

Solution: q_{ult} = c Nc Fcs Fcd Fci + q Nq Fqs Fqd Fqi + 0.5 g B Ng Fgs Fgd Fgi (Meyerhof)

Table 3.4						
Nc =	20.70	Nq =	10.65	Ng =	10.87	(Vesic)
	20.70		10.65		8.10	(Brinch Hansen)
	20.70		10.65		6.76	(Meyerhof)

Shape Factors		Depth Factors		Inclination Factors	
Fcs =	1.051	Fcd =	1.400	Fci = Fqi =	1.000
Fqs =	1.047	Fqd =	1.311		
Fgs =	0.960	Fgd =	1.000	Fgi =	1.000

lete) q_{ult} =	0	+	2,521	+	900	**= q ult =**	**3,421**	psf	(Vesic)
lete) q_{ult} =	0	+	2,521	+	671	**= q ult =**	**3,192**	psf	(Brinch Hansen)
lete) q_{ult} =	0	+	2,521	+	560	**= q ult =**	**3,081**	psf	(Meyerhof)

Average of 3, using all soil factors and the applied safety factor

$q_{allowable}$ = 1,140 psf

Not to Scale

Margaret Estates Apartments 9706 55th Ave NE, Marysville, WA 98270-5205 Parcel: 30051500301800	BEARING CAPACITY Date: 2023.06.10 Figure A.6.0

File#: 23026 Jason Engineering - (206) 786-8645 - Jason@Jasonengineering.com

COLUMN (Terzaghi Method)			CONTINUOUS WALL (Terzaghi Method)		
Unit Weight of Soil, in lbs/ft^3=	115		Unit Weight of Soil, in lbs/cf=	115	
Average Corrected SPT N-value=	5		Average Corrected SPT N-value=	5	
Total Load (load, footing, soil) in kips=	2		Total Load (load, footing, soil), in kips=	2	
Soil Internal Friction Angle (from Fig. 9-9)=	15		Soil Internal Friction Angle (from Fig. 9-9)=	15	
General or local shear (determine for Fig.9-9)=					
N_g (using above results and figure 9-7)=	3.94		N_g (using above results and figure 9-7)=	3.94	
N_v (using above results and Figure 9-7)=	2.65		N_v (using above results and Figure 9-7)=	2.65	
N_c (using figure 9-7)=	10.97		N_c (using figure 9-7)=	10.97	
Unconfined Compressive Strength, Cohesion (kips/sf)=	0		Unconfined Comp. Strength, Cohesion (kips/sf)=	0	
Cohesion of Soil=	0		Unit Cohesion=	0	
Embedment Depth, in feet=	1.5		Embedment Depth, in feet=	2	
Footing Width (square), in feet=	3		Footing Width, in feet=	4	
Ultimate Bearing Capacity (lbs/ft^2)=	1,045		Ultimate Bearing Capacity (lbs/ft2)=	1,515	
Actual Bearing from Total Load (lbs/ft^2)=	222		Actual Bearing from Total Load (lbs/ft2)=	125	
F_s Against Bearing Capacity Failure (>3.0)=	**4.70**		Fs Against Bearing Capacity Failure (>3.0)=	**12.12**	
SETTLEMENT CHECK (for sand and SPT values only)					
Maximum Settlement on Dry Sand=	**0.10**	feet	**Maximum Settlement on Dry Sand=**	**0.06**	feet
	1.20	inches		0.77	inches
If encountered, depth to groundwater, in feet=	10		If encountered, depth to groundwater, in feet=	10	
Max Settlement on Wet Sand (if applicable)=	0.04	feet	**Max Settlement on Wet Sand (if applicable)=**	0.04	feet
	0.53	inches		0.42	inches

Margaret Estates Apartments 9706 55th Ave NE, Marysville, WA 98270-5205 Parcel: 30051500301800	**SETTLEMENT CALCULATIONS** Date: 2023.06.10 Figure A.7.0
File#: 23026	Jason Engineering - (206) 786-8645 - Jason@Jasonengineering.com

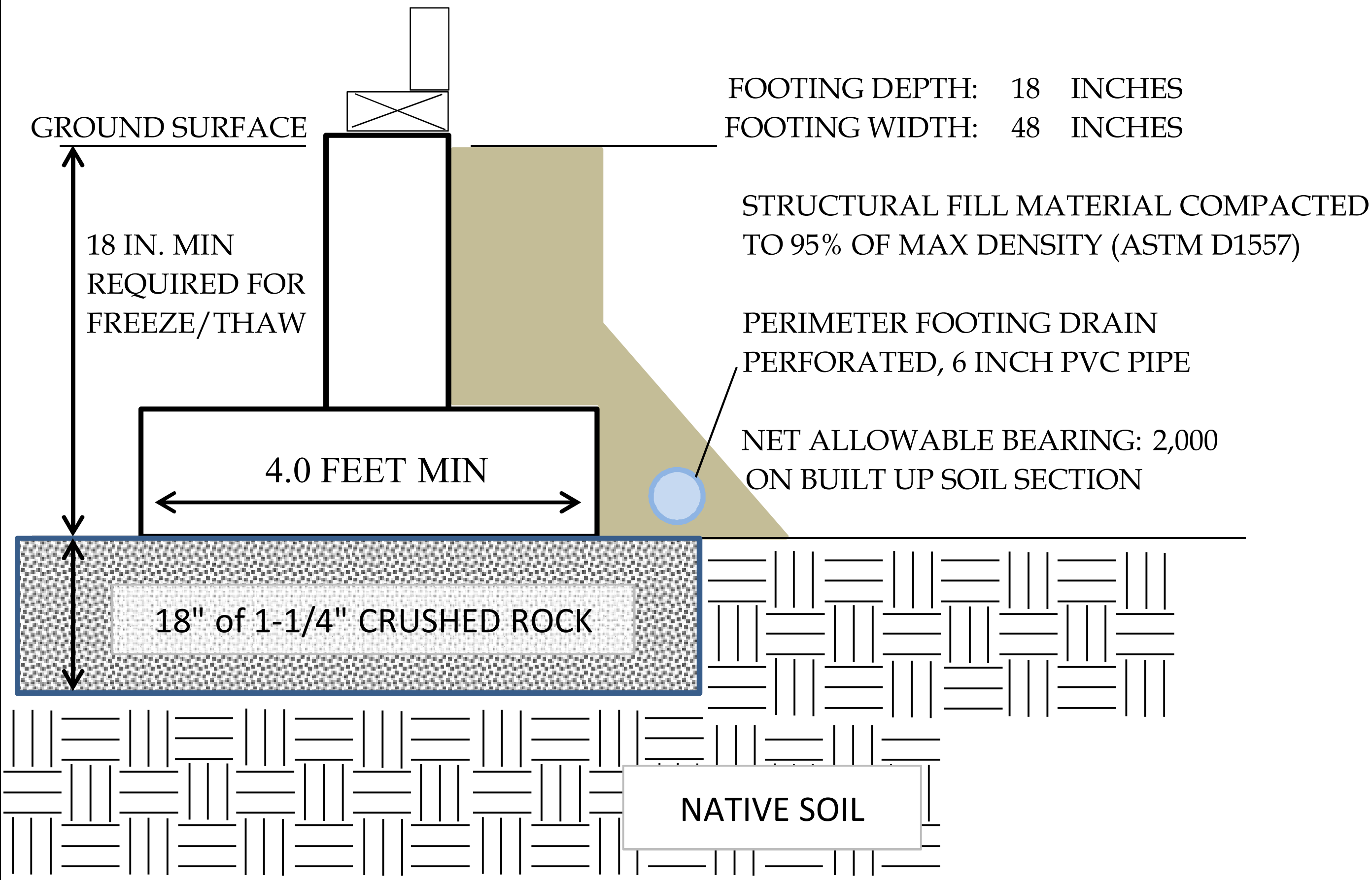
Name	Value	Description
S_S	1.119	MCE_R ground motion (period=0.2s)
S_1	0.434	MCE_R ground motion (period=1.0s)
S_{MS}	1.177	Site-modified spectral acceleration value
S_{M1}	0.68	Site-modified spectral acceleration value
S_{DS}	0.785	Numeric seismic design value at 0.2s SA
S_{D1}	0.453	Numeric seismic design value at 1.0s SA
SDC	D	Seismic design category
F_a	1.052	Site amplification factor at 0.2s
F_v	1.566	Site amplification factor at 1.0s
CR_S	0.974	Coefficient of risk (0.2s)
CR_1	0.946	Coefficient of risk (1.0s)
PGA	0.451	MCE_G peak ground acceleration
F_{PGA}	1.049	Site amplification factor at PGA
PGA_M	0.473	Site modified peak ground acceleration
T_L	6	Long-period transition period (s)
SsRT	1.119	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.149	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.434	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.459	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.6	Factored deterministic acceleration value (PGA)

Coordinates:
48.08344400000001, -122.158002
Reference Document:
IBC 2015
Hazard Type:
Seismic
Risk Catergory:
III
Site Class:
D
Elevation:
68

Margaret Estates Apartments
9706 55th Ave NE,
Marysville, WA 98270-5205
Parcel: 30051500301800

SEISMIC DESIGN DATA

Date: 2023.06.10 Figure A.8.0



ALERNATIVE: MAT FOUNDATION WITH 18" THICKENED EDGE UNDER ALL BEARING WALLS

NOT TO SCALE

Margaret Estates Apartments 9706 55th Ave NE, Marysville, WA 98270-5205 Parcel: 30051500301800	FOOTING DETAIL STANDARD FOOTING Date: 2023.06.10 Figure A.9.0

File#: 23026 Jason Engineering - (206) 786-8645 - Jason@Jasonengineering.com

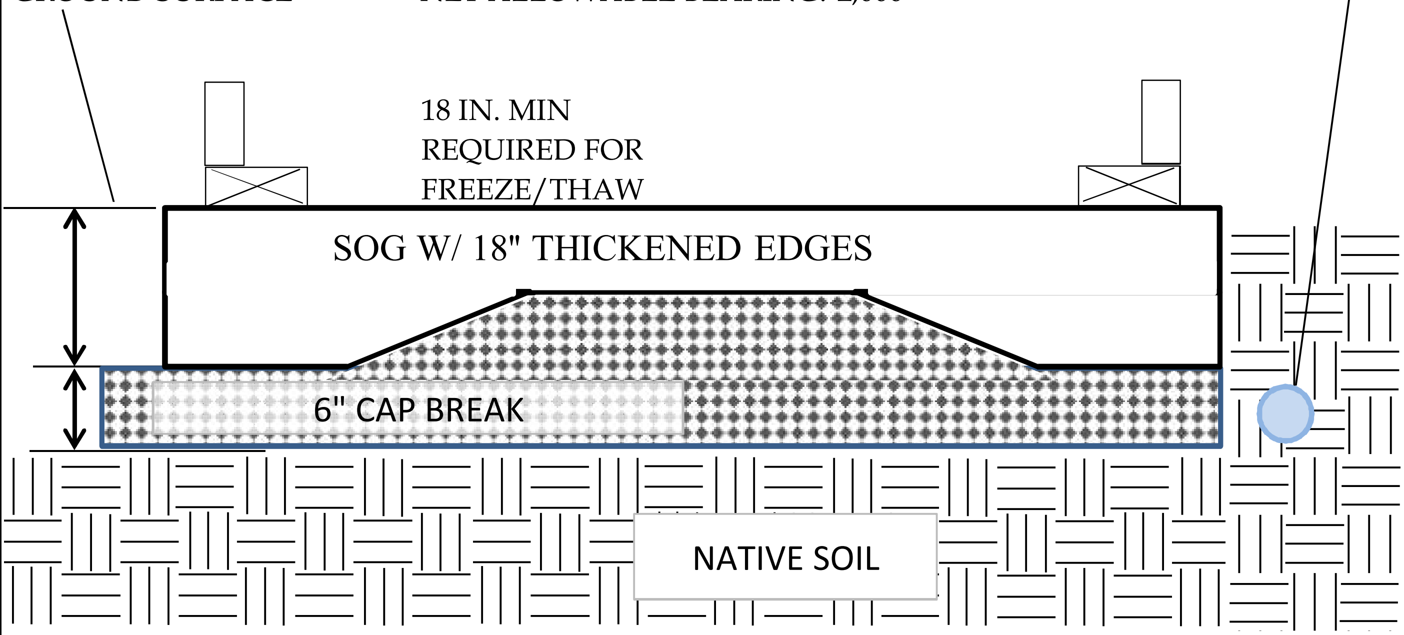
FOOTING DEPTH: 18 INCHES
FOOTING WIDTH: BUILDING WIDTH

STRUCTURAL FILL MATERIAL COMPACTED
TO 95% OF MAX DENSITY (ASTM D1557)

PERIMETER FOOTING DRAIN
PERFORATED, 6 INCH PVC PIPE

GROUND SURFACE

NET ALLOWABLE BEARING: 2,000



ALERNATIVE: MAT FOUNDATION WITH 18" THICKENED EDGE UNDER ALL BEARING WALLS

NOT TO SCALE

Margaret Estates Apartments
9706 55th Ave NE,
Marysville, WA 98270-5205
Parcel: 30051500301800

ALT. FOOTING DETAIL
MAT FOUNDATION

MAT 2023.06.10 Figure A.9.1

File#: 23026 Jason Engineering - (206) 786-8645 - Jason@Jasonengineering.com

Project Name: Margaret Estates
Project Address: 9706 55th Ave NE, Marysville, WA   Date: 6/10/2023
Permit Number:
Other Project Information: Parcel# 30051500301800

This infiltration test was performed by:
Company Name: Jason Engineering   Primary Contact Name: Jason Bell
Phone Number: 206-786-8645   Email Address: jason@jasonengineering.com

SMALL PILOT INFILTRATION TEST (PIT) AND LARGE PILOT INFILTRATION TEST (PIT):

Note: The test methods outlined below may be modified due to site conditions if recommended by the licensed professional and the reasoning is documented in the report.

A site map with test locations is included with this information.

1 Indicate type of test:
[X] Small Pit   [ ] Large Pit

2 Date and time of test: 6/10/2023, 10 am

3 Is the infiltration test within the footprint of the proposed infiltration facility
YES [X]   No [ ]

4 If "no" is testing being conducted within 50 feet of the proposed facility?
YES [ ]   No [ ]
Explain why:

5 What is the total proposed impervious area (does not include permeable pavement surfaces) to be infiltrated on the site? approximately 4,000 SF

6 Test pit excavated to bottom elevation of the proposed infiltration facility?
YES [X]   No [ ]

7 Test pit surface dimension (ft)   Length: 7.0   Width: 4.0   Depth: 3.0

8 Test pit bottom dimension (ft)   Length: 4.0   Width: 4.0

9 Test pit bottom area (ft^2)   16 = 2304 in^2

10 Small pit only: Is the surface area of the test pit bottom at least 12 ft^2
YES [X]   No [ ]

11 Large pit only: Is the surface area of the test pit bottom at least 32 ft^2
YES [ ]   No [ ]
a. If "no", indicate why?

12 Large pit only: The test pit bottom area should be as close to the bottom area of the proposed infiltration facility as feasible.
a. Bottom area of proposed infiltration facility:
b. Bottom area of test pit:

13 Identify device used to measure water level in test pit:
[ ] Pressure transducer (recommended for areas with slow draining soils)
[X] Vertical rod (min 5 ft. long, 1/2" increments, placed in center of pit)

14 Identify method of delivering water to the bottom of the test pit
Hose in a perforated pipe
(Method of delivery must reduce erosion that could cause clogging in the test pit)

15 ***Testing Procedure:***

a. Pre-soak period: Add water to maintain water level at least 12 inches above the bottom of the test pit for at least 6 hours. Record the time and depth of water hourly in the table below.

Time of Measurement	Depth of Water, inches
8:00	12
10:00	12
12:00	12
14:00	12

b. Steady-state period: The steady-state data is used to establish the measured infiltration rate (see step 16)

i. Add water to the test pit at a rate that will maintain a depth of 12 inches above the bottom of the test pit for 1 full hour. <u>During this hour, record time, depth of water, cummulative volume, and instantaneous flow rate every 15 minutes in the table below.</u>

ii. Calculate the infiltration rate for each 15 minute interval. First convert the flow rate to in3 /hr and the test pit bottom area (recorded in step 10) into in2. Divide the flow rate by the bottom area and record the result in the table below.

Time of Measurement (min)	Depth of Water (Inches)	Cumulative Volume (Gallons)	Flow Rate (gpm)	Flow Rate (in3/hr)	Infiltration Rate (In/hr)
0	12				
15	12	28.9	1.93	26704	11.59
30	12	57.9	1.93	26750	11.61
45	12	86.5	1.92	26642	11.56
60	12	114.7	1.91	26496	11.50

Note- 1 gallon = $231in^3$, $1ft^2 = 144in^2$ Test pit bottom area (ft^2) from step 9: 2304 in^2

c. Falling head period: The falling head data is used to confirm the measured infiltration rate calculated from the steady-state data.

i. At the end of the steady-state period, turn off all water and immediately record the time and depth of water in the table below. Record the time and depth of water every 15-minutes for a minimum of 1 hour, or until the pit is empty. (Note: in areas with slow draining soils, a pressure transducer is reccomended to improve the accuracy of change in depth readings. In addition, users are encouraged to extend the testing period and use longer intervals to improve accuracy.)

ii. Calculate the infiltration rate for each 15-minute interval (change in depth at each interval X 4) and record the results in the table below. Alternatively, users may also record the total time for fixed intervals of change in depth, and use those values to compute the infiltration rates.

Time of Measurement (minute intervals)	Depth of Water (Inches)	Infiltration Rate (In/hr)
0	12.0	
11	9.0	16.4
23	6.0	15.0
35.5	3.0	14.4
48.25	0.0	14.1

d. Check for high groundwater/immediate groundwater mounding:

1 Within 24 hours after the falling head period, excavate the bottom of the pit.

2 Is standing water or seepage visible in the excavation hole?

YES [ ] No [X]

3 If "yes" record depth: ________

16 ***Data Analysis "Measured Infiltration Rate" Selection*** (use the falling head data to confirm the measured infiltration rate calculated from the steady-state data):

a. Steady-state measured infiltration rate: Provide the lowest infiltration rate table above: 11.50 in/hr

b. Selected "Measured Infiltration Rate" 11.50 in/hr

(Include an explaination if the selected rate deviates from the steady state rate in step 16a)

____

c. If the lowest measured infiltration rate is less than the minimum rate associated with an infiltration BMP, that BMP can not be used.

d. If the measured infiltration rate is less than all minimum infiltration BMPs (see Table 1 in the reference table) no further investigation is required.

17 ***Calculate "Design Infiltration Rate":*** The desing infiltration rate shall be calculated by applying the appropriate correction factor to the above measured infiltration rate.

a. Select a correction factor.

CF v 1.00 CF t 0.50 CF m 0.90 CF = CFv*CFt*CFm 0.45

b. Calculate the "Design Infiltration Rate" below.

Design Infiltration Rate =	11.50	0.45	5.17
	Measured Rate Infiltration (In/hr)	X Correction Factor*	= Design Rate (in/hr)

* A Correction Factor may be used unless a different value is warranted by site conditions, as recommended and documented by a licensed professional.

I certify that I have followed the procedure outlined in this document to determine the infiltration BMP infiltration rate.

Date: 6/10/2023

Print Name: Jason Bell

Signature: Jason E Bell

Margaret Estates Apartments
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Parcel: 30051500301800

SIEVE ANALYSIS

Date: 2023.06.10 Figure A.11.0