March 26, 2024

Client: **Eddie Goodsir,** *owner*
Mac & Mac Electric Company, Inc.
1410 Iowa Street, Suite #102
Bellingham, WA 98229

Project: **Proposed Residential Redevelopment (#2024058)**
2302 Alabama Street, Bellingham, WA 98229 (WC Parcel #: 380320 540078)

Subject: **Geotechnical Evaluation & Stormwater Feasibility Assessment**

Dear Mr. Goodsir,

Element Solutions (ES) was retained by the client to perform a targeted geotechnical evaluation and stormwater infiltration feasibility assessment for the proposed project at the above referenced site. Final development plans were not available at the time of this report. The site was previously occupied by a single-family residence until recent years when it was fully demolished after a structure fire. We understand the project intends to redevelop the site with a small residence and detached ADU structure, a two- to three-unit townhome cluster, or a similar development concept in size and location. Buildings and pavement will be confined to the northwest part of the parcel, due to required minimum OHWM setbacks from the small creek (Fever Creek) that runs along the east and south boundaries of the parcel.

The purpose of this assessment was to provide the client and project team with information on soil characteristics of the study area, including subsurface composition, presence of groundwater, restrictive conditions if present, and interpretation of the potential for infiltration-based stormwater management as well as drainage protections. Additionally, this work has included an evaluation of key geotechnical aspects of foundation design and construction such as bearing capacity, footing subgrade, and general site preparations recommended for the project.

The project location has also been requested by the local municipality (City of Bellingham; COB) to have a geological review for geologically hazardous critical area conditions, and incorporation of mitigations as necessary, prior to permitting. For this site, potential geohazards to be evaluated include local steep slopes along the incised creek channel, and the potential erosion hazard related to the creek. Mapping/delineation of code-defined geohazard areas and evaluation of these aspects is requested prior to final site approval, and to inform development approach in proximity to critical area slope features. This letter summarizes the findings, interpretations, and conclusions of our geologic hazard assessment, including recommended buffers where applicable. This work has been conducted in accordance with Bellingham Municipal Code (BMC 16.55.4XX regarding Geologic Hazards), and the standard of care typical of the industry.

The scope of work completed to date has included:

- Desktop research of site features and mapped geologic/soil conditions.
- Observation of three (3) machine-excavated test pits with termination depths ranging from 6.3 to 7.0 feet below the existing grade surface in the proposed development area.
- Performance of one (1) dynamic cone penetrometer (DCP) test to a depth of approximately 3.1 feet BPG.
- Documentation of soil and groundwater conditions within test pits.
- Visual reconnaissance and interpretation of potential geologic hazard features.
- Interpretation of site-specific infiltration feasibility per governing criteria. *Note: Due to a lack of infiltrating conditions per field interpretation, no lab work was conducted.*
- Analysis of soil bearing capacity and determination of geotechnical foundation design parameters.
- Preparing this summary letter of the findings with our opinions and conclusions on the suitability of on-site stormwater management, geohazards, t and key geotechnical recommendations.

Subsurface conditions were assessed and cataloged by an ES Geologist during the March 12, 2024 field visit. A site vicinity map (Figure 1), an aerial photo map of test locations and general site features (Figure 2), a LiDAR Topographic Slope Map (Figure 3), field photos (Exhibit A), and test pit and DCP logs are attached in the Appendix. Should you have any questions concerning this report, please contact us at (360) 671-9172.

Physiography

The study area consists of a single parcel located along the south side of Alabama Street just east of the Woburn Street intersection, between Xenia and Yew Streets. The site is surrounded by developed single-family residential lots and small multi-family structures in all directions. A thin ROW defines the northern edge of the property along the Alabama Street frontage. The site is generally clear of vegetation along the north-central region of the site, with blackberry bushes and some mature trees along the east and southern boundaries of the site (along the Fever Creek channel). The property is accessed on foot from the north along Alabama Street. There is presently no developed vehicle access into the site, although the street pull-out remains.

The 0.23-acre parcel is rectangular in shape and primarily level along the proposed development area to gently sloping to the south-southeast to the south of the proposed structure. A small creek, known as Fever Creek, lines the eastern and southern boundaries of the site with gentle to steep ravine slopes on either side of the creek. Generally, creek slopes are between 30 to 60 percent grades with local areas exceeding 60 percent, and less than 8 feet in height within the parcel (discussed in detail below). The south part of the site is well vegetated with dense blackberry bushes and some mature trees. The northwest area of the site was previously developed with a single-family residence prior to 2014. The former home was damaged in a fire and subsequently demolished. Remnants of a small gravel-surfaced driveway are present along the north-central area of the site from prior development. The north half of the parcel outside of the creek channel area remains clear of large vegetation but is covered with maintained grass lawn.

Background Geology

The *Geologic Map of the Bellingham 1:100,000 Quadrangle, Washington* (Lapen, 2000) produced by the Washington Department of Natural Resources (DNR) indicates that the study area is underlain by thick Pleistocene glaciomarine drift ($Qgdm_e$). Glaciomarine drift (GMD) deposits are described as "moderately to poorly indurated, moderately to unsorted diamicton with lenses and discontinuous beds of moderately to well-sorted gravel, sand, silt, and clay" (Lapen, 2000).

In our experience, GMD soils are often capped at surficial levels by post-glacial sediments, as well as weathered/altered and weakened GMD in the upper few feet. These conditions tend to exhibit variable content and consistency locally. Below surface levels, the dominantly silt-and-clay-rich deposits are typically stiff to hard at upper levels, but tend to become lower in strength at lower depths under the influence of regional groundwater (typically 10 to 15 feet or below in the project vicinity based on past explorations). Shallow perched water is common where fine-grained GMD deposits reside from the surface among relatively flat to gently grading topography due to poor drainage and transmission character of GMD-related soils. The presence of seasonal perched water can weaken or influence properties of sensitive fine-grained soils in the upper profile.

<u>NRCS Soil Mapping</u>

The USDA Natural Resource Conservation Service (NRCS) Soil Survey (accessed online) has identified one local soil type in the study area; Urban land Whatcom-Labounty Complex, 0 to 8 percent slopes (map unit 172). The unit generally forms on hillslopes from a parent material of volcanic ash and loess over glaciomarine deposits. The soil is described as follows:

- Typical profile consists of ashy silt loam to 16 inches and loam to 60 inches.
- The natural drainage class of the soils is moderately well drained.
- Depth to seasonal high-water table (perched) is listed as about 18 to 36 inches. Restrictive feature is more than 80 inches.
- The unit is assigned to Hydrologic Soil Group C (moderately high runoff potential).
- Transmission capacity varies from low to moderate in upper soils, and typically low to nil in underlying deposits.

As described below, the findings of our explorations are generally consistent with regional unit mapping of glacial drift soil deposits. Fine-grained glacial deposits were found at shallow depths. Shallow soils likely consist of overburden and weathered/altered deviations of the underlying glaciomarine drift (GMD). Highly competent glacial drift soils were observed below the thin upper deposits at this site.

Subsurface Conditions

Subsurface conditions were interpreted and documented by an Element Geologist during the March, 12, 2024 field visit. Three (3) machine-excavated test pits were completed to evaluate the in-situ character of shallow soil conditions within the direct vicinity of the proposed footprint.

Excavations were extended as possible and terminated between 6.3 and 7.0 feet below present grade (BPG) at excavation equipment limits on refusal conditions (hard soils). Test pit TP-1 was conducted to the north of proposed development footprint, along the northern boundary of the site. Test pit TP-2 was conducted to the south of the proposed footprint. Test pit TP-3 was conducted to the east of the proposed footprint, near the western creek slope crest.

Subsurface conditions were interpreted and documented during test pit excavation by an Element Solutions Geologist. Soil classifications were determined per the Unified Soil Classification System (USCS – ASTM D-2487), and all stratigraphic depths and typical soil unit characteristics were recorded. Soil moisture contents, seepages, and evidence of seasonal groundwater presence of fluctuations such as mottling and oxidation were also recorded. Samples of representative soils were collected in sealed plastic bags for further examination and laboratory testing if necessary. Please refer to Figure 2, Appendix I, for a map depicting the testing locations. A photo array from the field explorations (Exhibit A) and complete test pit logs are also attached in the Appendix.

Subsurface Soil Conditions

The soil stratigraphy observed in the test pits was broadly consistent throughout the study area. Topsoil ranged from 0.8 to 1.5 feet thick, and was composed of organic-rich sandy silt with some small gravel. This layer was generally soft, moist to wet, dark brown in color, containing some charcoal, and small roots. Local fill deposits were observed overlying the topsoil in TP-1 to a depth of approximately 0.5 feet BPG. The fill consisted of gravelly sand with broken concrete slab pieces of variable size, and was generally dark brown, loose, containing some organics and grass sod. The fill is interpreted to be from past site development and demolition.

The shallow native soils below the topsoil horizon and local fill deposits consisted of sandy clay, interpreted as weathered glaciomarine drift. This layer was generally gray to tan-brown with moderate to strong mottling common throughout the entire layer, moist to very moist during field work, and medium stiff to stiff. Approximately 60% fines content was observed throughout the layer. The upper soil was not over-consolidated or cemented, and extended typically to 3.0 to 4.0 feet BPG. We interpret this unit to be weathered/altered soil derived from or related to the underlying glacial drift. Stiffness generally increased with depth, with stiff soils established by about 2.0 feet BPG on average.

Below the upper profile, lightly to well cemented sandy silt to sandy clay with some to minor gravel deposits representative of glacial drift was encountered at all locations. Depth to the unweathered drift soils was around 3.0 feet BPG at TP-2 and TP-3 to the south and east of the building zone, and as deep as about 4.0 feet BPG at the north end of the site where the surface is capped by fill remains of past development. These soils were generally gray to gray-brown with light oxidation staining observed locally to 4.0 to 4.5 feet BPG, dissipating with depth. The drift deposits were generally very stiff to hard and increasingly cemented with depth, and continued through termination depth of all test pits. Similar soils were observed locally where exposed within the flow channel of Fever Creek.

Shallow Soil Density Records

One DCP test was performed within the study area, between test pit TP-1 and TP-2 in the proposed building area. Results were used to correlate soil density/consistency with test pit observations, and to assist in determining soil strength and bearing capacity. DCP testing was performed using a DGSI S-200 model Dynamic Cone Penetrometer. The modified unit utilizes a sliding drop-hammer to drive a calibrated 1.5-inch diameter metal cone with extension rods into the ground. The cone is advanced by dropping a 15-pound weight (hammer) a height of 20 inches to strike the rods, and blow counts are recorded by depth in 3.5-inch intervals. Following field work, blow per section are converted to approximate SPT N-values and corresponding soil density/consistency based on published method empirical correlations, and compiled graphically into logs. The DCP test logs are attached in the Appendix.

DCP-1 recorded generally very loose/soft soils to approximately 1.0 feet BPG. This limit corresponds to depth of the loose/soft topsoil and marks the transition to fine-grained clay deposits. The upper 1.0 feet of the clay layer was medium stiff to stiff (down to about 2.0 feet BPG) with progressively very stiff to hard deposits extending to 3.2 feet (practical termination depth). The results of the DCP correlated with the observed stiff clay soils overlying very stiff to hard, cemented sandy silt to sandy clay deposits at depth.

Groundwater & Surface Water Conditions

Weather and site conditions were wet during the March 12, 2024 field investigation. Intermittent rainfall occurred on the day of field work, with light, intermittent rain events in the week leading up to field work. The conditions observed are interpreted to be indicative of early spring season conditions, which may be reduced from peak wet season levels but likely still near sustained winter conditions.

No surface ponding was observed at the subject parcel at the time of field work. Shallow water seepage was observed within the topsoil overlying the shallow fine-grained clay-rich deposits, and along the lower soil boundary of the cover soils over the clay unit. The seepage was observed in all test pits between 1.1 to 1.6 feet BPG. Moderate to strong mottling was common in the upper fine-grained clay-rich glaciomarine drift soils, along with high moisture soil conditions ranging from moist to wet locally. Dry and cemented glacial drift soils were observed with depth throughout the study area which had little to no signs of water transmission and were generally damp to dry. No regional water table was contacted by end depth.

We interpret that transient seasonal groundwater, storm-related runoff, and/or soil saturations likely perches on the shallow fine-grained clay units as well as above glacial drift at depth. Free water likely passes down-gradient through the site, migrating south-southeast towards the existing creek feature lining the site to the east and south. We expect some saturation of shallow soils occurs locally in the winter as a result of transient meteoric waters and surficial collection of rainfall. A pervasive perched water table is unlikely to develop at this site due to the gradually sloping grade and adjacent creek channel.

Groundwater and moisture conditions are expected to vary by season. Moisture observations noted on logs are valid only for the date of exploration. The current scope of work did not include characterization of wet season conditions or establishment and monitoring of groundwater wells. Discussion of seasonal variations not observed directly are interpretive and not intended as statements of fact.

Slope Conditions & Geohazard Designations

Local Code and Designations

City of Bellingham Municipal Code 16.55.420 defines geologic hazards relating to steep slopes as "erosion Hazards Areas (EHAs)" and "Landslide Hazard Areas (LHAs)". EHAs are described as areas prone to soil erosion, specifically, areas where the soil type is comprised of sand, clay, silt, and/or organic matter with slopes greater than 30 percent. From a geological perspective, we also extend EHAs to include those areas prone to erosion or interpreted risk of future erosion along evolving features, such as stream banks. Per CityIQ mapping (accessed online) and our GIS analysis***, the subject stream banks within the site are code-defined as an Erosion Hazard Area with grades typically over 30% and potential for creek erosion***.

LHAs are defined as areas prone to landslides and/or subsidence including slopes with grades equal to or greater than 40 percent grades with a vertical elevation change of at least 10 feet. Per CityIQ mapping and our GIS analysis, ***there are no slope areas within the site which exceed 40 percent grades for at least 10 feet in height***. There are also no known present or historical features that we would characterize as slope failures or acute instabilities surpassing what may be considered background bank erosion potential. Thus, we confirm the site is free of LHA features.

The creek feature and its associated bank slopes were observed in field for determining the extent of potential erosion hazards in relation to the proposed development. EHAs do not have prescribed buffer requirements, but in cases where erosion could occur progressively, development setbacks may apply as discussed at the end of this section.

Visual Assessment of Slopes & Stability

During the field visit, an ES geologist performed a detailed visual reconnaissance of the sloping areas of the site along the creek feature to assess for the presence of geologic hazards and erosional conditions. Reconnaissance findings of slope stability indicators pertinent to the proposed development are summarized below. Photos of representative conditions are attached in Exhibit A.

The eastern and southern border areas of the site are characterized by densely vegetated creek ravine slopes ranging from gentle to steep grades. The bank face on the project side is generally well vegetated with blackberry bushes and various brush, with spaced mature trees along the crest and further upland from the creek. Steeper grades generally line the northeast and southwest corners of the site along the creek channel. Bank grades generally decrease to the east along the southern boundary, and to the south along the eastern boundary, with grades less than 30 percent along the southeast margin of the parcel.

We found little to no exposed areas where active erosion of soils was occurring within the parcel boundaries. Where soil was exposed, the bank area did not display active soil wasting nor soil debris along the base, indicating the exposed soil's weathering/erosion is a slow process. Surface conditions appeared well stabilized within the parcel boundaries. The mature trees within the site display generally straight trunks aside from one tree along the southwest corner. The smaller trees along the northeastern slopes are commonly curved with some tilted and fallen trees along the base of the slope. The orientation of the younger trees appears to be related to growth patterns as opposed to ground instability.

The south and southeastern bank slopes, located off-site on the opposite sides of the creek, appear to display some minor evidence of weathering and erosion. The base of the slope within this area has existing support lagging, presumably to minimize the risk of erosion to adjacent properties. Various stormwater pipes were also observed along the southern slopes, releasing directly into the stream channel, with no evidence of erosion from the presence or continued operation of the outlet pipes. These observations on the far side of the creek at the southeast corner area are consistent with a higher erosion potential for the outer bend, or "cut bank" side, of the creek as it takes a sharp turn from south to west.

In conclusion, the steep banks found locally on both sides of the creek are interpreted to be from lateral erosion and creek downward incision occurring over a long period of time. No activity related to slope instability or active erosion was noted along the banks within the site in the present day. The majority of the slope is well vegetated with blackberry brush and small trees with some mature trees dispersed. The banks appear broadly stabilized with the existing grades and vegetative cover, with no acute present-day erosional features observed.

Given the hard clay soils underlying the site and observed condition of the creek banks, ***we interpret a generally low to very low erosion risk from the creek to the proposed development area***. Average creek bank erosion rates on the order of one inch per year may be anticipated on the project side of the channel. This estimate assumes flow conditions in the creek do not change significantly, and with a typical level of site and vegetation management by property owners, as well as avoiding direct impacts to the creek banks (e.g. from disturbance or vegetation clear-cutting) during construction or in the future.

Summary of Key Findings & Geohazard Determination

Our study included the review of readily available maps and documents, GIS analysis of LiDAR topography and local grades, and site and slope visual reconnaissance. Based on the results presented above, we have drawn the following key interpretations and conclusions.

- The general lack of surficial expressions of instability or geomorphic features indicating historical slope failure, along with the local geology, suggests a very low probability for future slope instability and acute erosional issues in the project area.
- The creek banks within the site are internally comprised of very stiff to hard glacial drift deposits, also observed with depth in our explorations. The soils are interpreted to be overall conducive to supporting steep to moderate native slopes with minimal erosion potential for the small creek.
- The stream bank slopes within the site typically have grades between 30 and 60 percent and heights less than 8 feet. Therefore, the site does meet the definition of an Erosion Hazard Area per COB code, but does not qualify as a Landslide Hazard Area.

We conclude that the vegetated creek bank slopes along the eastern and southern margins of the site do meet the definition of an "Erosion Hazard Area" per COB code. It is our opinion that the hazard of erosion and slope instability at the project location is generally low to very low. As such, the slope conditions do not present an unacceptable level of risk for site development in general if a suitable approach is taken from a geotechnical perspective with respect to locations of construction. Nor does the planned development present a risk of geologic hazards to the site or adjacent areas, assuming proper design and construction methods are applied.

Setback Discussion

Regardless of a slope feature's code-defined designation as a geologic hazard for erosion and/or landslide potential, structure construction with respect to slopes must be considered from a geotechnical perspective. We understand the minimum building setback from the creek channel (OHWM) will be 25 feet, if a creek buffer reduction is granted (review in progress). We have considered the adequacy of the 25-foot creek setback, or need for an additional slope setback, with respect to geologic hazards.

A minimum setback is not typically required from designated EHAs based on grade, although setbacks may be appropriate where cumulative erosion is a concern. For small slopes in City of Bellingham jurisdiction identified as LHAs, minimum structural setbacks of 10 feet from the slope crest are required by code without encroaching and addressing "alterations". While the low bank slopes are not defined as LHAs, this minimum setback requirement is a useful consideration.

The rough order of magnitude erosion potential interpreted for creek banks at this site, extrapolated over a 100-year design life, is on the order of 8 to 10 feet. By our interpretation of cumulative bank erosion potential, there would be sufficient building protection from the creek bank over the design life if a 25-foot minimum creek channel OHWM setback (or greater) is adopted. Therefore, an additional geohazard setback is not considered necessary past the minimum setback required for a creek buffer.

For the sake of clarity and physical reference, ***we recommend a minimum structural setback of 20 feet from the creek bank slope crest to new foundations***. This assumes building foundations are constructed as recommended below, and incorporates the interpreted bank erosion potential over the design life of the project. The 20-foot setback line from mapped top of bank is illustrated on Figure 3 along with the approximate proposed building vicinity which we understand will meet or exceed this setback.

For protection of the creek bank from any landward impacts, we also recommend a minimum 10-foot non-disturbance buffer be maintained from the crest of the steep bank during and after construction. Within this buffer, existing vegetation should be retained, and, if impacted incidentally in construction, replaced with new plantings of equal or better coverage. Additional planting to fully establish and maintain vegetation coverage in the buffer is encouraged. General recommendations are provided below which are intended to limit site disturbance during and after construction, and to assist in ground stabilization and long-term care of the site in proximity to the creek bank.

Stormwater Feasibility Commentary

For on-site infiltration to be feasible as a stormwater management strategy for residential development, the subsurface profile must have a combination of suitably transmissive soils and adequate separation to seasonal high groundwater or restrictive layers. Common criteria adopted for single-family residential purposes call for at least 3.0 feet of permeable soils for infiltration systems to be feasible. For residential use, small systems and pervious surfacing must be able to maintain at least 1.0 feet of infiltrating soil and separation above restrictive soil/rock horizons or seasonal high groundwater levels (2019 SWMMWW). The site is also within the City of Bellingham jurisdiction, which requires that at least 3.0 feet of permeable soils, and at least 1.0 feet of separation, must be available for residential downspout infiltration systems to be feasible (per published feasibility criteria).

Our explorations have found multiple limiting factors present at the site that are not conducive to residential downspout infiltration or pervious surfacing per typical guidelines:

- First, the depth to fine-grained clay-rich soils interpreted as partially to fully restrictive to water transmission was between 1.1 feet to 1.5 feet BPG. Underlying the upper clay-rich weathered GMD soils to termination depths is lightly cemented, fine-grained unweathered glacial drift deposits which appear to be fully restrictive to water transmission.
- Second, upper weathered glaciomarine drift soils contained moderate to strong oxidation mottling which is evidence of periodic saturation interpreted to be from perched and/or transient water migrating through the site along the fully restrictive horizon. The shallow subsurface was also locally moist to very moist during our early spring field work, and we observed seepage along the base of the topsoil/cover soils.

Thus, we interpret that the site is not feasible for an infiltration design approach to stormwater disposal due to the limiting factors present and according to typical residential stormwater guidelines. The recommended approach to stormwater management includes the controlled collection and conveyance of stormwater to a local COB catch basin or directly to the creek along the eastern and southern margins of the site, as discussed below.

Stormwater & Drainage Control Recommendations

Development drainage features and stormwater controls should be installed so that they do not lead to an increased potential for erosion of the site upland or nearby creek slopes. Based on the findings of this study, we conclude and recommend the following criteria for proper management of new stormwater generated by the development.

- Based on the combination of unfavorable factors within the site, including shallow restrictive glacial clay soils and seasonal perched water, it is our opinion that ***on-site infiltration is broadly infeasible***. Direct release of roof water onto the site surface is also not advised due to the limited space and potential for runoff to impact nearby creek banks.
- From both a practical and geotechnical perspective, nearby public stormwater utilities or open conveyances typically present a good option for safe disposal within densely developed areas. Stormwater should be conveyed via tightline from the building area to a preferred utility connection or open conveyance outlet point.
- In the case of this site, the closest stormwater facility is the Fever Creek channel, which acts as a conveyance and is open through the site area between culverts along Alabama Street and Xenia Street. During reconnaissance, we also noted other nearby sites have drain pipes with outlets into the creek channel. Furthermore, nearby catch basins are mapped to drain into the creek. We recommend the stormwater from the project site can be directed into the creek in lieu of connecting to a separate nearby utility, as the fate of the water will be the same (assuming local agency approval).

Stormwater control / conveyance general recommendations for protection of slopes are as follows:

- As applicable, stormwater from roof runoff, foundation and wall drains, and exterior surface or yard drains should be tightlined from their collection points to a catch basin in the vicinity of the structure. Stormwater should travel down-gradient via a primary conveyance pipe to an approved release location.
- Foundation and wall drains should be conveyed separately from other sources, or adjoined at a suitably down-gradient location, to prevent the backflow of water into these drains.
- For concentrated flow outfalls, appropriate energy-reducing features should be used at release point to minimize erosion. Examples include a perforated T-stub/spreader pipe or rock pad.
- Any above-grade tightlines, if utilized, should be composed of sturdy rigid material (i.e. PVC or welded HDPE pipe), sized adequately for anticipated volume, and anchored sufficiently to minimize potential for damage and failure. Exposed tightlines should be inspected periodically by the owner, and repaired or replaced as needed to maintain a safe working condition.

Geotechnical Recommendations

The below recommendations address specific geotechnical aspects for the project design and construction that will be key to successful completion with respect to conditions encountered. These recommendations are also site- and project-specific based on our understanding of the proposed development options at this time. If modifications or additions are made to the plans, or if the style or location of the construction is changed, Element should be contacted to review the changes and provide revised or additional geotechnical recommendations as needed. Element Solutions may be contacted to provide a further scope of work or consultation in support of the design process and construction.

We generally understand that the client plans to develop one or more small, low-story residential structures. Development options are still under consideration at this time, but will be similar in aerial extent and location along the northwest quadrant of the parcel. Full building plans have not been provided at this time. It is assumed that structures will be constructed of wood framing with typical shallow perimeter concrete footings and light loads. No daylight basement areas or retaining wall-style foundation elements will be employed. Loads and construction practices are assumed to be typical for the type and style of construction.

Foundation Design and Construction:

- <u>Bearing Strata</u> – Throughout the site, native deposits of glacial origin suitable for bearing residential foundation loads was encountered in all explorations below topsoil and shallow low strength subsoils. To ensure long-term foundation base support with minimized settlement risk, ***we recommend that foundations be placed either directly on stiff to very stiff native glacial drift soils, or on well-compacted dense structural fill placed over stiff or better native conditions*****.**
- <u>Anticipated Bearing Excavation Depths</u> - Per our test pit and DCP test results, ***minimum depth to non-organic soils is typically between 1.1 to 1.5 feet BPG, varying locally***. However, soils directly underlying topsoil are commonly low-strength (medium stiff), and generally become more stiff with depth. The depth to optimal bearing conditions (unweathered glacial drift soils) is typically 3.0 to 4.0 feet BPG. Stiff glaciomarine drift soils were generally observed by 2.0 feet BPG.

  - The consistency of observations between all test pits provides confidence in this interpretation of excavation needed. *However, some variability should be anticipated as is common for natural soil deposits.*

- Bearing Capacity – Foundations shall be founded either on (1) well-compacted structural fill placed and compacted over suitable native soils, or (2) stiff or better glacial clay soils encountered with depth.
  - For shallow footings placement over stiff native soils with minimum excavation (approximately 2 feet), we recommend ***an allowable vertical bearing capacity of up to 2,000 pounds per square foot (psf)*** with preparations as recommended herein.
  - For greater embedment scenarios resulting in placement of footings directly on generally dense / hard unweathered glacial drift subgrade (below about 3.0 to 4.0 feet BPG), the allowable bearing capacity may be increased to a maximum of 3,000 psf.
  - Structural fill is allowed to be placed below footing locations if properly compacted to a uniformly dense condition. For this project, we recommend limiting the use of fill to only as necessary to level and/or backfill over-excavated areas and variations in subgrade elevations, or as a planned base lift below footings. Construction of an elevated fill pad is not expected and generally discouraged.
  - Where structural fill is incorporated, the corresponding bearing capacity for underlying native soils should be maintained (i.e. 2000 psf over shallow soils, 3000 psf over deeper hard subgrade).
  - The allowable bearing value can be increased by up to 1/3 for short-term transient loading such as due to seismic or wind loads.
- Minimum Footing Depth – To adequately protect the SFR foundation from the effects of freeze-thaw amid shallow perched water concerns and moisture-sensitive soils, we recommend a minimum footing depth of 18 inches below final exterior grade. Additionally, footing placement depth should follow the recommendations above for adequate bearing.
- Minimum Footing Dimensions – Foundations shall be sized sufficiently to meet either the maximum allowable bearing load requirements, or minimum size required per IBC, whichever is larger. Bearing capacities above assume minimum width dimensions typical for residential structures and/or small multi-family townhouse style structures.
- Expected Settlement – Expected settlements will be largely elastic and well within structural tolerances for the proposed building type, provided footing bearing surfaces are carefully prepared and not disturbed or properly recompacted. Settlements should not exceed 1-inch total, nor ½-inch differential over 50 lineal feet.
- Sliding Friction – Sliding resistance between a concrete footing and subgrade contributes to total lateral resistance. We recommend:
  - ***A friction coefficient of 0.35*** for placement either on very stiff to hard native glacial drift soils at depth, or on a properly compacted structural fill base (includes a factor of safety of at least 1.5).
  - If footings are placed shallowly on weathered clay soils, contribution of sliding friction to total lateral resistance should be negated. If friction resistance is a required factor of structure design, we recommend shallow footings be placed over a structural fill base layer.

- Passive Resistance – For passive resistance associated with foundation backfill, we recommend:
  - A ***maximum passive lateral bearing value of 250 pcf (equivalent fluid weight)***. This assumes either the placement of footings directly against very stiff to hard glacial soils at depth, or backfill of shallow foundations with structural fill compacted as recommended below for at least 3 feet horizontal distance. A factor of safety of about 2 or more is incorporated. The contribution of the upper 12 inches to lateral resistance should be ignored, unless paved.
  - For any foundations placed “neat” against native shallow soils, or for cases of minimal backfill, the passive lateral pressure should be reduced accordingly. ***A general value of 100 pcf (EFW) is recommended for broad use, which takes into account the range of possible conditions***.
- Foundation Backfill- Foundation backfills shall be comprised of structural fill consisting of a granular, non-plastic aggregate of suitable gradation, relatively uniform in composition, and containing no discernable organics or other deleterious material. The maximum particle size of about 4 inches is advised. We recommend that imported material meet WSDOT SS 9-03.14(1) – Gravel Borrow specification, or similar material.
  - Foundation/over-excavation backfill and key backfill shall be compacted as recommended in the site preparation & grading section below to achieve optimal performance and meet design requirements.
- Foundation Drainage - The site exhibits shallow restrictive glacial soils, and may be susceptible to shallow water migration and perched water conditions in winter months. Embedded foundations will impede transient water flow and may exacerbate perched water build-up. We highly recommend use of perimeter foundation drains to promote long-term dry foundation conditions and avoid build-up of water against the structure.
  - In addition to foundation drains, we recommend exterior ground surfaces and pavements be graded to slope away from the structure. Building ancillary features should avoid those that could allow water to collect and pond against the outside of the structure. Exterior pavements and flatworks near the structure should incorporate local surface drains to control runoff.
  - For greatest effectiveness, footing drains should be placed even with the base of the footing along the full exterior of the structure. A continuous, 4-inch minimum diameter, perforated pipe that is sloped for gravity-assisted drainage and wrapped in filtration fabric or a filter sock is recommended. The area around the pipe and extending against the adjacent foundation wall should be backfilled with drain rock and separated from adjacent soils by use of soil separation fabric. Unless otherwise specified by design, the upper 1.0 foot of subsurface should be capped by low permeability fill material or pavement to minimize vertical water transmission from the building exterior to the foundation.
  - Connect footing drains via tight-line to a catch basin or discharge facility separately from roof drains and other exterior surface drains to avoid backwards transmission or flooding of the foundation drain system by stormwater sources.

Foundation and Site Preparations & Grading:

- **Foundations:** Strip all cover soils, organic-rich deposits, uncontrolled fills, apparently disturbed native soils, and unsuitably loose or soft native soils from foundation locations prior to final benching and recompacting preparations for footing construction. All foundations should bear directly on suitably dense, well-compacted structural fill or on suitably stiff or better native soils (as detailed above). Enlist a geotechnical professional to verify foundation subgrades by a combination of visual assessment and T-probing to confirm suitability.

- **Slab-on-Grade:** Below any floor and garage slab areas, strip all organic soils, unsuitable fills, and loose/soft subgrades prior to placing base fills. Recompact granular subgrades with suitably sized, vibratory equipment for the purpose (i.e. a plate compactor for interior slabs, small drum roller for garage and exterior pavement slab areas). Perform visual/T-probe inspection to assess for yielding or pumping areas under the observation of a geotechnical professional. Replace problematic areas by over-excavating and backfilling with structural fill. For sloping subgrades over about 5:1 (H:V) grade, incorporate regular benching to establish a level surface for fill placement.
- All fills placed below or as backfill for foundations are considered structural fill. Materials should consist of a granular, non-plastic aggregate of suitable gradation, relatively uniform in composition, and containing no discernable organics or other deleterious material. A maximum particle size of about 4 inches is advised. For broad use, we recommend that imported material meet WSDOT SS 9-03.14(1) – Gravel Borrow, or similar material.
- Place and compact structural fills in accordance with typical industry standards. A minimum compaction standard of 95% is recommended for all structural purposes (based on the material's optimum dry density as determined by ASTM D-1557 laboratory testing). Fills should be placed in 8- to 10-inch loose lifts, compacted with appropriate equipment, and field compaction tested on a regular basis as needed to verify the suitable installation of each lift.

Critical Area Development Recommendations

The following recommendations are intended to help protect sensitive areas from degradation, and to minimize the inherent risks associated with development in the vicinity of a geologically sensitive critical area. They are site- and project-specific based on our understanding of the proposed development at this time.

Critical Area Site Management During Construction:

- Placement of new fills on or above steep slope areas should be avoided. Fills placed on a slope face outside of the confines of a structure add weight to the slope, may increase risk of instability or erosion, and are generally discouraged. It is our understanding that no filling/grading or other disturbance is proposed on or in close proximity to the creek bank erosion hazard areas.
- We have recommended a minimum 10-foot non-disturbance buffer from the creek bank slopes which shall apply to both construction phase activities and future site use. Vegetation maintenance is allowed and enhancement planting is encouraged within this buffer. Significant clearing of existing vegetation on the slopes or within the buffer should not be done without review and involvement of qualified professionals.
- Temporary stockpiling of excavated material or fills, or storage of heavy construction materials and machinery, shall be avoided on or above sloping areas. We recommend minimizing the amount and duration of materials stored by exporting/importing along with site preparation work. Materials should be stored within the northwest part of the site, away from the steep creek slopes, pending transport or use.
- Construction practices shall take care to disturb or impact as little area as possible outside of the building locations, and especially within the recommended slope setback zone. Impacted areas should be restored with top-dressing and appropriate seeding or plantings for the environment following construction. Avoid all vegetation and ground disturbance outside of the established development boundaries and within defined critical areas.

- Temporary erosion controls:
  - Systems and procedures should be put into place as appropriate for the site, project, and timeframe/season of construction. TESC measures should include clear slopeward clearing/disturbance limit barriers or demarcations.
  - During periods of major excavation and during benching of subgrade for foundation installs, additional safeguards should be installed as needed to prevent soil debris and sediment from leaving the project area / site. In particular, protect the creek areas from any impacts associated with earthwork. A sturdy physical barrier is advised to be employed for creek protection during site preparations.
  - The contractor is responsible for implementing and maintaining TESC throughout earthwork activities, and for working within accepted project limits to avoid unnecessary impacts to adjacent areas (especially critical areas).

Long-term Erosion Control and Maintenance:

- We recommend goals of low impact or vegetative enhancement be adopted for exterior areas outside the building and driveway/parking areas. We advise planting of appropriate vegetation and/or reseeding among the ancillary areas near improvements that are disturbed during construction, either at the end of construction or in the near future. This will help reestablish surface stability and minimize future erosion risk.
- Any areas outside of established clearing limits accidentally disturbed during construction must be restored promptly, and at latest at the end of earthwork activities. If mature vegetation is impacted or destroyed, replace in kind with new native plantings.
- Promoting future growth of strong-rooting brushy plants and new trees is encouraged both following construction and in the long term for areas of the site upland near the creek banks. Thick and healthy vegetation will assist in retaining cover soils, increase the hydrologic resistance of the surface conditions, and thus lessen the risk of erosion that could result from incidental surface runoff or other overland drainage issues.
- Major landscaping alterations should be avoided outside of the planned development area unless properly reviewed by a geotechnical professional and found to be suitable for the location and surrounding conditions, including with respect to potential geohazard areas. No alterations to steep slopes or proximal areas should be done without prior review and approval.
- If conditions are observed to evolve or deteriorate in the future and pose a potential concern for stability of the site or adjacent areas, we recommend conditions be re-observed at that time. Element Solutions should be contacted to reassess the site conditions, and can provide guidance for stabilization and best management practices at request of the property owner.

Assumptions and Limitations

The depth and extent of explorations for the limited geotechnical evaluation performed in this study was limited by reasonable feasibility constraints, available time and site access during field work, and the equipment utilized. Exploration findings presented in this report represent the locations and dates of field work. Conditions may not be fully representative of areas of the site not explored, or other times of the year. A typical degree of natural variation should be anticipated for native subsurface conditions; greater variation is likely where previously altered conditions or uncontrolled fills are encountered. If conditions are found in construction that differ from those documented, Element Solutions should be contacted to provide additional review and consultation, and to reevaluate our recommendations if necessary.

No in-field Pilot Infiltration Test (PIT) was conducted during the assessment. No laboratory work has been completed at this time. The conclusions are based on visual classification methods and visual-based interpretations of conditions as observed in the explorations, as well as our past project experience with similar soils and conditions as found at this site. At request of the client, Element can conduct additional site assessment or analysis services if required for final design.

The scope of work has not included conclusive measurement of groundwater conditions, observation of temporal or seasonal fluctuations, or establishment of monitoring wells. The inferences made regarding seasonal groundwater changes shall be understood as interpretive in nature, not statements of direct observation or fact.

We recommend a review of final project plans by Element Solutions to ensure that the intent of the recommendations provided is followed in design prior to the start of construction. If changes are made to the project scope that could impact the intent or applicability of the recommendations, Element Solutions should be contacted for additional review.

The client shall understand that the project is located within proximity to a designated geologically hazardous critical area, and has elected to develop/reside at this location. Findings of this report are not intended to provide a guarantee or warranty of future site conditions which may change as a result of natural processes as well as surrounding influences. We have conducted this work in accordance with typical industry standards for geologic hazard assessment, and provided recommendations intended to minimize but not necessarily eliminate risk as possible from a geotechnical perspective. However, it is not feasible to fully anticipate all potential future risks or evolution of site conditions that may occur. It is the client's choice to pursue the project upon review of this report and acceptance of its findings. The client shall accept that there are inherent risks associated with geologically active areas, and assume sole responsibility for its future consequences, both as detailed herein and unknown. Element Solutions, its staff and owners, shall be indemnified and held harmless from the consequences of development and residence in proximity to a geologically hazardous area.

References

Lapen, T.J., Geologic Map of the Bellingham 1:100,000 Quadrangle, Washington. Washington State Department of Natural Resources, Division of Geology and Earth Resources Open File Report 2000-5, December 2000.

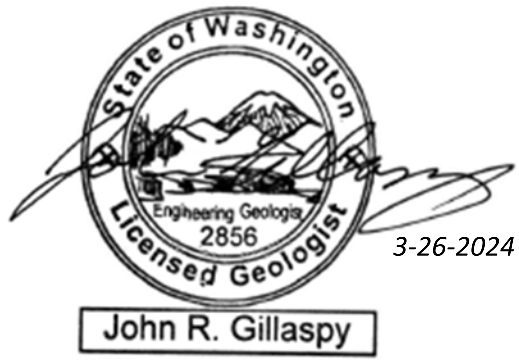
Natural Resources Conservation Service, Web Soil Survey, U.S. Department of Agriculture. Accessed online at: http://websoilsurvey.nrcs.usda.gov/app/WebSoilSurvey.aspx

Washington State Department of Ecology, *Stormwater Management Manual for Western Washington.* Publication No. 19-10-021. July 2019.

Closure

Thank you for the opportunity to contribute our expertise to your project. Please feel free to contact us at (360) 671-9172 if you have any questions or comments regarding this report.

Sincerely,



3-26-2024

Joely Marsyla, B.A.
Staff Geologist

John Gillaspy, M.S., LEG
Environmental Services Manager

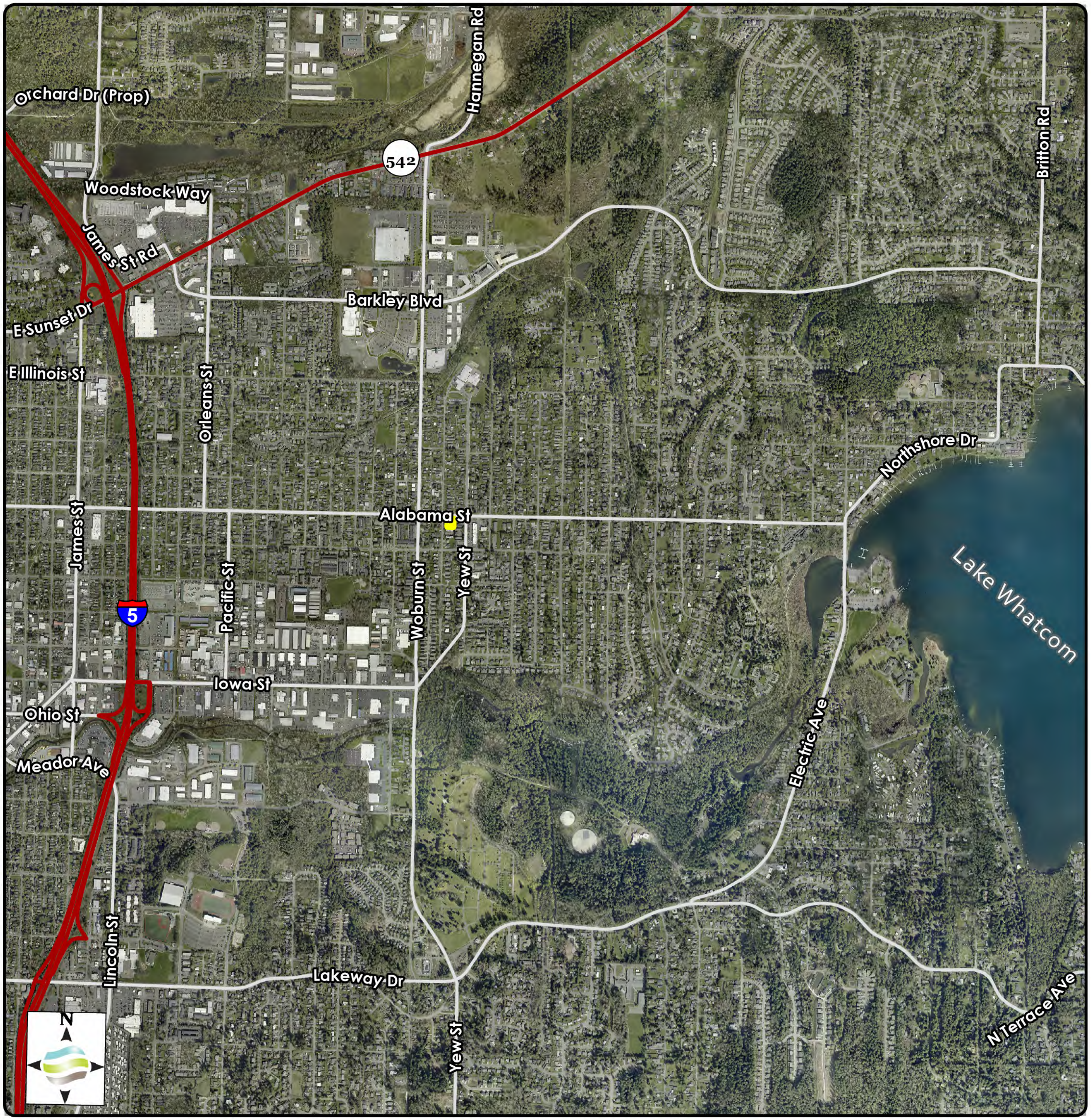
Statement of Limitations

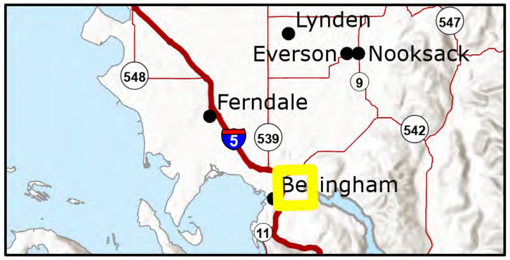
This document has been prepared by Element Solutions for exclusive use and benefit of the client. No other party is entitled to rely on any of the conclusions, data, opinions, or other information contained herein. This document represents Element Solution's best professional judgment based on the information available at the time of its completion and as appropriate for the project scope of work. Services performed in developing the content of this document have been conducted in a manner consistent with that level and skill ordinarily exercised by members of the geologic engineering profession currently practicing under similar conditions. No warranty, expressed or implied, is made.

If the client elects to retain another consultant to continue work on the project in a similar capacity, that firm or individual must be responsible for fully reviewing this report and any associated documents. They shall either accept responsibility for the findings and implementing the recommendations presented in this report, or shall offer their own conclusions and recommendations superseding those of Element Solutions as they see fit. In no way will Element Solutions be held responsible for misapplication or disregard of our recommendations by the client, contractors, or other consultants. Element Solutions is not responsible for misuse or misunderstanding of our recommendations, and recommends that we be contacted in the event that clarification or guidance is needed. Non-compliance of these stipulations or to the recommendations in this report will release Element Solutions from any associated liability.

Appendix

1) Figure 1 –1:24,000 Site Vicinity Map
2) Figure 2 –Site Map Aerial Photo with Test Locations
3) Figure 3 – LiDAR Percent Slope Map with Test Locations
4) Test Pit Logs: TP-1 to TP-3 (March 12, 2024)
5) Exhibit A – Field Photos (March 7 & 12, 2024)





Data Credits:
[Parcels] Whatcom County 2018
[Roads] COB 2018
[Imagery] Whatcom 2019

Subject Parcel

1:24,000
0 1,000 2,000 4,000 Feet

909 Squalicum Way, Ste 111
Bellingham, WA 98225
info@elementsolutions.org
Phone: 360. 671. 9172

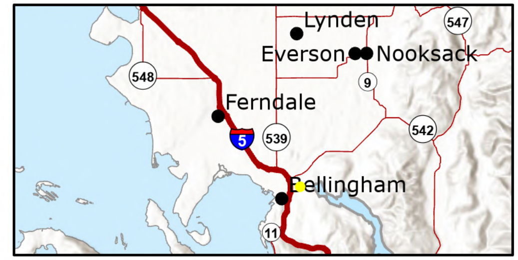
ELEMENT solutions

Figure 1

Stormwater Feasibility Assessment
2302 Alabama Street, Bellingham, WA
Site Vicinity Map

Date: 3/26/2024

P:\Pse Project\2024058\ENVRMNT\GIS\Alabama Master Map.aprx



Data Credits:
[Parcels] Whatcom County 2018
[Roads] COB 2018
[Imagery] Whatcom 2019

Subject Parcel
Test Pits
DCP
5ft Contour
2ft Contour
Fever Creek

1:300
Feet
0 12.5 25 50

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Figure 2

Stormwater Feasibility Assessment
2302 Alabama Street, Bellingham, WA
Aerial Site Map

Date: 3/26/2024

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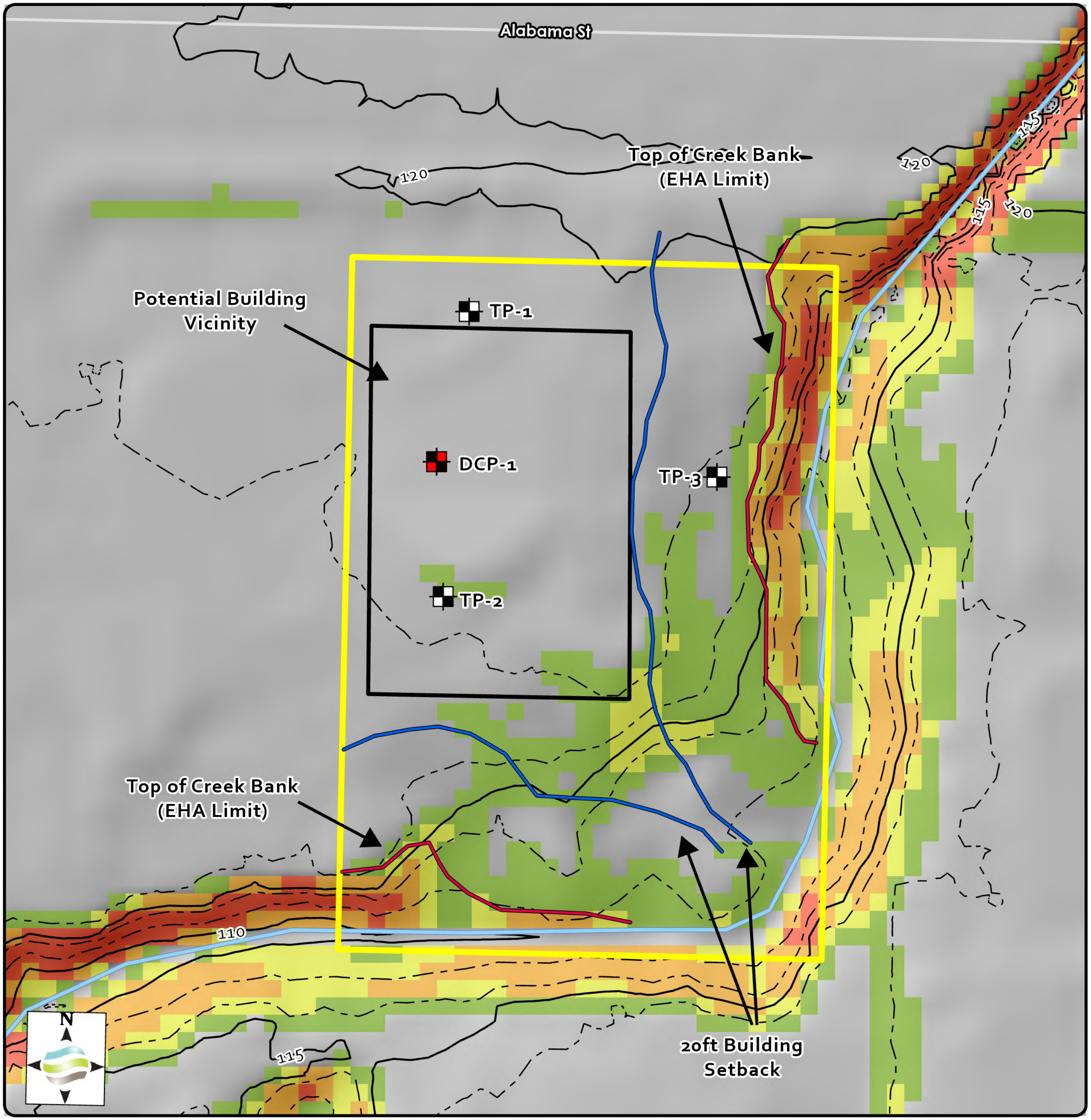

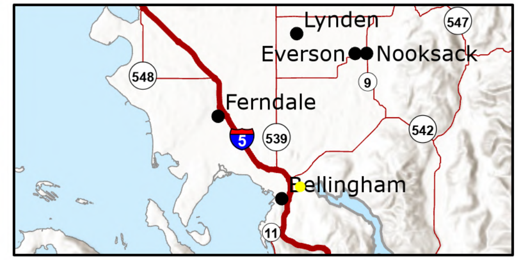

Data Credits:
[Parcels] Whatcom County 2018
[Roads] COB 2018
[Imagery] Whatcom 2019

Subject Parcel
Test Pits
DCP
Top of Creek Bank
20ft Building Setback
5ft Contour
2ft Contour
Fever Creek

Percent Slope

Value

Value
0 - 15
15 - 30
30 - 40
40 - 60
60 - 100

1:300

0 12.5 25 50 Feet

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Figure 3

Stormwater Feasibility Assessment
2302 Alabama Street, Bellingham, WA
LiDAR Percent Slope Map

Date: 3/26/2024

P:\Pse Project\2024058\ENVRMNT\GIS\Alabama Master Map.aprx

TEST PIT NUMBER TP-1

ELEMENT
Element Solutions
909 Squalicum Way, Suite 111
Bellingham, WA 98225
Telephone: 360-671-9172

CLIENT Eddie Goodsir	**PROJECT NAME** Stormwater Feasibility Assessment
PROJECT NUMBER 2024058	**PROJECT LOCATION** 2303 Alabama Street, Bellingham, WA
DATE STARTED 3/12/24 **COMPLETED** 3/12/24	**GROUND ELEVATION** **TEST PIT SIZE**
EXCAVATION CONTRACTOR Client	**GROUND WATER LEVELS:**
EXCAVATION METHOD Rubber Tracked Mini Excavator	**AT TIME OF EXCAVATION** ---
LOGGED BY Joely Marsyla **CHECKED BY** John Gillaspy	**AT END OF EXCAVATION** ---
NOTES Light seepage at 1.3 feet BPG. No groundwater observed.	**AFTER EXCAVATION** ---

DEPTH (ft)	SAMPLE TYPE NUMBER	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0.0		SP	0.5	(SP) SAND WITH GRAVEL to GRAVELLY SAND, dark brown, moist, loose, some broken concrete slabs **FILL**
		ML	1.3	(ML) SANDY SILT, organic-rich, some charcoal, dark brown, moist to very moist, soft Buried locally where fill was observed **TOPSOIL** Light seepage at base of layer
2.5	GB GB	SC	4.0	(SC) SANDY CLAY, medium-grained sand, gray to tan brown with moderate to strong mottling, damp to moist, medium stiff to stiff with depth, qu = 2.0 tsf **GLACIOMARINE DRIFT (WEATHERED)** Becomes more stiff with depth
5.0	GB	CL-ML	6.3	(CL-ML) SANDY CLAY to SANDY SILT, some small gravel, gray with light oxidation staining in upper horizon, dry, very stiff to hard with depth **GLACIAL DRIFT** Becomes lightly cemented with depth

Bottom of test pit at 6.3 feet.

Element Solutions
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Bellingham, WA 98225
Telephone: 360-671-9172

TEST PIT NUMBER TP-2

CLIENT Eddie Goodsir
PROJECT NAME Stormwater Feasibility Assessment
PROJECT NUMBER 2024058
PROJECT LOCATION 2303 Alabama Street, Bellingham, WA
DATE STARTED 3/12/24 **COMPLETED** 3/12/24
GROUND ELEVATION **TEST PIT SIZE**
EXCAVATION CONTRACTOR Client
GROUND WATER LEVELS:
EXCAVATION METHOD Rubber Tracked Mini Excavator
AT TIME OF EXCAVATION ---
LOGGED BY Joely Marsyla **CHECKED BY** John Gillaspy
AT END OF EXCAVATION ---
NOTES Light seepage at 1.3 feet BPG. No groundwater observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE TYPE NUMBER	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0.0		ML		(ML) SANDY SILT, some small gravel, organic-rich, some charcoal, dark brown, moist to wet, soft **TOPSOIL** Light seepage at base of layer
			1.5	
2.5	GB	SC		(SC) SANDY CLAY, gray to tan-brown with strong mottling, moist to very moist, medium stiff to stiff with depth, qu = 2.0 tsf **GLACIOMARINE DRIFT (WEATHERED)**
			3.0	
5.0		CL-ML		(CL-ML) SANDY CLAY to SANDY SILT, some small gravel, tan-brown to gray with light oxidation staining to 4.5 feet BPG, dry, very stiff to hard with depth **GLACIAL DRIFT** Becomes lightly cemented with depth
			7.0	

Bottom of test pit at 7.0 feet.

TEST PIT NUMBER TP-3

ELEMENT
Element Solutions
909 Squalicum Way, Suite 111
Bellingham, WA 98225
Telephone: 360-671-9172

CLIENT Eddie Goodsir
PROJECT NAME Stormwater Feasibility Assessment
PROJECT NUMBER 2024058
PROJECT LOCATION 2303 Alabama Street, Bellingham, WA
DATE STARTED 3/12/24 **COMPLETED** 3/12/24
GROUND ELEVATION **TEST PIT SIZE**
EXCAVATION CONTRACTOR Client
GROUND WATER LEVELS:
EXCAVATION METHOD Rubber Tracked Mini Excavator
AT TIME OF EXCAVATION ---
LOGGED BY Joely Marsyla **CHECKED BY** John Gillaspy
AT END OF EXCAVATION ---
NOTES Light seepage at 1.0 feet BPG. No groundwater observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE TYPE NUMBER	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0.0		ML		(ML) SANDY SILT, some small gravel, organic-rich, some charcoal, dark brown, moist to wet, soft **TOPSOIL** Light seepage at base of layer
			1.1	
	GB	SC		(SC) SANDY CLAY, gray with strong mottling, very moist to moist with depth, soft in upper horizon, stiff to very stiff with depth **GLACIOMARINE DRIFT (WEATHERED)**
2.5			3.0	
5.0		CL-ML		(CL-ML) SANDY CLAY to SANDY SILT, some small gravel, trace cobbles, gray with moderate oxidation staining to 4.2 feet BPG, damp to dry with depth, very stiff to hard with depth **GLACIAL DRIFT** Becomes lightly cemented with depth
			7.0	

Bottom of test pit at 7.0 feet.

ELEMENT solutions

DYNAMIC CONE PENETROMETER LOG

DCP-1

Project Name: Stormwater Feasibilty Assessment and Geotechnical Evaluation

Project Number: 2024058

Test Location : Between TP-1 & TP-2

Testing Date: 3/12/2024

Max Depth (ft): 3.2'

DEPTH	BLOWS PER 3-1/2"	N'	GRAPH OF N' VALUE (0 10 20 30 40 50)	TESTED CONSISTENCY COARSE	TESTED CONSISTENCY FINE
0	3	1		Very Loose	Very Soft
	3	1		Very Loose	Very Soft
	7	4		Very Loose	Soft
	17	9		Loose	Stiff
	12	6		Loose	Medium Stiff
	17	9		Loose	Stiff
	22	11		Medium Dense	Stiff
2	40	17		Medium Dense	Very Stiff
	71	24		Medium Dense	Very Stiff
	95	28		Medium Dense	Very Stiff
	150	50		Very Dense	Hard
4					
6					
8					
10					
12					

Depth (ft)

Exhibit A – March 7 & 12, 2024 Field Photos (2302 Alabama Street)

Photo 1: Upland site conditions during recon visit – Looking south.

Photo 2: Site frontage – looking east.

Photo 3: TP-1 conditions. Note mottled GMD layer overlying very stiff/hard glacial drift.

Photo 4: TP-2 conditions. Note seepage along topsoil-GMD boundary.

Photo 5: TP-3 conditions

Photo 6: Fever Creek along NE corner of site – Looking south.

Photo 7: Creek exiting culvert along NE corner of site.

Photo 8: East-southeast slope conditions along creek.

Photo 9: Southeast slope conditions to the south of subject parcel. Note pipes directing to creek.

Photo 10: Southern boundary of site. Grades are generally steeper out of site. Note lagging along southern slope.