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May 17, 2024

Inderjet Dhaliwal
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Attention: Jim Toporek, AIA, Studio 3 Architecture (jim@studio3architecture.com)

Re: **Report of Geotechnical Engineering Services**
Fairview Apartments
2110 Strong Road SE
Salem, Oregon 97302

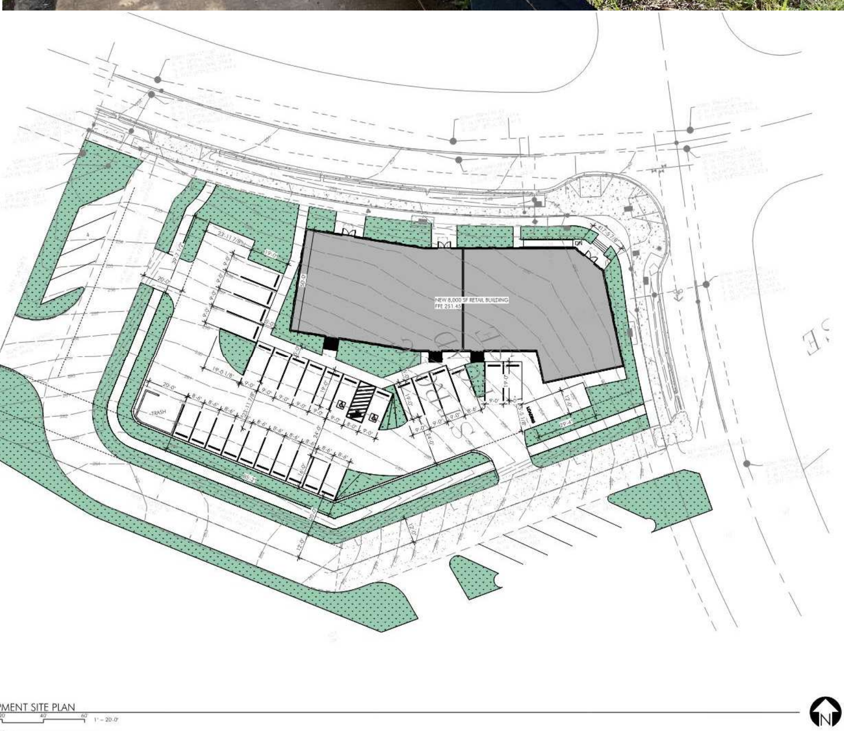
CGS Project: Dhaliwal-1-01

Central Geotechnical Services, LLC (CGS) is pleased to submit this report of geotechnical engineering services for the proposed Fairview Apartments project located at 2110 Strong Road SE in Salem, Oregon. The report was prepared for conformance with the signed proposal dated March 6, 2024. Please feel free to call our office with questions about this report.

Respectfully,

Central Geotechnical Services, LLC

Julio Vela, PhD, P.E., G.E.
Principal Engineer



Report of Geotechnical Engineering Services:

Fairview Apartments

CGS Project: Dhaliwal-1-01

Prepared For:

Inderjet Dhaliwal
2433 NW Broadway Street
Albany, Oregon

May 17, 2024

Submitted by:

CENTRAL
GEOTECHNICAL SERVICES, LLC

Table of Contents

LIST OF FIGURES

APPENDICES

1.0 INTRODUCTION

Central Geotechnical Services, LLC (CGS) is pleased to submit this geotechnical engineering report through Studio 3 Architecture (Studio 3) for the proposed Fairview Apartments development located at 2110 Strong Road SE in Salem, Oregon. The 1.08-acre parcel is bordered by Strong Road to the north, Lindburg Road to the east, and apartment buildings to the south and west. Figure 1 shows the site vicinity relative to surrounding features.

The site plan provided to us by Studio 3, indicates the proposed development consists of a mixed-use building with associated parking and landscaped areas. We have assumed that structural development will consist of wood-frame construction consistent with surrounding development and concept drawings provided to us.

Structural design loads were not provided at the time this report was prepared. However, we have assumed maximum column loads on the order of 50 kips per column and maximum wall loads on the order of 3 kips per lineal foot (klf), and floor loads for slabs on grade of 100 psf or less. We have also assumed that maximum cuts and fills for general site grading (not including cuts for structures below grade) will be less than 4 feet, and that on-site retaining walls will be less than 8 feet in height.

2.0 PURPOSE AND SCOPE OF WORK

The purpose of our services was to evaluate soil and groundwater conditions as a basis for developing geotechnical engineering design criteria for the proposed project. Our scope of services was provided in general accordance with our proposal dated March 6, 2024, which included the following:

1. Reviewed information regarding subsurface soil and groundwater in the vicinity of the site, including reports in our files, selected geologic maps, and other geotechnical engineering-related information.
2. Coordinated and managed the field investigation, including public utility notification and scheduling of subcontractors and CGS's field staff.
3. Explored subsurface soil and groundwater conditions at the project site by completing five drilled boring explorations (B-1 through B-5) to depths ranging between 5.5 to 21.5 feet below ground surface (bgs).
4. Obtained samples at representative intervals from the explorations, observed groundwater conditions and maintained detailed logs in general accordance with ASTM International (ASTM) Standard Practices Test Method D 2488.
5. Prepared to perform two (2) field infiltration tests as typically required by City of Salem for similar projects at a depth of 4 to feet bgs at locations as discussed with the project team. As a result of groundwater observed at proposed infiltration depths, we did not complete full testing as water seeped into the test borings which would negate infiltration measurements.
6. Performed laboratory tests on selected soil samples obtained from the explorations including moisture content determinations, Atterberg limit tests, and percent fines tests.

7. Provided a geologic hazard assessment of the site under separate cover as a memorandum and a geotechnical engineering evaluation and design recommendations provided in this geotechnical report addressing the following geotechnical components:
    a. A general description of site topography, geology and subsurface conditions.
    b. An opinion as to the adequacy of the proposed development from a geotechnical engineering standpoint.
    c. A summary of attempted field infiltration tests.
    d. Recommendations for site preparation measures, including disposition of undocumented fill and unsuitable native soils, recommendations for temporary cut slopes and constraints for wet weather construction.
    e. Recommendations for temporary excavation and temporary excavation protection, such as excavation sheeting and bracing.
    f. Recommendations for earthwork construction, including use of on-site and imported structural fill and fill placement and compaction requirements.
    g. Recommendations for foundations to support the proposed structures, including minimum width and embedment, design soil bearing pressures, settlement estimates (total and differential), coefficient of friction and passive earth pressures for sliding resistance. We have assumed that shallow foundations can be used to adequately support the structures.
    h. Recommendations for site retaining walls for walls up to 8 feet tall, including static and seismic active earth pressures, and drainage and backfill recommendations.
    i. Recommendations for supporting on-grade slabs, including base rock, capillary break, and modulus of subgrade reaction.
    j. Seismic design parameters, including soil site class evaluation in accordance with the 2018 International Building Code (IBC) and the 2019 version of the Oregon State Structural Code.
    k. Recommendations for constructing asphaltic concrete pavements for proposed on-site roadways, including subgrade, drainage, base rock and pavement section. Our recommendations will be based on estimated traffic loads based on City standards or loads provided by the project team and on subsurface data obtained as a part of this scope of work.

3.0 SITE CONDITIONS

3.1. Geology

Salem is situated in the Willamette Valley, which extends from Cottage Grove in the south to the Portland Basin in the north (Burns, 1998; Orr and Orr, 1999). The Willamette Valley is a tectonically active lowland and part of the Puget-Willamette Trough physiographic province; a forearc basin associated with the tectonically active Cascadia convergent margin. The lowland is generally an elongated alluvial plain bordered on the west by the Coast Ranges and on the east by the Cascade Mountains.

Basement rocks generally consist of the Miocene Columbia River Basalt Group (CRBG) (approximately 17 million to 6 million years old) (Bela, 1981; Tolan and Beeson, 2000). The CRBG comprises a series of thick basalt flows that filled lowland areas throughout much of the northern Willamette Valley. The basalt was subsequently faulted by the compressional tectonics of the region, which also resulted in the uplifting of the Salem Hills. The CRBG ranges up to hundreds of feet thick in areas where it is present (Tolan and Beeson, 2000).

The site is located in a low-lying area east of the Willamette River and north of the Salem Hills. The near-surface geologic unit is mapped as the Willamette Group, a Quaternary terrace deposits composed of mixed grained sediments (Yeats et.al, 1996).

3.2. Surface Conditions

The approximately 1.08-acre parcel is bordered by Strong Road to the north, Lindburg Road to the east, and apartment buildings to the south and west. The project site is currently a fenced, undeveloped parcel covered with low-lying grass and brush.

Based on USGS topography, the site is located in an area of gentle slopes that incline to the northeast at about 10 percent grade at an approximate elevation of 260 feet above mean sea level. Based on our observations, it appears that fill material has been stockpiled on the site with the ground surface extending 1 to 4 feet above the adjacent street grade. We observed concrete pieces and construction debris in the built-up fill material along Village Center Loop SE. The fill material appears to be thickest on the southwest portion of the site.

3.3. Subsurface Conditions

We explored subsurface conditions at the site by advancing five borings (B-1 through B-5). Borings were advanced to maximum depths of 16.5 and 21.5 feet below ground surface (bgs). The approximate locations of the explorations are shown on Figure 2. A description of the field explorations, the exploration logs, and the results of laboratory testing are present in Appendix A.

A 3- to 4-inch-thick root zone was observed at the ground surface in every exploration.

3.3.1. Fill

Fill was observed in all our explorations to depths of 3.5 to 7.6 feet below existing ground surface. The fill consists of soft to stiff silt with varying amounts of sand, gravel, and organic debris. Laboratory testing on select samples of the fill material indicates moisture contents of 25 to 27 percent at the time of our explorations.

3.3.2. Silt

Medium-stiff to very-stiff silt with varying amounts of sand and gravel was observed underlying the fill material. Laboratory testing on select samples indicated the silt material has high plasticity, with moisture contents ranging from 30 to 35 percent at the time of our explorations.

3.3.3. Sand

Underlying the silt on site, sand with varying amounts of silt and gravel was encountered in borings B-1, B-2, B-3, and B-5. Based on SPT blow counts observed in the field, the sand is generally medium-dense to dense. Moisture contents ranged from 36 to 50 percent at the time of our explorations. A particle size analysis conducted on one select sample of the sand indicated a percent fines of approximately 25%.

3.3.4. Gravel

Gravel was observed in borings B-2 and B-5 at depths of 18 and 17.5 feet bgs, respectively. The gravel was observed to the maximum depths explored. Based on SPT blow counts observed in the field, the gravel is dense to very dense.

3.4. Groundwater Conditions

Groundwater was observed within our explorations at depths of 4.2 feet to 5.5 feet bgs. The depth to groundwater may fluctuate in response to seasonal changes, prolonged rainfall, changes in surface topography, and other factors not observed in this study.

4.0 INFILTRATION TESTING

We were not able to perform infiltration testing at the time of our explorations due to shallow groundwater observed. Because of the generally sloped topography, the fine-grained nature of the near-surface site soil, and the presence of shallow groundwater, it is our opinion that the site is not well-suited for on-site infiltration systems for the disposal of stormwater. If infiltration facilities are required to be built as part of project development we recommend using a field infiltration rate of 1/8-inch per hour (in/hr) and designed facilities be designed using an overfill discharge to a suitable stormwater disposal system. The field infiltration rate is based on site conditions observed during our explorations and do not account for a factor of safety for site variability, or reduction factors that should be applied by the facility designer that depend on the type of system selected.

5.0 CONCLUSIONS

Based on our explorations, testing, and analyses, it is our opinion that the site is suitable for the proposed project from a geotechnical standpoint, provided the recommendations in this report are incorporated into the project design and implemented during construction. We offer the following conclusions regarding geotechnical engineering design and construction at the site.

- Groundwater was observed during our explorations between 4.2 and 5.5 feet bgs. Shallower levels of groundwater may be present during or following periods of persistent rainfall and on perched stiffer soil layers.
- Undocumented fill is present across the site. The undocumented fill encountered is generally soft to medium stiff.

- The proposed mixed-use structure can be satisfactorily supported on continuous and isolated shallow foundations supported on the firm native soils or on structural fill that extends to the firm native soils.
- There is a risk for poor performance of floor slabs established directly over undocumented fill. If undocumented fill is present at the proposed finished floor slab elevations, we recommend that the undocumented fill be removed and replaced with structural fill.
- Slabs on grade for proposed mixed-use structure can be satisfactorily supported on aggregate base that is founded on the firm native soils or on structural fill that extends to the firm native soils. We recommend that slabs-on-grade be provided with proper moisture control by constructing the aggregate base as a capillary break and providing a vapor retarder for moisture-sensitive applications.
- At minimum, the upper 12 inches of pavement subgrade should be improved by replacing it with imported granular structural fill or scarifying and recompacting it in place.

6.0 EARTHWORK RECOMMENDATIONS

6.1. Site Preparation

In general, initial site preparation and primary earthwork operations will include stripping and grubbing of upper organics, areas of light logging and brush clearing mass grading to create level working surfaces, excavating and filling for roads pavements, foundations, and utilities, demolition of existing structures, recompacting (dry weather) or replacing (wet weather) the undocumented fill in areas of the site that are to receive fill, fine-grading to establish final grades, or structural improvements.

All existing utilities in the proposed earthwork construction areas should be identified prior to excavation. Live utility lines beneath proposed structures should be completely removed or filled with grout to reduce potential settlement of new structures. Soft or loose soil encountered in utility line excavations should be removed and replaced with structural fill where it is located within structural areas.

Debris materials generated during demolition of existing improvements or relocation of utilities should be transported off site for disposal. Existing voids and new depressions created during site preparation, and resulting from removal of existing utilities, or other subsurface elements, should be cleaned of loose soil or debris down to firm soil and backfilled with compacted structural fill. Disturbance to a greater depth should be expected if site preparation and earthwork are conducted during period of wet weather.

6.2. Stripping

The existing root zone should be stripped and removed from all fill areas. Based on our explorations, the average depth of stripping will be approximately 3 to 4 inches, although greater stripping depths may be required to remove localized zones of loose or organic soil. The actual stripping depth should be based on field observations at the time of construction. Stripped material should be transported off site for disposal or used in landscaped areas.

6.3. Site Subgrade Preparation and Evaluation

Upon completion of site preparation activities, exposed subgrades should be proof-rolled with a fully loaded dump truck or similar heavy rubber-tired construction equipment where space allows to identify soft, loose or unsuitable areas. Probing may be used for evaluating smaller areas or where proof-rolling is not practical. Proof-rolling and probing should be conducted prior to placing fill and should be performed by a representative of CGS who will evaluate the suitability of the subgrade and identify areas of yielding that are indicative of soft or loose soil. We anticipate that there will be areas where soft or otherwise unsuitable soil is identified during subgrade evaluation. Unsuitable soil should be replaced by imported granular material or should be improved by scarifying and compacting the material in accordance with the "Structural Fill and Backfill" section.

As discussed in the Subgrade Protection and Wet Weather Considerations of this report, the fine-grained soil at the surface can be sensitive to small changes in moisture content and will be difficult, or not possible, to compact adequately during wet weather. While tilling and compacting the subgrade is the economical method for subgrade improvement, it will likely only be possible during extended dry periods and following moisture-conditioning of the soil.

During wet weather, or when the exposed subgrade is wet or unsuitable for proof-rolling, the prepared subgrade should be evaluated by observing excavation activity and probing with a steel foundation probe. Observations, probing and compaction testing should be performed by a member of our staff. Wet soil that has been disturbed due to site preparation activities or soft or loose zones identified during probing should be removed and replaced with compacted structural fill.

6.4. Subgrade Protection and Wet Weather Considerations

Upper site soils are highly susceptible to moisture. If wet weather construction practices are necessary based on conditions observed at the time of construction, it may be necessary to use track-mounted equipment, load removed material into trucks supported on gravel haul roads, use gravel working pads and employ other methods to reduce ground disturbance. The contractor should be responsible for protecting the subgrade during construction.

Earthwork planning should include considerations for minimizing subgrade disturbance. We provide the following recommendations if wet weather construction is considered:

- The ground surface in and around the work area should be sloped so that surface water is directed to a sump or discharge location. The ground surface should be graded such that areas of ponded water do not develop. Measures should be taken by the contractor to prevent surface water from collecting in excavations and trenches. Measures should be implemented to remove surface water from the work areas.
- Earthwork activities should not take place during periods of heavy precipitation.
- Slopes with exposed soils should be covered with plastic sheeting or similar means.
- The site soils should not be left in a disturbed or uncompacted state and exposed to moisture. Sealing the surficial soils by rolling with a smooth-drum roller prior to periods of precipitation may reduce the extent to which these soils become wet or unstable.

- Construction activities should be scheduled so that the length of time that soil is left exposed to moisture is reduced to the extent practicable.
- Construction traffic should be restricted to specific areas of the site, preferably areas that are not susceptible to wet weather disturbance such as haul roads and areas that are adequately surfaced with working pad materials.
- When on-site, moisture sensitive soils are wet of optimum, they are easily disturbed and will not provide adequate support for construction traffic nor for the proposed development. The use of granular haul roads and staging areas will be necessary to support heavy construction traffic. Generally, a 12- to 16-inch-thick mat of Imported Select Structural Fill should be sufficient for light staging areas for the building pad and light staging activities but is not expected to be adequate to support repeated heavy equipment or truck traffic. The thickness of the Imported Select Structural Fill for haul roads and areas with repeated heavy construction traffic should be increased to between 18 and 24 inches. The actual thickness of haul roads and staging areas should be determined at the time of construction and based on the contractor’s approach to site development and the amount and type of construction traffic.
- The base rock (Aggregate Base and Aggregate Subbase) thicknesses described in the “Pavement Recommendations” sections of this report are intended to support post-construction design traffic loads. The design base rock thicknesses will likely not support repeated heavy construction traffic during site construction or during pavement construction. A thicker base rock section as described above for haul roads will likely be required to support construction traffic.
- During periods of wet weather, concrete should be placed as soon as practical after preparing foundation excavations. Foundation bearing surfaces should not be exposed to standing water. Should water infiltrate and pool in the excavation, the water should be removed, and the foundation subgrade should be re-evaluated before placing reinforcing steel or concrete. Foundation subgrade protection, such as a 3- to 4-inch thickness of Aggregate Base/Aggregate Subbase or lean concrete, may be necessary if footing excavations are exposed to extended wet weather conditions.

During wet weather, or when the exposed subgrade is wet or unsuitable for proof-rolling, the prepared subgrade should be evaluated by observing excavation activity and probing with a steel foundation probe. Observations and probing should be performed by a member of our staff. Wet soil that has been disturbed due to site preparation activities, or soft or loose zones identified during probing, should be removed, and replaced with Imported Select Structural Fill.

6.5. Dewatering

As discussed in the “Groundwater” section of this report, groundwater was encountered in our explorations. However, we do not expect groundwater to be a major factor during shallow excavations and earthwork. Excavations that extend into saturated/wet soils, or excavations that extend into perched groundwater, should be dewatered. Sump pumps are expected to adequately address groundwater encountered in shallow excavations. In addition to groundwater seepage, surface water inflow to the excavations during the wet season can be problematic. Provisions for surface water control during earthwork and excavations should be included in the project plans and should be installed prior to commencing earthwork.

6.6. Excavation

Excavations will be required for the installation of new foundations, utilities, and other earthwork activities. Conventional earthmoving equipment in proper working conditions should generally be capable of making the necessary excavations. Excavations deeper than 4 feet bgs will likely require shoring or should be sloped.

Open excavation techniques may be used to excavate trenches to depths of 4 feet bgs, provided the walls of the excavation are cut at a slope of 1H:1V and groundwater seepage is not present. At this inclination, the slopes may ravel and require some ongoing repair. Excavations should be flattened if excessive sloughing or raveling occurs. In lieu of large and open cuts, approved temporary shoring may be used for excavation support. A wide variety of shoring and dewatering systems are available. Consequently, we recommend the contractor be responsible for selecting the appropriate shoring and dewatering systems.

If box shoring is used, it should be understood that box shoring is a safety feature used to protect workers and does not prevent caving. If excavations are left open for extended periods of time, caving of sidewalls will likely occur. The presence of caved material will limit the ability to properly backfill and compact the trenches. The contractor should be prepared to fill voids between the box shoring and the sidewalls of the trenches with sand or gravel before caving occurs.

If shoring is used, we recommend that the type and design of the shoring system be the responsibility of the contractor, who is in the best position to choose a system that fits the overall plan of operation.

6.7. Drainage Considerations

6.7.1. Temporary

During earthwork at the site, the contractor should be responsible for the temporary drainage of surface water as necessary to prevent standing water and/or erosion at the working surface. During rough and finished grading of the site, the contractor should keep all pads and subgrade free of ponding water.

6.7.2. Surface

The ground surface around the finished building pads should be sloped away from the edge of the pad at a minimum 2 percent gradient for a distance of at least 5 feet. Downspouts or roof scuppers should discharge into a storm drain system that carries collected water to an appropriate stormwater system. Trapped planter areas should not be created adjacent to pavement and structures without providing means for positive drainage (e.g., swales or catch basins).

6.8. Permanent Slopes

Permanent cut and fill slopes should not exceed 2H:1V. Slopes that will be maintained by mowing should be constructed steeper than 3H:1V. Access roads and pavement should be located at least 5 feet from the top of cut and fill slopes. The setback should be increased to 10 feet for buildings. The slopes should be planted with appropriate vegetation to provide protection against erosion as soon as possible after grading. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope.

6.9. Structural Fill and Backfill

6.9.1. General

Structural areas include areas beneath foundations, floor slabs, pavements, and any other areas intended to support structures or within the influence zone of structures. Fill intended for use in structural areas should meet the criteria for structural fill presented below. All structural fill soils should be free of debris, clay balls, roots, organic matter, frozen soil, man-made contaminants, particles with greatest dimension exceeding 4 inches (3-inch-maximum particle size in building footprints) and other deleterious materials.

Recommendations for suitable fill material are provided in the following sections.

6.9.2. On-Site Soil

The on-site material should generally be suitable for use as general structural fill, provided it is properly moisture conditioned; free of debris, organic material, and particles over 6 inches in diameter; and meets the specifications provided in OSSC 00330.12 (Borrow Material). The suitability of soil for use as structural fill will depend on the gradation and moisture content of the soil. As the amount of fines in the soil matrix increases, the soil becomes increasingly sensitive to small changes in moisture content and achieving the required degree of compaction becomes more difficult or impossible.

When used as structural fill, native soil should be placed in lifts with a maximum uncompacted thickness of 6 to 8 inches and compacted to not less than 95 percent of the maximum dry density for granular soil, as determined by ASTM D1557.

If desired, an experienced geotechnical engineer from CGS can determine the suitability of on-site soil encountered during earthwork activities for use as structural fill.

6.9.3. Imported Granular Material

Imported granular material used as structural fill should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand. The imported granular material should also be durable, angular, and fairly well graded between coarse and fine material; should have less than 5 percent passing the U.S. No. 200 sieve (3 percent for retaining walls) by dry weight; and should have at least two mechanically fractured faces. In addition, aggregate base shall have a minimum of 75 percent fractured particles according to AASHTO T-355 and a sand equivalent of not less than 30 percent based on AASHTO T-176.

6.9.4. Trench Backfill

Backfill for pipe bedding and in the pipe zone should consist of well-graded granular material with a maximum particle size of ¾ inch and less than 5 percent passing the U.S. No. 200 sieve. The material should be free of organic matter and other deleterious materials. Further, the backfill should meet the pipe manufacturer's recommendations. Above the pipe zone backfill, Imported Select Structural Fill may be used as described above.

6.9.5. Pavement Aggregate Base

Imported granular material used as base rock for pavement should consist of ¾- or 1 ½-inch-minus material. In addition, the aggregate should have less than 5 percent fines by dry weight and have at least two mechanically fractured faces. The aggregate base should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

6.9.6. Drain Rock Material

Drain rock should consist of angular, granular material with a maximum particle size of 2 inches. The material should be free of roots, organic material, and other unsuitable material; should have less than 2 percent by dry weight passing the U.S. Standard No. 200 sieve (washed analysis); and should have at least two mechanically fractured faces. Drain rock should be compacted to a well-keyed, firm condition.

6.9.7. Fill Placement and Compaction

Structural fill should be compacted at moisture contents that are within 3 percent of the optimum moisture content as determined by ASTM International (ASTM) Test Method D 1557 (Modified Proctor). The optimum moisture content varies with gradation and should be evaluated during construction. Fill material that is not near the optimum moisture content should be moisture conditioned prior to compaction.

Fill and backfill material should be placed in uniform, horizontal lifts and compacted with appropriate equipment. The appropriate lift thickness will vary depending on the material and compaction equipment used. Fill material should be compacted in accordance with Table 2. It is the contractor's responsibility to select appropriate compaction equipment and place the material in lifts that are thin enough to meet these criteria. However, in no case should the loose lift thickness exceed 18 inches. Initial lift thickness over pipe may need to be thicker than 18 inches to prevent damage to the pipe during the application of compactive effort.

A representative from CGS should evaluate the compaction of every two vertical feet (or less) and 500 cubic yards of fill material placed. Compaction should be evaluated by compaction testing unless other methods are proposed for oversized materials and are approved by CGS during construction. These other methods typically involve procedural placement and compaction specifications together with verification requirements such as proof-rolling.

Table 2. Compaction Criteria

Fill Type.	Compaction Requirements		
	Percent Maximum Dry Density Determined by ASTM Test Method D 1557 at ± 3% of Optimum Moisture		
	0 to 2 Feet Below Subgrade	> 2 Feet Below Subgrade	Pipe Zone
Fine-grained soils (non-expansive)	92	92	-----
Imported Granular, maximum particle size < 1½ inch	95	95	-----
Imported Granular, maximum particle size 1½ inch to 6 inches (3-inch-maximum under building footprints)	n/a (proof-roll)	n/a (proof-roll)	-----
Retaining Wall Backfill*	92	92	------
Nonstructural Zones	90	90	90
Trench Backfill	95	92	90

Note:

* Measures should be taken to prevent overcompaction of the backfill behind retaining walls. We recommend placing the zone of backfill located within 5 feet of the wall in lifts not exceeding about 6 inches in loose thickness and compacting this zone with hand-operated equipment such as a vibrating plate compactor or a jumping jack.

6.10. Retaining Walls

6.10.1. Assumptions

Our retaining wall design recommendations are based on the following assumptions: (1) walls consist of conventional, cantilevered retaining walls, (2) walls are less than 8 feet in height, (3) the backfill is drained and consists of imported granular materials, and (4) the backfill has a finish slope flatter than 4H:1V. Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project varies from these assumptions.

6.10.2. Subgrade Preparation

Wall footings should bear on a minimum 12-inch-thick layer of imported granular material underlain by firm, undisturbed native soil. The imported granular material should be fairly well graded between coarse and fine material, have less than 6 percent by dry weight passing the U.S. Standard 200 sieve, should have at least two mechanically fractured faces, and be compacted to 95% of maximum dry density as obtained from ASTM D1557.

6.10.3. Wall Design Parameters

Wall footings prepared as recommended should be sized based on an allowable bearing pressure of 2,500 psf. The weight of the footing and overlying backfill can be ignored in calculating footing sizes.

For unrestrained retaining walls, an active equivalent fluid pressure of 35 pcf should be used for design. Where retaining walls are restrained from rotation (such as basement walls), an at-rest equivalent fluid pressure of 55 pcf should be used for design. A superimposed seismic lateral force should be calculated based on a dynamic force of $7H^2$ pounds per lineal foot of wall, where H is the height of the wall in feet, and applied as a distributed load with the centroid located at a distance of 0.6H from the base of the wall.

If surcharges (e.g., structure foundations, concrete slabs, vehicles, steep slopes, terraced walls, etc.) are located within a horizontal distance from the back of a wall equal to the height of the wall, additional pressures will need to be accounted for in the wall design. Our office should be contacted for appropriate wall surcharges based on the actual magnitude and configuration of the applied loads. The base of the wall footing should extend a minimum of 12 inches below the lowest adjacent finish grade.

Lateral loads on the proposed structures can be resisted by a combination of sliding resistance on the base of footings and passive earth pressure on the sides of footings. We recommend a coefficient of friction of 0.35 for footings bearing on undisturbed, native silt, and 0.45 for footings bearing on granular engineered structural fill.

Passive earth pressures on the sides of buried spread footings may be calculated using an equivalent fluid pressure of 300 pcf per foot of embedment. For this value, backfill against the footing should be compacted to at least 92% of the maximum dry density as obtained from ASTM D1557. The upper foot of embedment should be neglected unless protected by pavement or concrete slabs on grade.

6.10.4. Wall Drainage and Backfill

The above design parameters have been provided assuming back-of-wall drains will be installed to prevent buildup of hydrostatic pressures behind all walls. If a drainage system is not installed, CGS should be contacted for revised design forces.

Backfill material placed behind retaining walls and extending a minimum horizontal distance of H, where H is the height of the retaining wall, should consist of select granular wall backfill meeting the requirements described in the "Structural Fill and Backfill" section. All wall backfill should be compacted to at least 92% of the maximum dry density as obtained from ASTM D1557. Wall backfill in the top 2 feet should be compacted to at least 95% of the maximum dry density when under structural areas such as footings, concrete slabs, pavement.

Perforated collector pipes should be placed at the base of the granular backfill behind the walls. The pipe should be embedded in a minimum 1-foot-wide zone of angular drain rock and the drain rock should be wrapped in a drainage geotextile fabric. The collector pipes should discharge at an appropriate location away from the base of the wall. The discharge pipe should not be tied directly into stormwater drain systems, unless measures are taken to prevent backflow into the drainage system of the wall.

6.10.5. Settlement

Settlement of up to 1 percent of the wall height commonly occurs immediately adjacent to the wall as the wall rotates and develops active lateral earth pressures. Consequently, we recommend that construction of flatwork adjacent to retaining walls be postponed at least four weeks after backfilling of the wall, unless survey data indicates that settlement is complete prior to that time.

6.11. Asphalt Concrete (AC)

Design pavement sections are provided in Section 8.0 below. Pavement material recommendations are described in the following subsection.

6.11.1. Asphalt Concrete Pavement (ACP)

The AC should be Level 2, ½ inch, dense ACP according to OSSC 00744 (Asphalt Concrete Pavement) and compacted to 91 percent of the theoretical maximum density of the mix, as determined by AASHTO T209. The minimum and maximum lift thicknesses are 2 and 3.5 inches, respectively, for ½-inch ACP. Asphalt binder should be performance graded and conform to PG 64-22 or better. The binder grade should be adjusted depending on the aggregate gradation and amount of recycled asphalt pavement and/or recycled asphalt shingled in the contractor's mix design controls.

6.11.2. Cold Weather Paving Considerations

In general, AC paving is not recommended during cold weather (surface temperature less than 40 degrees Fahrenheit). Compacting under these conditions can result in low compaction and premature pavement distress.

Each AC Mix design has a recommended compaction temperature range that is specific for the particular AC binder used. In colder temperatures, it is more difficult to maintain the temperature of the AC mix as it can lose heat while stored in the delivery truck, as it is placed, and in the time between placement and compaction. In Oregon, the surface temperature during paving should be at least 40 degrees Fahrenheit for lift thickness greater than 2.5 inches and at least 50 degrees Fahrenheit for lift thicknesses between 2 and 2.5 inches.

If paving activities must take place during cold weather construction as defined above, the project team should be consulted and a site meeting should be held to discuss ways to lessen low compaction risks.

7.0 STRUCTURAL DESIGN RECOMMENDATIONS

7.1. Foundation Support Recommendations

Based on the results of our explorations, and assumed building loads, the proposed buildings can be supported on conventional shallow footings bearing on firm, undisturbed native soil or structural fill placed over firm, undisturbed native soil. Continuous wall and isolated spread footings should be at least 16 and 20 inches wide, respectively. The bottom of exterior column or continuous footings should be at least 18 inches below the adjacent exterior grade. The bottom of the interior footings should be established at least 12 inches below the base of the slab.

Foundations should not be established over the existing undocumented fill or on soft soil. Grading plans were not available at the time of this report. Based on a site plan provided by Studio 3 Architecture, the proposed building will be located in the northeast corner of the site. The undocumented fill was observed to depths of approximately 4 to 5 feet in the planned location of the building. We recommend that the undocumented fill be completely removed from all foundation areas prior to site grading. It is important to do this prior to grading so that the undocumented fill is not covered by fill in isolated areas during grading.

7.1.1. Foundation Subgrade Preparation

The subgrades beneath proposed structural elements should be prepared as described below and in the "earthworks" section of this report. We recommend loose or disturbed soils resulting from foundation excavation be removed before placing reinforcing steel and concrete. Foundation bearing surfaces should not be exposed to standing water. If water infiltrates and pools in the excavation, the water, along with any disturbed soil, should be removed before placing reinforcing steel and concrete. A thin gravel layer consisting of Aggregate Base or Aggregate Subbase material can be placed at the base of foundation excavations to help protect the subgrade from weather and light foot traffic. The layer thickness for the gravel layer should be determined at the time of construction but is typically 3 to 4 inches. The gravel layer should be compacted as described in the "Fill Placement and Compaction" section.

We recommend CGS observe all foundation subgrades before placing concrete forms and reinforcing steel to determine that bearing surfaces have been adequately prepared and the soil conditions are consistent with those observed during our explorations.

7.1.2. Spread Footings

We recommend conventional footings be proportioned using a maximum allowable bearing pressure of 2,500 psf if supported on firm native soils or on structural fill placed over firm native soils. This bearing pressure applies to the total of dead and long-term live loads and may be increased by one-third when considering earthquake or wind loads. This is a net bearing pressure. The weight of the footing and overlying backfill can be ignored in calculating footing sizes.

7.1.3. Foundation Settlement

Foundations designed and constructed as recommended are expected to experience settlements of less than 1 inch. Differential settlements of up to one half of the total settlement magnitude can be expected between adjacent footings supporting comparable loads.

7.1.4. Lateral Resistance

Lateral loads can be resisted by a combination of friction between the footing and the supporting soil, and by the passive lateral resistance of the soil surrounding the embedded portions of the footings. A coefficient of friction between the concrete and soil of 0.35 and a passive lateral resistance corresponding to an equivalent fluid density of 250 pcf may be used for design. These values are appropriate for foundation elements that are poured directly against the native soils or surrounded by compacted structural fill.

The passive earth pressure and friction components may be combined, provided the passive component does not exceed two-thirds of the total.

The passive earth pressure value is based on the assumptions that the adjacent grade is level and static groundwater remains below the base of the footing throughout the year. The top 1 foot of soil should be neglected when calculating passive lateral earth pressures unless the adjacent area is covered with pavement. The lateral resistance values do not include safety factors.

7.2. Drainage Considerations

We recommend the ground surface be sloped away from buildings at least 5 percent for a minimum distance of 10 feet measured perpendicular to the face of the wall in accordance with section 1804.4 of the 2018 International Building Code (IBC). All downspouts should be tightlined away from the building foundation areas and should also be discharged into a stormwater disposal system. Downspouts should not be connected to footing drains.

Based on the potential for perched groundwater to move downslope during conditions at the time of our explorations, we recommend the inclusion of perimeter footing drains for the multistory-residential structures constructed on the east and west slopes. While not required for other structures based on the groundwater depths observed in our explorations, if perimeter footing drains are used for other site structures they should be installed at the base of exterior footings.

Perimeter footing drains should be installed for below-grade structural elements or crawlspaces to control relatively shallow perched groundwater conditions. Footing drains should be installed at the base of exterior building footings where interior spaces should be protected from inflowing water from surrounding soils. Perimeter footing drains should be provided with cleanouts and should consist of at least 4-inch-diameter perforated pipe placed on a 3-inch bed of and surrounded by 6 inches of drainage material enclosed in a non-woven geotextile such as Mirafi 140N (or approved equivalent) to prevent fine soil from migrating into the drain material in accordance with OSSC Section 1805.4.2. We recommend against using flexible tubing for footing drainpipes. The perimeter drains should be sloped to drain by gravity to a suitable discharge point, preferably a storm drain. We recommend that the cleanouts be covered and placed in flush-mounted utility boxes. Water collected in roof downspout lines must not be routed to the footing drain lines.

7.3. Floor Slabs

Satisfactory subgrade support for floor slabs on grade supporting the planned 1000 psf floor loads can be obtained provided the floor slab subgrade is described in the "Earthwork Recommendations" section of this report. Slabs should be reinforced according to their proposed use and per the structural engineer's recommendations. Subgrade support for concrete slabs can be obtained from the firm native soils underlying the topsoil or on structural fill placed over firm native soils.

We recommend that on-grade slabs be underlain by a minimum 6-inch-thickness of aggregate base in order to provide the structural design support for subgrade reaction as described below and to act as a capillary break material to reduce the potential for moisture migration into the slab. The aggregate base section should be placed as recommended in the "Fill Placement and Compaction" section of this report.

If dry on-grade slabs are required, for example at interior spaces where adhesives are used to anchor carpet or tile to the slab, a waterproof liner may be placed as a vapor barrier below the slab. The vapor barrier should be selected by the structural engineer and should be accounted for in the design floor section and mix design selection for the concrete, to accommodate the effect of the vapor barrier on concrete slab curing.

Load-bearing concrete slabs prepared as recommended should be designed assuming a modulus of subgrade reaction (k) of 150 psi per inch. This value is for a 1-foot by 1-foot square plate. The coefficient of subgrade reaction for a foundation varies based on its minimum width according to the following equation:

$$k_s = k_{s1}\left[\frac{B+1}{2B}\right]^2$$

Where k_s is the coefficient of subgrade reaction, ks1 is the coefficient of subgrade reaction for a 1-ft by 1-ft plate, and B is the minimum width or lateral dimension of the mat.

If dry on-grade slabs are required, for example at interior spaces where adhesives are used to anchor carpet or tile to the slab, a waterproof liner may be placed as a vapor barrier below the slab. The vapor retarder should be selected by the structural engineer and should be accounted for in the design floor section and mix design selection for the concrete, to accommodate the effect of the vapor barrier on concrete slab curing.

We estimate that concrete slabs constructed as recommended will settle less than ½ inch. Floor slab subgrades should be evaluated according to the "Subgrade Evaluation" section of this report.

7.4. Seismic Design

Parameters provided on Table 4 are based on the conditions encountered during our subsurface exploration program and the procedure and requirements outlined in the 2018 IBC. Per American Society of Civil Engineers (ASCE) 7-16 Section 11.4.8, a site-specific response analysis is required for site class F sites, and a ground motion hazard analysis or site-specific response analysis is required to determine the design ground motions for structures on Site Class D and E sites with S_1 greater than or equal to 0.2g.

For this project, the site is classified as site class D; therefore, the provisions of 11.4.8 are applicable. Alternatively, the parameters listed in Table 4 may be used to determine the design ground motions if the exceptions provided in ASCE 7-16 Supplement 3 are met and the fundamental period of the structure is less than 0.5 seconds. Exception 2 states "Structures on Site Class D sites with S_1 greater than or equal to 0.2, provided the value of the seismic response coefficient C_S is determined by Eq. (12.8-2) for values of $T \leq 1.5T_S$ and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) for $T_L \geq T \leq 1.5\ T_S$ or Eq. (12.8-4) for $T > T_L$". If it is desirable to avoid these exceptions, a ground motion hazard analysis or site response analysis which is outside of the scope of services for this report, would need to be completed to determine the design seismic parameters for the site.

Table 4. Mapped ASCE 7-16 Seismic design parameters

Parameter	Recommended Value[1]
Site Class	D
Mapped Spectral Response Acceleration at Short Period (S_S)	0.814 g
Mapped Spectral Response Acceleration at 1 Second Period (S_1)	0.41 g
Site Modified Peak Ground Acceleration (PGA_M)	0.462 g
Site Amplification Factor at 0.2 second period (F_a)	1.175
Site Amplification Factor at 1.0 second period (F_v)	1.89
Design Spectral Acceleration at 0.2 second period (S_{DS})	0.637 g
Design Spectral Acceleration at 1.0 second period (S_{D1})	0.517 g

Note:
[1] Parameters developed based on Latitude 45.009579° and Longitude -123.02222 °using the ATC Hazards online tool).

7.5. Liquefaction Potential

Liquefaction is a phenomenon caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. The excessive buildup of pore water pressure results in the sudden loss of shear strength in a soil. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. Sand boils and flows observed at the ground surface after an earthquake are the result of excess pore pressures dissipating upwards, carrying soil particles with the draining water. In general, loose, saturated sand soil with low silt and clay contents is the most susceptible to liquefaction. Low plasticity, silty sand may be moderately susceptible to liquefaction under relatively higher levels of ground shaking.

Based on the subsurface conditions encountered in the explorations and the laboratory testing results, it is our opinion that there is a low risk of seismic-induced liquefaction at the site during the design-level earthquake.

8.0 PAVEMENT RECOMMENDATIONS

Our pavement recommendations are based on the results of our on-site field testing as described below, and our analysis. The recommended pavement sections assume that final improvements surrounding the pavement will be designed and constructed such that stormwater or excess irrigation water from landscape areas does not infiltrate below the pavement section into the base rock materials.

New pavement will be required for the planned new parking lot south of the existing building. Pavement should be installed on native subgrade or engineered fills prepared in conformance with the "Earthwork Recommendations" and "Structural Fill" sections. Our pavement recommendations are based on the following assumptions:

- The top 12 inches of soil subgrade are compacted to at least 92 percent of its maximum dry density, as determined by ASTM D1557, or until proof rolling with heavy equipment indicates that it is firm and unyielding.

- Resilient moduli of 4,500 psi and 20,000 psi were assumed for the subgrade and aggregate base, respectively.
- The design manual provided for the project specifies pavement recommendations based on a design life of 20 years.
- Initial and terminal serviceability indices of 4.2 and 2.5, respectively.
- Reliability of 85 percent and standard deviation of 0.45.
- Traffic consists of passenger vehicles and two- to three-axle trucks, such as delivery or garbage trucks.

We have assumed average daily traffic of up to 300 passenger vehicle trips per day and up to 5 truck trips per day. Our pavement design recommendations and specific assumed breakdown of traffic conditions are summarized in the table below. All recommended pavement sections will be able to support occasional 75,000-pound fire truck traffic.

Table 2. Minimum AC Pavement Sections for on-site Development

Section	Minimum Asphalt Thickness (inches)	Minimum Aggregate Base Thickness (inches)	Assumed Traffic Loading (Design Life ESAL's)
Light Duty (General Automobile Traffic)	2.5	6.0	<10,000
Heavy Duty (drive aisles and heavy delivery areas)	3.0	8.0	50,000

All thicknesses are intended to be the minimum acceptable. Design of the recommended pavement section assumes that construction will be completed during an extended period of dry weather. Wet weather construction could require an increased thickness of aggregate base.

Construction traffic should be limited to non-building, unpaved portions of the site or haul roads. Construction traffic should not be allowed on new pavement. If construction traffic is to be allowed on newly constructed pavement, an allowance for this additional traffic will need to be made in the design pavement section. The aggregate base thicknesses do not account for construction traffic, and haul roads and staging areas should be used.

9.0 LIMITATIONS OF REPORT

We have prepared this report for the exclusive use of Inderjet Dhaliwal and their authorized parties for the project specifically identified in this report only. The report should be provided in its entirety to prospective contractors for bidding and estimating purposes; however, the conclusions and interpretations presented should not be construed as a warranty of the subsurface conditions. Experience has shown that soil and groundwater conditions can vary significantly over small distances. Inconsistent conditions can occur between explorations that may not be detected by a geotechnical study. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, CGS should be notified for review of the recommendations of this report, and revision of such if necessary.

This report is not intended for use by others, and the information contained herein is not applicable to other sites. No other party may rely on the product of our services unless we agree in advance and in writing to such reliance.

We recommend that CGS be retained to review the plans and specifications and verify that our recommendations have been interpreted and implemented as intended. Sufficient geotechnical monitoring, testing and consultation should be provided during construction to confirm that the conditions encountered are consistent with those indicated by explorations. Recommendations for design changes will be provided should conditions revealed during construction differ from those anticipated. Should CGS not be retained for Design or Construction related services further into the development process, this report and its recommendations should be considered void, as we cannot take on responsibility for construction operations that were unobserved by our office.

Within the limitations of scope, schedule and budget, the analysis, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology in this area at the time the report was prepared.

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11.0 SIGNATURES

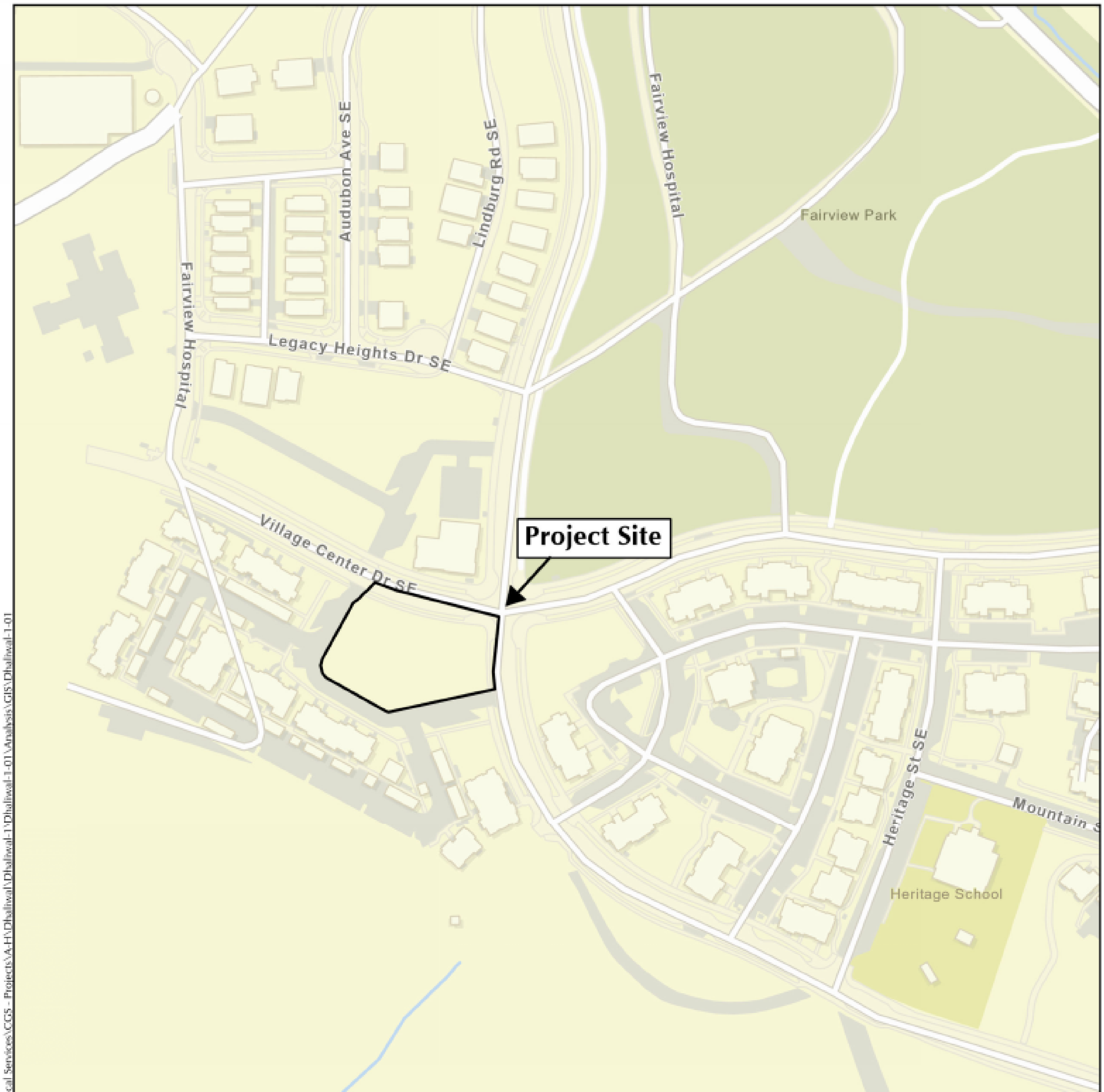
We appreciate the opportunity to provide geotechnical services to support your project. If you feel obliged, we welcome referrals from our previous clients and would enjoy the opportunity to work with others in your professional and personal networks.

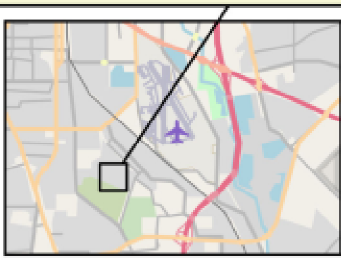
Central Geotechnical Services, LLC

REGISTERED PROFESSIONAL ENGINEER
60333
Digitally signed by Julio Vela
05/17/24
OREGON
FEBRUARY 08, 2000
JULIO C VELA
EXPIRES: 06/30/24

Julio Vela, PhD, PE, GE
Principal Engineer

Jessica Pence, EIT
Project Manager





N

0 250 500 Feet

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Dhaliwal-1-01
Project Vicinity
CENTRAL GEOTECHNICAL SERVICES, LLC | Figure-1

Default Folder: C:\Users\cgsuser.GIS-WORKSTATION\Central Geotechnical Services\CGS - Projects\A-H\Dhaliwal\Dhaliwal-1\Dhaliwal-1-01\Analysis\GIS\Dhaliwal-1-01

Legend

Exploration Designation and Approximated Location

Boring

Infiltration

N

0 75 150 Feet

Dhaliwal-1-01	
Site Plan	
CENTRAL Geotechnical Services, LLC	Figure-2

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CENTRAL
GEOTECHNICAL SERVICES

APPENDIX A: Field Explorations

APPENDIX A
FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

Soil and groundwater conditions at the proposed project were explored on March 28, 2024, by completing five drilled borings (B-1 through B-5) at the approximate locations shown on the Site Plan, Figure 2. The exploratory borings were extended to final depths of between 5.5- to 21.5-feet below ground surface using a 4-3/8-inch diameter solid stem auger drilling technique. The borings were completed with a drill rig owned and operated by Dan J. Fischer Excavating.

The drilling were continuously monitored by a qualified staff from our office who maintained detailed logs of subsurface explorations, visually classified the soil encountered and obtained representative soil samples from the borings. Representative soil samples were obtained from each boring at approximate 2½-and 5 foot-depth intervals using a 1-inch, inside-diameter, standard split spoon sampler. The samplers were driven into the soil using a 140-pound hammer, free-falling 30 inches on each blow. The number of blows required to drive the sampler each of three, 6-inch increments of penetration were recorded in the field. The sum of the blow counts for the last two, 6-inch increments of penetration is reported on the boring logs as the ASTM International (ASTM) Test Method D 1556 Standard Penetration Test (SPT) N-value.

Recovered soil samples from exploratory borings were visually classified in the field in general accordance with ASTM D 2488 and the classification chart listed in Key to Exploration Logs. Logs of the borings are presented in this Appendix. The logs are based on interpretation of the field and laboratory data and indicate the depth at which subsurface materials, or their characteristics change, although these changes may actually be gradual.

Laboratory Testing

Soil samples obtained from the explorations were visually classified in the field and in our laboratory using the USCS and ASTM classification methods. ASTM Test Method D 2488 was used to visually classify the soil samples, while ASTM D 2487 was used to classify the soils based on laboratory tests results. Moisture content tests were performed in general accordance with ASTM D 2216-05. Results of the moisture contents testing are presented in the appropriate exploration logs at the respective sample depths.

Selected samples were “washed” through the U.S. No. 200 mesh sieve to estimate the relative percentages of coarse- and fine-grained particles in the soil. The percent passing value represents the percentage by weight of the sample finer than the U.S. No. 200 sieve. These tests were conducted to verify field descriptions and to estimate the fines content for analysis purposes. The tests were conducted in accordance with ASTM D 1140, and the results are shown on the exploration logs in Appendix A at the respective sample depths.

Exploration and Soil Classification Key

rev. 051624

Relative Density - Coarse-Grained Soil

Term	SPT (140-lb Hammer)*	D&M Sampler (140-lb Hammer)*	D&M Sampler (300-lb Hammer)*
Very-loose	0-4	0-11	0-4
Loose	4-10	11-26	4-10
Medium-dense	10-30	26-74	10-30
Dense	30-50	74-120	30-47
Very-dense	>50	>120	>47

Consistency - Fine-Grained Soil

Term	SPT (140-lb Hammer)*	Sampler (140-lb Hammer)*	Sampler (300-lb Hammer)*	Pocket Pen (tsf)	Torvane (tsf)
Very-soft	0-2	0-3	0-2	<0.25	<0.13
Soft	2-4	3-6	2-5	0.25-0.5	0.13-0.25
Medium-stiff	4-8	6-12	5-9	0.5-1	0.25-0.5
Stiff	8-15	12-25	9-19	1.0-2.0	0.5-1.0
Very-stiff	15-30	25-65	19-31	2.0-4.0	1.0-2.0
Hard	>30	>65	>31	>4.0	>2.0

SPT N-value correlation based off ASTM D1586

Unified Soil Classification System (USCS)

USCS Symbols	Graph	Typical Descriptions
GP		Poorly graded GRAVEL, <5% fines
GP-GM/GP-GC		Poorly graded GRAVEL w/ silt/clay, 5 to 12% fines
GM		silty GRAVEL, over 12% fines
GC		clayey GRAVEL, over 12% fines
GW		well graded GRAVEL, <5% fines
SP		poorly graded SAND, <5% fines
SP-SM/SP-SC		poorly graded SAND w/ silt/clay, 5 to 12% fines
SM		silty SAND, over 12% fines
SC		clayey SAND, over 12% fines
SW		well graded SAND, <5% fines
ML		SILT, low plasticity
MH		SILT, high plasticity
CL		CLAY, low plasticity
CH		CLAY, high plasticity
OL		ORGANIC SILT, low plasticity
OH		ORGANIC CLAY, medium to high plasticity
PT		PEAT

ADDITIONAL CONSTITUENTS

Silt/Clay in: Percent*	Fine-Grained	Coarse-Grained	Sand/Gravel in: Percent*	Fine-Grained	Coarse-Grained
<5	trace	trace	<5	trace	trace
5-12	minor	with	5-15	minor	minor
>12	some	silty/clayey	15-30	with	with
			>30	sandy/gravelly	with

GEOTECHNICAL TESTING EXPLANATIONS

ATT	Atterberg Limits
CBR	California Bearing Ratio
CON	Consolidation
DD	Dry Density
DS	Direct Shear
HYD	Hydrometer Gradation
LL	Liquid Limit
PL	Plastic Limit
PI	Plasticity Index
MC	Moisture Content
MD	Moisture-Density
NP	Non-Plastic
OC	Organic Content
P	Pushed Sample
PP	Pocket Penetrometer
Passing No.200	Percent Passing U.S. Std. No.200 Sieve
RES	Resilient Modulus
SIEV	Sieve Gradation
TOR	Torvane
UC	Unconfined Compressive Strength
VS	Vane Shear

CONTACT LINES

(solid line)	Distinct contact between soil strata (approximate location)
(dashed line)	Approximate contact between soil strata

WATER LEVELS

	Water Level at Time of Drilling, or as labeled
	Water Level at End of Drilling, or as labeled
	Static Water Level, or as labeled

Moisture (ASTM D2488)

Dry	Very low moisture, dry to touch
Moist	Damp, without visible moisture
Wet	Visible free water, usually saturated

ADDITIONAL MATERIALS

AC		ASPHALT CONCRETE
CC		CEMENT CONCRETE
CR		CRUSHED ROCK
SOD		SOD/FOREST DUFF
FILL		FILL

Samplers

SYMBOL	SAMPLER DESCRIPTIONS	SYMBOL	SAMPLER DESCRIPTIONS
	Location of grab sample (GS)		Location of sample collected using Standard Penetration Test with recovery (SS)
	No Recovery		Location of sample collected using Shelby tube/Geoprobe sample with recovery (ST)
	Location of rock coring interval (RC)		Location of sample collected using Dames & Moore sampler or pushed with recovery (D&M)

CENTRAL Geotechnical Services
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10240 SW Nimbus Ave, Suite L6
Portland, OR 97223
Telephone: (503) 616-9419

Project No: Dhaliwal-1-01

BORING LOG B-1

Project: Fairview Apartments
Location: 2110 Strong Road SE, Salem, OR 97302
Client: Studio 3 Architecture

Date Started: 3/28/24
Date Completed: 3/28/24

Approximate Ground Elevation: 252ft
Groundwater first observed: 5.50 ft / Elev 246.50 ft
Groundwater at end of drilling: 5.30 ft / Elev 246.70 ft

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	N-Value	LAB RESULTS/ REMARKS
0–5.1	Medium-stiff, dark-brown, SILT (ML), trace sand, moist (4-inch-thick root zone) (FILL)					
3–4.5		SS S-1	16	27	7	
5.1–12.5	Stiff, light-brown to brown SILT (ML), moist to wet					
5.5–7		SS S-2	18		15	
8–9.5		SS S-3	18	33	9	
10.5–12		SS S-4	18		9	
12.5–18.0	Very-stiff, red-brown with black streaks, gravelly SILT with sand (ML), moist, gravel and sand are weathered rock fragments					
15.5–17		SS S-5	18		31	
18.0–21.5	Dense, brown with black streaks, silty SAND with gravel (SM), moist, sand and gravel are weathered rock fragments					
20–21.5		SS S-6	18	50	42	Passing No.200 = 25.4%

Exploration completed at 21.5 feet bgs.
Groundwater first observed at 5.5 feet bgs and settled at 5.3 feet bgs after completion.

SPT completed using two wraps with a cathead.

Operator: Dan J. Fischer Excavating
Equipment: Trailer Mounted Drill Rig **Rig Number:** NA
Drilling Method: 4 3/8" Solid Stem Auger

Logged By: Ruslan P.
Checked By: Jessica P.

Approximate Location Coordinates:
Lat: NM Long: NM

Remarks:

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10240 SW Nimbus Ave, Suite L6
Portland, OR 97223
Telephone: (503) 616-9419

Project No: Dhaliwal-1-01

BORING LOG B-2

Project: Fairview Apartments
Location: 2110 Strong Road SE, Salem, OR 97302
Client: Studio 3 Architecture

Date Started: 3/28/24
Date Completed: 3/28/24

Approximate Ground Elevation: 250ft
Groundwater first observed: 9.20 ft / Elev 240.80 ft
Groundwater at end of drilling: 4.20 ft / Elev 245.80 ft

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	N-Value	LAB RESULTS/ REMARKS
0–4.5		Soft, dark-brown, SILT (ML), trace sand, moist (3-inch-thick root zone) (FILL)	SS S-1	8		3	
		Grades to stiff at 2.5 feet bgs	SS S-2	16		15	
4.5–14.0		Stiff, light-brown to brown, SILT (ML), moist to wet	SS S-3	16		8	
			SS S-4	18	34	9	
			SS S-5	18		9	
14.0–18.0		Very-stiff, red-brown with black streaks, silty SAND with gravel (SM), moist, gravel and sand are weathered rock fragments					
		Grades to more gravel at 15.5 feet bgs	SS S-6	18	36	32	
18.0–21.0		Very-dense, dark-gray with orange streaks, GRAVEL with sand and silt (GP-GM), moist, sand and gravel are weathered rock fragments					
21.0			SS S-7	12		50 for 4"	

Exploration completed at 21.0 feet bgs.
Groundwater first observed at 9.2 feet and settled at 4.2 feet bgs after completion.

SPT completed using two wraps with a cathead.

Operator: Dan J. Fischer Excavating
Equipment: Trailer Mounted Drill Rig **Rig Number:** NA
Drilling Method: 4 3/8" Solid Stem Auger

Logged By: Ruslan P.
Checked By: Jessica P.

Approximate Location Coordinates:
Lat: NM Long: NM

Remarks:

CENTRAL Geotechnical Services
Central Geotechnical Services
10240 SW Nimbus Ave, Suite L6
Portland, OR 97223
Telephone: (503) 616-9419

Project No: Dhaliwal-1-01

BORING LOG B-3

Project: Fairview Apartments
Location: 2110 Strong Road SE, Salem, OR 97302
Client: Studio 3 Architecture

Date Started: 3/28/24
Date Completed: 3/28/24

Approximate Ground Elevation: 255ft
Groundwater first observed: 7.40 ft / Elev 247.60 ft
Groundwater at end of drilling: 7.40 ft / Elev 247.60 ft

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	N-Value	LAB RESULTS/ REMARKS
0–5.8		Medium-stiff, light-brown to dark-brown, SILT (ML), trace sand, gravel and organics (woody debris), moist, gravel is angular (3-inch-thick root zone) (FILL) Grades to no organics at 1 foot bgs	SS S-1	11		5	
		Grades to stiff at 2.5 feet bgs	SS S-2	16		10	
			SS S-3	16		12	
5.8–13.5		Stiff, light-brown to brown,SILT (MH), moist to wet	SS S-4	18	32	11	LL = 52 PL = 30 PI = 22
			SS S-5	18	35	11	
13.5–18.0		Stiff, red-brown with black streaks, SILT with sand (ML), moist to wet, sand is weathered rock fragments	SS S-6	18		12	
18.0–21.0		Medium-dense, red-brown with black streaks, SAND with silt and gravel (SM), moist, gravel and sand are weathered rock fragments	SS S-7	11		16	

Exploration completed at 21.0 feet bgs.
Groundwater first observed and settled at 7.4 feet bgs after completion.

SPT completed using two wraps with a cathead.

Operator: Dan J. Fischer Excavating
Equipment: Trailer Mounted Drill Rig **Rig Number:** NA
Drilling Method: 4 3/8" Solid Stem Auger

Logged By: Ruslan P.
Checked By: Jessica P.

Approximate Location Coordinates:
Lat: NM Long: NM

Remarks:

CENTRAL Geotechnical Services
Central Geotechnical Services
10240 SW Nimbus Ave, Suite L6
Portland, OR 97223
Telephone: (503) 616-9419

Project No:
Dhaliwal-1-01

BORING LOG B-4

Project: Fairview Apartments
Location: 2110 Strong Road SE, Salem, OR 97302
Client: Studio 3 Architecture

Date Started: 3/28/24
Date Completed: 3/28/24

Approximate Ground Elevation: 253ft
Groundwater first observed: 5.50 ft / Elev 247.50 ft
Groundwater at end of drilling: 4.70 ft / Elev 248.30 ft

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	N-Value	LAB RESULTS/ REMARKS
0–5.1		Soft, dark-brown, SILT (ML), trace sand and gravel, moist, gravel is angular (4-inch-thick root zone) (FILL)	SS S-1	9		3	
		Grades to stiff below 2.5 feet bgs	SS S-2	14		10	
5.1–14.0		Stiff, light-brown to brown, SILT (MH), trace sand, moist to wet	SS S-3	18	30	10	
			SS S-4	0		9	
			SS S-5	18		10	
14.0–16.5		Stiff, red-brown with black streaks, gravelly SILT with sand (ML), moist, gravel and sand are weathered rock fragments	SS S-6	18		17	

Exploration completed at 16.5 feet bgs.
Groundwater first observed at 5.5 feet bgs and settled at 4.7 feet bgs after completion.

SPT completed using two wraps with a cathead.

Operator: Dan J. Fischer Excavating
Equipment: Trailer Mounted Drill Rig **Rig Number:** NA
Drilling Method: 4 3/8" Solid Stem Auger

Logged By: Ruslan P.
Checked By: Jessica P.

Approximate Location Coordinates:
Lat: NM Long: NM

Remarks:

Central Geotechnical Services
10240 SW Nimbus Ave, Suite L6
Portland, OR 97223
Telephone: (503) 616-9419

Project No:
Dhaliwal-1-01

BORING LOG B-5

Project: Fairview Apartments
Location: 2110 Strong Road SE, Salem, OR 97302
Client: Studio 3 Architecture

Date Started: 3/28/24
Date Completed: 3/28/24

Approximate Ground Elevation: 260ft
Groundwater first observed: ---
Groundwater at end of drilling: ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	N-Value	LAB RESULTS/ REMARKS
0–2.0	Medium-stiff, dark-brown to dark-gray, SILT (ML), trace sand and gravel, moist, gravel is angular (3-inch-thick root zone) (FILL)	SS S-1	10		5	
2.0–3.0	1-foot layer of medium angular GRAVEL					
3.0–7.6	Stiff, dark-brown to dark-gray, SILT (ML), trace sand and gravel, moist, gravel is angular (FILL)	SS S-2	0		14	
		SS S-3	17	25	12	
7.6–13.0	Medium-stiff, light-brown to brown, SILT (MH), moist to wet	SS S-4	18		10	
	Grades to stiff at 10.0 feet bgs	SS S-5	18		15	
13.0–17.5	Dense, red-brown with orange and brown streaks, SAND with silt and gravel (SM), moist, gravel and sand are weathered rock fragments	SS S-6	18		43	
17.5–20.5	Dense, dark-brown with black streaks, GRAVEL with silt (GP-GM), moist, sand and gravel are weathered rock fragments	SS S-7	18		42	

Exploration completed at 20.5 feet bgs.
No groundwater observed.

SPT completed using two wraps with a cathead.

Operator: Dan J. Fischer Excavating
Equipment: Trailer Mounted Drill Rig **Rig Number:** NA
Drilling Method: 4 3/8" Solid Stem Auger

Logged By: Ruslan P.
Checked By: Jessica P.

Approximate Location Coordinates:
Lat: NM Long: NM

Remarks:

BORING TEMPLATE V05.07.24 - CGS BORING LOG.GDT - 5/9/24 10:09 - C:\USERS\CGSUSER\CENTRAL GEOTECHNICAL SERVICES\CGS - PROJECTS\A-H\DHALIWAL\DHALIWAL-1\DHALIWAL-1-01\FIELD EXPLORATION\2_FIELD AND DRAFT LOGS\DHALIWAL-1

CENTRAL GEOTECHNICAL SERVICES
Central Geotechnical Services
10240 SW Nimbus Ave, Suite L6
Portland, OR 97223
Telephone: (503) 616-9419

Project No: Dhaliwal-1-01

INFILTRATION LOG INF-1

Project: Fairview Apartments
Location: 2110 Strong Road SE, Salem, OR 97302
Client: Studio 3 Architecture

Date Started: 3/28/24
Date Completed: 3/28/24

Approximate Ground Elevation: 250ft
Groundwater first observed: 5.25 ft / Elev 244.75 ft
Groundwater at end of drilling: 5.00 ft / Elev 245.00 ft

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	LAB RESULTS/ REMARKS
0–3.5		Medium-stiff, light-brown to dark-brown SILT (ML), trace sand and gravel, moist (4-inch-thick root zone) (FILL)		
3.5–5.5		Stiff, light-brown to brown SILT (ML), moist to wet	GS S-1	

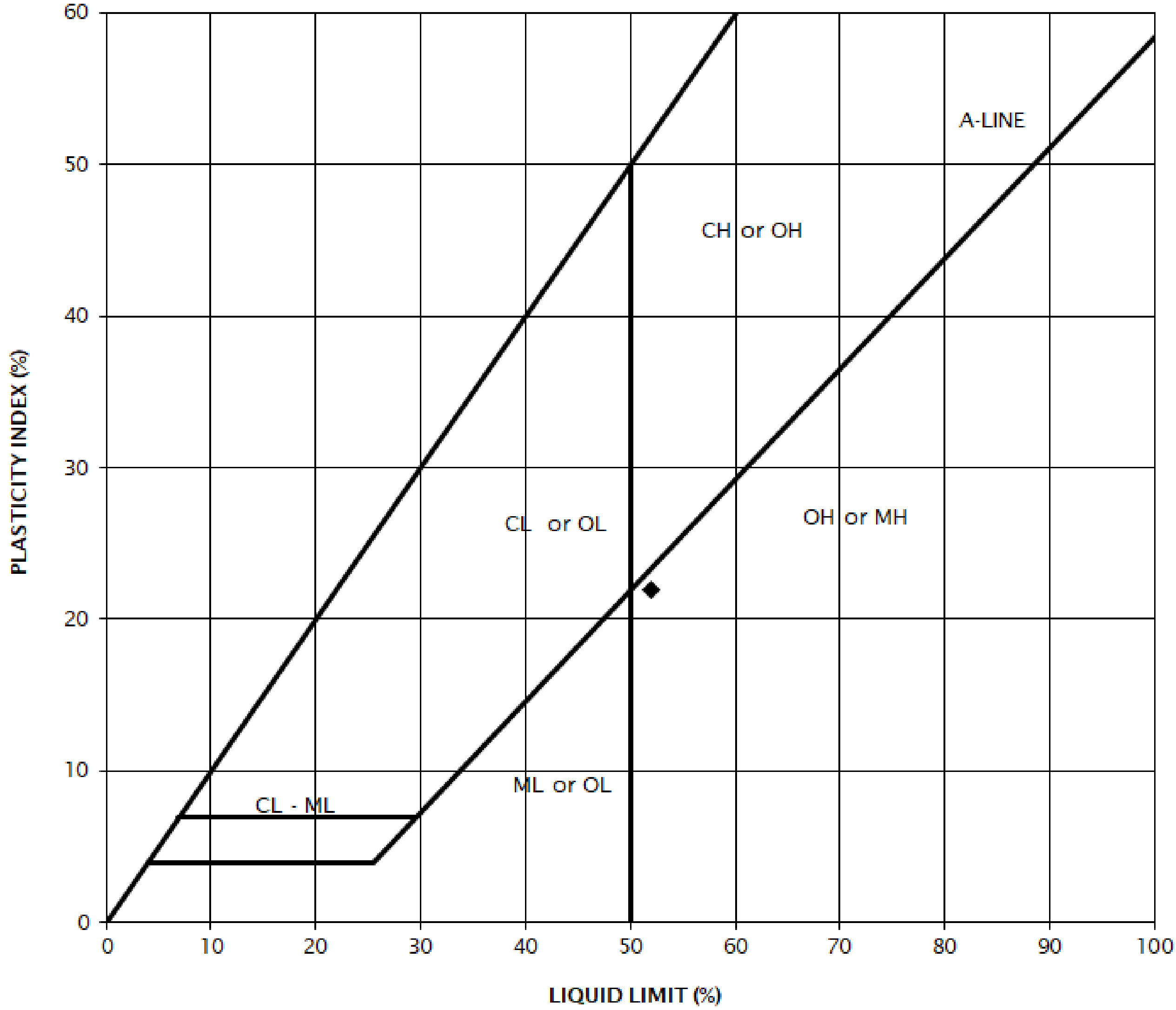
Exploration completed at 5.5 feet bgs.
Groundwater was observed after pipe embedment.

Operator: Dan J. Fischer Excavating
Equipment: Trailer Mounted Drill Rig **Rig Number:** NA
Drilling Method: 4 3/8" Solid Stem Auger

Logged By: Ruslan P.
Checked By: Jessica P.

Approximate Location Coordinates:
Lat: NM Long: NM

Remarks:



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	LIQUID LIMIT (PERCENT)	PLASTICITY INDEX (PERCENT)	SOIL DESCRIPTION
◆	B-3	7.5	32	52	22	brown SILT (MH), trace sand

CENTRAL GEOTECHNICAL SERVICES

Atterberg Limits Test Results	
Dhaliwal-1-01	May 2024
Proposed Fairview Apartments	Figure A-8

CENTRAL
GEOTECHNICAL SERVICES

APPENDIX B: Geologic Hazard Assessment

CENTRAL GEOTECHNICAL SERVICES

Memorandum

10240 SW Nimbus Avenue Suite L6, Portland, Oregon 97223 www.centralgeotech.com

To:	Inderjet Dhaliwal
From:	Paul Crenna, CEG Julio Vela, Ph.D., PE, GE
CGS Project Number:	Dhaliwal-1-01
Date:	May 17, 2024
Subject:	Geologic Hazard Assessment Fairview Apartments 2110 Strong Road SE Salem, Oregon

GEOLOGIC HAZARD ASSESSMENT

As requested, Central Geotechnical Services (CGS) is pleased to submit this memorandum presenting the results of the geologic hazard assessment (GHA) conducted in accordance with the requirements of the City of Salem Revised Code (SRC) Chapter 810 for the proposed Fairview Apartments Development located at 2110 Strong Road SE in Salem, Oregon.

The GHA portion of our scope of work included:

1. Review of published Geologic Hazard Maps and publicly available geologic and geotechnical information for the site vicinity.
2. Geologic field reconnaissance of the site to observe ground surface conditions.
3. Determination of the Total Landslide Hazard Risk using the Graduated Response Tables in Section 810.025.
4. Preparation of this letter report summarizing our conclusions and recommendations with respect to site geologic hazards relative to Section 810.030 (a).

We conducted this Geologic Hazard Assessment in conjunction with a geotechnical investigation of the site. This Memorandum is intended to be an addendum appended to the geotechnical report for the project. Based on information provided to us, we understand that project development will include a multi-family apartment building that is 3- to 4-stories tall incorporating wood-frame construction. Development will also include associated improvements such as underground utilities, paved driveways and parking, and sidewalks.

DESKTOP REVIEW

We completed a desktop review for the site prior to our field reconnaissance visit. Our desktop review included a Total Landslide Hazard Risk Assessment in accordance with Salem Revised Code Section 810.025 based on review of the following:

1. Oregon Department of Geology and Mineral Industries (DOGAMI) maps including:

    a. Earthquake Induced Landslide Susceptibility Interpretive Map Series IMS-17 and IMS-18.

    b. Water Induced Landslide Susceptibility Interpretive Map Series IMS-5 and IMS-6.

    c. Active/Inactive Slide Hazard Areas Map (DOGAMI Open File Report 0-77-4 (Map Plates not available).

    d. Excessive Slope Areas within Marion County (map).

    e. DOGAMI Statewide Landslide Information Database for Oregon (SLIDO 4.4)

Landslide Hazard Risk Assessment Per Section 810.030(a)

Based on our desktop review of the criteria in Salem Revised Code 810.030 (a)

1. Table 810-1A Step 1 (Earthquake-Induced Landslide Susceptibility Ratings) requires review of IMS-17 and IMS-18. The subject site is outside of the mapped areas of IMS-17 and IMS-18 **(0 points)**.
2. Table 810-1B Step 2 (Water Induced Landslide Hazards Susceptibility Ratings) requires review of IMS-5, IMS-6, and DOGAMI 0-77-4. The subject site is not within the mapped area of IMS-5 or IMS-6, and slopes are less than 15 percent grade **(0 points)**.
3. Table 810-1C Step 3 (Activity Ratings) – We assume that the project will include excavation and/or fill greater than 2 feet or 25 cubic yards, and a multi-family development **(5 points).**
4. Table 810-1D Step 4 (Cumulative Score) totals the cumulative score for the subject site. As we interpret the first three steps above, the cumulative score for the site is as follows: Step 1 (0 points) plus Step 2 (0 points) plus Step 3 (5 points) **Total = 5 points.**

Per Table 810-1E Step 5 (Total Landslide Hazard Risk) a cumulative score of 5 to 8 points falls into category B – Moderate Landslide Risk that requires a Geologic Hazard Assessment (GHA) and Geotechnical Engineering Report may be required. A summary of the GHA follows.

Landslide Hazard Mapping – SLIDO Review

Landslide inventory and hazard mapping in Oregon has been compiled by DOGAMI in their SLIDO 4.5, database, which was last updated in April of 2024. The inventory shows no mapped landslides in the immediate vicinity of the site. Regional mapping of landslide susceptibility by DOGAMI shows the site to be located in a low landslide susceptibility area.

Review of LiDAR Imagery

We reviewed Light Detection and Ranging (LiDAR) bare earth digital elevation imagery of the site from SLIDO 4.5. Landforms at the site are generally gentle-sloping, smooth, and uniform, consistent with stable slope conditions. Short steeper slopes are present in localized areas where fill and shallow excavations have been made. We did not observe any obvious landforms associated with possible prior landsliding within the site boundaries.

Regional Geologic Mapping

Regional geologic mapping shows the site vicinity to be underlain by older alluvium consisting of poorly consolidated clay, silt, sand and gravel filling the Willamette River Basin (Walker and Duncan, 1989; O'Conner et. al. 2001).

SITE RECONNAISSANCE

CGS staff geologist, Ruslan Pavlenko, conducted a site reconnaissance on March 28, 2024, during which we traversed the site to look for surface indications of geologic hazards, including past slope instability, marginally stable fill slopes and poor drainage conditions. Site vegetation is variable, but largely consists of low grasses.

The site is an approximate 150-foot-wide and 300-foot-long area surrounded by existing development with Strong Road on the north and Linburg Road on the east. Site grades are gently sloping at less than 15% grade. There is an approximate 1H:1V fill slope about 4 feet tall on the margin of the site.

At the time of our visit, we observed no obvious indications of slope instability or previous landsliding activity; however, the natural ground surface was obscured by fill.

GEOLOGIC HAZARD CONCLUSIONS

Based on our geologic hazard assessment as presented herein, we consider the site to have a low susceptibility to landsliding; however, the site is underlain by areas of undocumented fill and is located in an area of possible shallow groundwater. Per the Salem revised code, a geotechnical report should be prepared to investigate subsurface soil and groundwater conditions with respect to geologic hazards, and provide recommendations for site grading and remedial measures, if necessary. This assessment memorandum should be considered a part of the geotechnical report being concurrently prepared for the proposed project at the site.

LIMITATIONS

We have prepared this GHA memorandum for the exclusive use of Inderjet Dhaliwal and their authorized parties for the project specifically identified in this letter only. Within the limitations of scope, schedule and budget, this report was prepared in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology in this area at the time the report was prepared. No warranty, express or implied, is made. The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water, or groundwater at this site.

RENEWS 11-1-24

Julio Vela, PhD, PE, GE
Principal Engineer

Paul Crenna, CEG
Principal Engineering Geologist

REFERENCES

Oregon Department of Geology and Mineral Industries (DOGAMI) 2023. Statewide Landslide Information Database for Oregon, Version 4.5, April 12, 2024. Accessed at https://www.oregongeology.org/slido/ on May 14, 2024.

O'Connor et al., 2001, Quaternary Geologic Units in the Willamette Valley, Oregon; U.S. Geological Survey Professional Papper 1620, Plate 1, map scale 1:250,000.

Walker, G.W. and Duncan, R.A., 1989, Geologic Map of th Salem 1 by 2 degree Quadrangle, Western Oregon; U.S. Geological Survey Miscellaneous Investigations Series Map I-1893, map scale 1:250,000.