Geotechnical Engineering Report

Strong Heights Pavements, Geologic Assessment, and Infiltration Testing
Old Strong Road and Reed Road SE
Salem, Oregon

for
Ward Development, LLC

April 21, 2021

GeoEngineers
Earth Science + Technology

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File No. 24737-002-00

April 21, 2021

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GEOENGINEERS

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1.0 INTRODUCTION

GeoEngineers, Inc. (GeoEngineers) is pleased to submit this pavement recommendations, infiltration testing, and geologic assessment for a portion of the former Fairview Hospital and Training Center located south of Old Strong Road and west of Reed Road SE in Salem, Oregon. The location of the site is shown in the Vicinity Map, Figure 1.

The site is an approximately 4.5-acre portion of the site named Strong Heights Subdivision. The overall site property was historically owned by the state of Oregon as part of the campus of the former state hospital. GeoEngineers has worked on several areas of the overall site as part of current development, including fill placed in September 2019 at the north end of the Strong Heights site. Overall, site buildings and roadways have been demolished and the site partially graded over a several year process. The site is currently vacant of buildings or roadways. Based on discussions with Ward Development, development for Strong Heights will include individual lots and infrastructure that includes public roadways and utilities. The scope of work for this report includes geotechnical and geologic conclusions and recommendations for infrastructure development of roadways and overall site development. Recommendations for individual lot development are not included.

2.0 SCOPE OF SERVICES

The purpose of our services was to evaluate the existing slope, soil, and groundwater conditions as a basis for providing a geological assessment in general accordance with the requirements of the City of Salem Revised Code Chapter 810.030(a), as well as to evaluate soil and groundwater conditions as a basis for developing geotechnical engineering design recommendations for pavement recommendations and to provide in-situ infiltration rates. Our proposed scope of services included:

1. Reviewing information regarding slope, subsurface soil and groundwater conditions at the site based on review of selected geologic maps, and other geotechnical engineering related information on file in our office.
2. Conducting a site visit to assess the surficial slope, and general subsurface soil and groundwater conditions at the site. This included a reconnaissance of the existing site slopes and vegetation conditions, and an assessment of shallow subsurface conditions by advancing 11 test pit explorations and direct three cone penetration (DCP) tests. Test pits were advanced using a backhoe excavator from K&E Excavation provided on site by Ward Development.
3. Performing two infiltration tests conducted in accordance with the downhole test procedure outlined in “Division 004” of the City of Salem Department of Public Works Administrative Rules Design Standards (COSDS) at an approximate depth of 4 feet below ground surface (bgs). Infiltration tests were conducted at the south end of the site as noted on preliminary development plans.
4. Collecting samples at representative intervals from the explorations, observing groundwater conditions, and maintaining detailed logs in general accordance with ASTM International (ASTM) Standard Practices Test Method D 2488.
5. Performing laboratory tests on selected soil samples obtained from the explorations to evaluate pertinent engineering characteristics. Laboratory test results are included in the exploration logs in Appendix A-Field Explorations and Laboratory Testing.

6. Providing a summary geotechnical report, including a geological assessment of the site to address the following components:
    a. A general description of site topography, geology, and subsurface conditions.
    b. An opinion as to the existing general stability of the site and a geologic assessment, including a determination of level of landslide hazard.
    c. A summary of infiltration test results at the tested locations and a discussion of the adequacy of infiltration systems on the site.
    d. Recommendations for constructing asphaltic concrete (AC) pavements for the proposed roadways, including subgrade, drainage, base rock and pavement section. Our recommendations will be based on estimated traffic loads 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. Surface Conditions

The 4.5-acre portion for the Strong Heights site is a portion of the approximately 18-acre site bordered by Old Strong Road SE to the north, Salem City parkland to the east, and residential developments to the south and west. The overall site was once occupied by the former Fairview Hospital, which consisted of numerous buildings, building basements, access tunnels between buildings, AC access roads and AC parking lots. The buildings have been demolished, and the surface of the site has been developed over the last two years, including cleanup and partial re-use of demolition debris ranging from scattered brick, concrete, and metal fragments to large stockpiles of brick and soil materials. Building access tunnels and utility ducts have also been demolished and filled with structural fill.

A building used as part of Fairview Hospital was located on the north end of the Strong Heights site. The building was demolished and the basement slab removed and backfilled with locally available fill material in September and October 2019. GeoEngineers observed fill placement and compaction under a separate contract. Based on those observations, summarized in daily field reports, the fill in that area was placed and compacted as structural fill. At the time of our site reconnaissance, the ground surface was generally covered by rough field grass, and large, mature oak, fir and cedar trees in small stands or as individual, isolated trees.

The site topography is dominated by a gentle hilltop in the south center of the parcel. From the hill the slopes trend generally downward relatively gently to the south and east parcel boundaries. The west and north slopes, however, terminate in moderately steep gradients down to Old Strong Road SE to the north and the adjoining property to the west. Site elevations range from approximately 270 feet above mean sea level (MSL) at the top of the hill to approximately 215-220 feet MSL along the northern and western boundaries.

In general, the hillside gradients range from almost level and gently undulatory to roughly 10 to 15 percent. These inclinations are typical of the southern and eastern sides of the hilltop. Along Old Strong Road SE, and near the western parcel boundaries, the gradients increase to between about 50 to 70 percent. However, these steep slopes are typically forested with mature conifers with some intermixed deciduous trees, and a thick forest understory of brush and grasses.

3.1.1. Geologic Hazard Observations

During our site reconnaissance, we observed the site for surface indications of geologic hazards, including past slope instability, marginally stable fill slopes and evidence of high groundwater.

The hilltop itself, the southern and eastern slopes, and the upper portion of the northern and western slopes descending from the hilltop, are planar to convex and regular. The trunks of the large trees within these portions of the site are vertical, indicating that the trees have not been displaced or tilted over a period of 80 years or more. Our subsurface investigation suggests that the soils within these relatively gentle slopes consist of a mantle of man-made fill presumably associated with grading during demolition of the former hospital over relatively undisturbed silty fine-grained flood sediments.

The steep north and west facing slopes are generally planar and typically thickly vegetated with mature, straight conifer trees. We did not observe surface indications of global instability along these steeper slopes, such as semicircular depressions in the ground surface, bulging at the base of the slopes, unusual drainage features such as seeps or springs, open ground cracking, leaning or bowed conifer trees.

3.2. Site Geology

The site is shown on Bela (1981) as largely underlain by Miocene-age Columbia River Basalt (CRB) below the hilltop, with Pleistocene-age “higher terrace deposit” sand and silt along the lower portions of the northern slopes. O’Conner and others (2001) suggest that instead most or all of the site as mantled by the “main body of Missoula Flood deposits.” Well logs on file at the Oregon Water Resources Department (OWRD) suggest that below these alluvial silts and sands the site is underlain by a mixture of coarse-grained flood deposits over the CRB bedrock mapped by Bela (1981).

The geologic mapping does not show faults crossing the site. Based on a review the United States Geologic Survey (USGS) quaternary fault and fold data base of the United States (USGS 2019), the nearest Quaternary fault to the site is the Owl Creek Fault, which is located approximately 21 miles southwest of the site.

3.3. Subsurface Conditions

We completed field explorations at the site on April 14, 2021. Our explorations included a total 11 test pits excavated to depths ranging between 4 feet and 10½ feet bgs at the locations shown in the Site Plan, Figure 2. A summary of our exploration methods as well as the test pit logs are contained in Appendix A.

Our investigation suggests that the lower, eastern portions of the site are underlain by medium stiff silt native soil, mantled in local areas by aggregate fill, relict concrete foundations, or asphalt pavement. Note that a portion of the lowermost eastern section of the site appears to have been an earlier right-of-way of Old Strong Road, and a roughly 1-foot-thick layer of coarse angular basalt aggregate base mixed with AC fragments should be anticipated along this alignment.

The western, higher portions of the site are also underlain by the native silt but are mantled by a variable mixture of fills, including:

- A surface layer of loose to dense aggregate and crushed concrete debris. This material mantles all but the furthest northern and eastern portions of the western hilltop. Along the south side of the upland portions of the site this fill is underlain by native silts similar to those described for the eastern site areas, above.

- To the north we encountered medium dense silty sand and silty gravel and medium stiff sandy silt soil fills below the aggregate-concrete materials. This soil fill typically contained a variety of man-made debris, which was commonly encountered as individual concrete fragments, pieces of metal or concrete pipe, or geotextile fabric. However, one localized concentration of debris was encountered (in TP-7), which contained a mass of large concrete fragments to 2 foot dimension, metal pipe, aluminum ducting, brick, and fabric. The soil fill depths ranged from as little as 20 inches bgs to as deep as 9½ feet bgs. This appears to be an isolated area at the margins of the fill placed after basement demolition and fill placement.
- A roughly 2-foot-thick layer of organic soil fill was encountered underlying the soil fill and above the native silt in TP-10 and TP-11.

We did not encounter groundwater in the test pits. Well logs on file with OWRD suggest that permanent groundwater is approximately 15 feet bgs. We anticipate that groundwater levels at the site will fluctuate depending on site utilization, precipitation or other factors.

4.0 GEOLOGIC HAZARD MAPPING AND SITE STABILITY ASSESSMENT

We evaluated the qualitative stability of the existing conditions of the site by researching published hazard mapping and performing a site reconnaissance to observe surficial indications of past or existing mass wasting (concave or convex bulges in slopes, cracking in the ground, unusual drainage conditions (anomalous seeps and springs), or unusual or disturbed soil or vegetation conditions. Observations noted during our geologic site reconnaissance are described above in Section 3.1.1.

4.1. Hazard Mapping

We reviewed published earthquake-induced landslide hazards maps and water-induced landslide hazard maps in the area, including Department of Geology and Mineral Industries (DOGAMI) Interpretive Map Series IMS-17 (Hofmeister and Wang 2000) and IMS-6 (Harvey and Peterson 1998). The site falls outside the southern or eastern boundaries of the publications.

Landslide mapping and landslide hazards for the site are compiled by the DOGAMI Statewide Landslide Information Layer for Oregon (SLIDO). This publication shows no mapped or historic landslides within the site and maps the gentle eastern slopes of the hilltop as susceptible to low to moderate landslide hazard. The western portion of the northern slope is mapped “high – landsliding likely,” apparently based largely on the steeper slope gradient.

Seismic hazard to the site is mapped by DOGAMI Geological Map Series GMS-105 (Wang and Leonard 1996). This map series includes hazard maps for overall earthquake hazard, liquefaction susceptibility, amplification susceptibility (amplification of peak rock accelerations associated with earthquake shaking) and landslide susceptibility associated with earthquake shaking.

The liquefaction hazard to the hilltop that forms the bulk of the site is mapped as Category 0, lowest of a six-category scale and described as “No susceptibility, with possible exceptions in small, localized areas.” The northern slope along Old Strong Road SE is mapped as Category 2, “6-12’ estimate thickness of liquefiable material." However, well logs for the Fairview Hospital site identifies the groundwater as approximately 15 feet bgs at the contact between the surficial silt and underlying gravel, suggesting that permanent groundwater is typically below the base of the potentially liquefiable flood sediment.

Peak ground amplification hazard to the hilltop is mapped as Category 1, second lowest on a six-category scale, ">1.2 amplification factor for peak rock accelerations." The northern slope is, again, rated higher than the gentler slopes, in this case as Category 3, "≥1.4-1.6 2 amplification factor for peak rock accelerations," third highest on the peak ground amplification hazard scale.

4.1.1. Mapping Summary

The landslide susceptibility hazard to the site is shown as Category 5, the highest of six categories; "high susceptibility of landsliding in areas of existing landslides." This categorization leads, in turn, to the overall seismic hazard mapping of the site within Zone A, the highest hazard zone of the four-category scale.

This landslide susceptibility and overall hazard mapping is problematic. While landslide deposits and active landslides are not shown on O'Conner and others (2001), the mapping of Bela (1981) does identify landslide scarps and areas of "landslide topography" (landslide-affected areas, including landslide debris); the subject site is not mapped as a landslide or within landslide topography by Bela (1981).

4.1.2. Conclusion of Mapping Summary and Site Observations

As discussed above, no existing, active, or recent landslides are mapped by SLIDO (2019). The Light Detection and Ranging (LiDAR) bare earth model does not show indication typical of landsliding such as a steep, concave scarp, bulging or "bull-nose" lower slopes, internal hummocky or bench-and-scarp block topography. Instead, the site slopes appear on the LiDAR mapping as generally gentle and planar to convex and consistent with the mapping of Bela (1981).

Based on our observations, the published geologic mapping, and the LiDAR topography, it is our opinion that although the possibility cannot be ruled out based on state research and mapping, the likelihood of the site being within an existing landslide as mapped by Wang and Leonard (1996) is very low. Mapping of this site may have been done using general topographic observations for the 1996 report and not based on direct observations.

4.2. Flood Mapping

Flood mapping we reviewed (FEMA 2019) shows the site located in "Zone X" indicating the site lies outside the area of the 500- and 100-year floods. Based on the site topography and vegetation, we concur with the flood hazard mapping.

4.3. LiDAR Hillshade Model Review

We reviewed a LiDAR bare earth hillshade model of the site on the DOGAMI web-based LiDAR viewer (DOGAMI 2019). As noted above, we did not see indications of deep-seated landsliding within the site boundaries in the hillshade model.

4.4. Existing Site Stability

Based on our site reconnaissance, most of the site occupies relatively gentle slopes with a low risk of slope instability. The north-facing slope is generally inclined at 2H:1V (horizontal to vertical) and showed no indications of active or recent instability in the past. Portions of this slope, however, may contain isolated pockets of man-made debris which are not as stable as the overall slope, as is discussed in Section 5.0, below.

5.0 GEOLOGIC HAZARD CONCLUSIONS

Based on our geologic hazard evaluation as presented herein, the primary geologic hazards at the site consist of the potential for future development to adversely affect the stability of portions of the north slope of the site that may be underlain by uncontrolled debris fills.

Based on our observations, future development on or near the north-facing slopes could adversely affect the stability of these slopes if there are uncontrolled debris materials concentrated in the area. Our observations during site grading suggest that this will be unlikely to affect much of the slope, but the concentration of demolition debris encountered in TP-7 means that this cannot be ruled out entirely.

It is our opinion that future development, including placement of structures or raising site grades by placement of fill, should not be located near the tops of north-facing slope at the site. Offsets of as much as 1 to 1.5 times the total height of the slope for fill or surface slabs, or to the bottom of supporting elements (footings) from the tops of slopes may be required depending on the type of development but should be based on development-specific geotechnical evaluations that include subsurface explorations. In addition, surface and subsurface water management plans should be developed to route surface water away from these slopes, and to limit infiltration of collected surface water or septic field fluids near the crests of these slopes.

No indications of active or recent slope instability were observed during our site reconnaissance. However, due to the hazard mapping of Wang and Leonard (1996), site development may require conducting a comprehensive geotechnical investigation of the site when project-specific development plans are developed to provide geotechnical engineering recommendations for the site if development is planned closer than 1.5 times the total height of the slope from the tops of slopes at any particular point on the property. A geotechnical study should evaluate the risk of development adversely affecting the geologic hazards identified in this evaluation (existing slopes) and provide recommendations to reduce or mitigate the risk of geologic hazards affecting proposed development.

6.0 SITE PREPARATION

6.1. Demolition

Because of prior development, on-site buried utilities or structural elements may still be present below the surface. Existing utilities in the proposed construction area should be identified prior to excavation. Live utility lines identified beneath proposed pavements that are too shallow or unacceptable to remain beneath finished roadways should be relocated. Abandoned utility lines beneath structures should be completely removed or filled with grout. Soft or loose soil encountered in utility line excavations should be removed and replaced with structural fill where it is located within structural areas.

Voids resulting from removal of utility or structural elements such as tunnels or structural remnants should be backfilled with compacted structural fill, as discussed in Section 6.9 of this report. The bottom of such excavations should be excavated to expose a firm subgrade before filling and the sides sloped at a minimum of 1H:1V to allow for more uniform compaction at the edges of the excavations.

Materials generated during demolition of existing improvements 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 periods of wet weather.

6.2. Subgrade Preparation

In order to provide consistent pavement section support, pavement construction specifications shall require pavement subgrade to be scarified at least 1 foot and re-compacted to 95 percent of the maximum dry density (MDD), as determined by American Association of State Highway and Transportation Officials (AASHTO) T-180 or ASTM Test Method D 1557 (modified proctor). Compacting the upper 1 foot of the subgrade is required as a part of the final design sections provided below. If existing subgrade is not improved, a thicker rock section will be required for support of prescribed traffic levels.

In addition, several areas of the site are surfaced with oversized concrete debris material and should be removed to at least 12 inches below bottom of asphalt elevation to avoid concentrated hard transition zones within the subgrade that might result in surface cracks in paved areas. The 12 inches should be backfilled with compacted crushed rock consistent with the asphalt section base section recommended below.

The upper fine-grained soil materials anticipated at subgrade elevation are susceptible to disturbance when oversaturated. If construction timing or other limitations do not allow for scarification and recompaction, we recommend disturbed material be removed and replaced with compacted structural fill. We recommend protecting the subgrade by following the recommendations provided in Section 6.7 of this report.

6.2.1. Subgrade Improvement for the Disturbed Zone

Portions of the overall site are mantled with an upper “disturbed zone” or upper tilled soils from previous agriculture land use or other site uses. These areas are generally comprised of moist, loose or previously loosened silt with organics from roots or other plantings and extends to a depth of approximately 18 inches bgs.

The previously disturbed or tilled soils are not consistently compacted across the site and in most areas would be unreliable without improvement to support pavements. Therefore, if the existing upper layer of soil remains in place to receive site fills during mass grading, it should be either: (1) scarified, moisture-conditioned and compacted in-place during the dry season; or (2) removed and replaced with Imported Select Structural Fill if construction occurs during the wet season, or at other times when the material cannot be compacted in place. If the tilled zone is cut away (cuts extend below the tilled zone) as a part of mass grading, recompaction or removal of in-place undisturbed soils is not required.

Because it is generally in a more loosened state, the tilled zone soil will provide marginal to poor support for construction equipment. Wet weather construction practices will be required when improving the tilled zone, except during the dry summer months.

Subgrade improvement for the tilled zone can be accomplished by removing and replacing or scarifying and re-compacting the tilled zone. Scarification is typically performed by ripping with agricultural discs and aerating the soils to dry them during dry weather periods. Considerable soil processing, including moisture conditioning (primarily drying to reduce the existing moisture content), should be expected to adequately compact the tilled zone. If the soil cannot be properly moisture conditioned (dried), the subgrade should be removed and replaced with Imported Select Structural Fill. If the project specifications allow, the tilled zone can be cement amended as described Section 6.8 of this report. Cement amendment is typically performed to depths of 12 to 18 inches. When performed in silty soils, such as those at the site, multiple tilling and application passes may be required to adequately blend and amend the soils.

6.3. Subgrade Evaluation

Prior to placement of base rock and AC section, a member of our geotechnical staff should observe the prepared subgrades to determine if the subgrades have been prepared in accordance with our recommendations above. Where prepared areas are accessible to equipment, a field representative should observe a proof-roll of the subgrade under load of a fully loaded dump truck or similar heavy, rubber-tire construction equipment in order to identify soft, loose, or unsuitable subgrade areas. If exposed subgrade appears wet or too soft to support proof-rolling equipment, the area should be evaluated by probing.

6.4. Excavation

Based on the material encountered in our subsurface explorations, it is our opinion that conventional earthmoving equipment in proper working condition should be capable of making necessary general excavations.

The earthwork contractor should be responsible for reviewing this report, including the boring logs, providing their own assessments, and providing equipment and methods needed to excavate the site soils while protecting subgrades.

6.5. Dewatering

Based on test pits conducted along the alignments of Lindburg Road and Strong Road, perched groundwater should be expected to be encountered in utility excavations, especially in lower elevations of the site. In general, we do not anticipate excavations to extend below the areal ground water elevation, but locally perched and persistent water may be encountered even in relatively shallow excavations. Sump pumps are expected to adequately address perched water if encountered at shallower depths. Alternatively, if excavations are performed during the dry season, perched water will be less likely to be present.

In addition to groundwater seepage, surface water or subsurface water trapped in former utilities or filled-in features may flow into the excavations and can be problematic. Provisions for inflow water control during earthwork and excavations should be included in the project plans and should be considered prior to commencing earthwork.

6.6. Shoring

All trench excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations. In our opinion, native soils are generally OSHA Type B. Excavations deeper than 4 feet should be shored or laid back at an inclination of 1.25H:1V or flatter if workers are required to enter. Excavations made to construct footings or other structural elements should be laid back or shored at the surface as necessary to prevent soil from falling into excavations.

Shoring for trenches less than 6 feet deep that are above the effects of groundwater should be possible with a conventional box system. Moderate sloughing should be expected outside the box. Shoring deeper than 6 feet or below the groundwater table should be designed by a registered engineer before installation. Further, the shoring design engineer should be provided with a copy of this report.

The site earthwork contractor should expect that unsupported cut slopes will likely experience some sloughing and raveling if exposed to water. Plastic sheeting, placed over the exposed slope and directing water away from the slope, will reduce the potential for sloughing and erosion of cut slopes during wet weather.

In our opinion, the contractor will be in the best position to observe subsurface conditions continuously throughout the construction process and to respond to the soil and groundwater conditions. Construction site safety is generally the sole responsibility of the contractor, who also is solely responsible for the means, methods and sequencing of the construction operations and choices regarding excavations and shoring. Under no circumstances should the information provided by GeoEngineers be interpreted to mean that GeoEngineers is assuming responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred.

6.7. Wet Weather Construction

The fine-grained soils at the site surface are highly susceptible to disturbance from traffic when wet. Wet weather construction practices will be necessary if work is performed during periods of wet weather. If site grading will occur during wet weather conditions, it will be necessary to use track-mounted equipment, load removed material into trucks supported on existing gravel surfacing or haul roads, use gravel working pads and employ other methods to reduce ground disturbance. The contractor should be responsible to protect the subgrade during construction.

During wet weather, some of the exposed soils could become muddy and unstable. If so affected, we recommend that:

- 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.
- The site soils should not be left uncompacted and exposed to moisture. Sealing the surficial soils by rolling with a smooth-drum roller prior to periods of precipitation will reduce the extent to which these soils become wet or unstable.
- Construction activities should be scheduled so that the length of time that soils are 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 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 Section 7.0 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.

6.8. Soil Amendment with Cement

The project site is relatively small and may not be accessible to large equipment necessary to cement amend wet soils. We have included this section in case site conditions can be modified for use of cement amended subgrade. Cement amending is often used as an alternative to using Imported Select Structural Fill material for wet weather structural fill or to reduce the pavement section thickness. An experienced contractor may be able to amend the on-site soil with portland cement concrete (PCC) to obtain suitable support properties. It is often less costly to amend on-site soils than to remove and replace soft soils with imported granular materials. Alternate pavement design sections below consider a 12-inch depth of improvement. Single pass tilling depths for cement amendment equipment is typically 18 inches or less. Multiple tilling passes may be required to adequately blend in the cement with the soils and to sufficiently process the soils in some areas with fine-grained soils. It may also be necessary to place the recommended cement quantities in multiple passes between tilling passes, which requires intermediate compaction.

The contractor should be responsible for selecting the means and methods to construct the amended soil without disturbing exposed subgrades. We recommend low ground-pressure (such as balloon-tired) cement spreading equipment be required. We have observed other methods used for spreading that have resulted in significant site disturbance and high remedial costs. For example, we have observed amendment efforts to use a spreader truck equipped with road tires pulled by track-mounted equipment that resulted in significant disturbance to the work area and required re-working large areas of cement-amended product at additional expense.

Areas of standing water, or areas where traffic patterns are concentrated and disturbing the subgrade, will also create a need for higher amounts of cement to be applied and additional tilling for better mixing and cement hydration prior to final compaction.

Successful use of soil amendment depends on the use of correct mixing techniques, the soil moisture content at the time of amendment and amendment quantities. Specific recommendations based on exposed site conditions for soil amending can be provided, if necessary. However, for preliminary planning purposes, it may be assumed that a minimum of 5 percent cement (by dry weight, assuming a unit weight of 100 pounds per cubic foot [pcf]) will be sufficient for improving on-site soils. Treatment depths of 12 to 16 inches are typical (assuming a seven-day unconfined compressive strength of at least 80 pounds per square inch [psi]), although they may be adjusted in the field depending on site conditions. Soil amending should be conducted in accordance with the specifications provided in Oregon Structural Specialty Code (OSSC) 00344 (Treated Subgrade).

We recommend a target strength for cement-amended soils of 80 psi. The amount of cement used to achieve this target generally varies with moisture content and soil type. It is difficult to predict field performance of soil-to-cement amendment due to variability in soil response and we recommend laboratory testing to confirm expectations. However, for preliminary design purposes, 4 to 5 percent cement by weight of dry soil can generally be used when the soil moisture content does not exceed approximately 20 percent. If the soil moisture content is in the range of 20 to 35 percent, 5 to 7 percent by weight of dry soil is recommended. The amount of cement added to the soil should be adjusted based on field observations and performance.

PCC-amended soil is hard and has low permeability; therefore, this soil does not drain well nor is it suitable for planting. Future landscape areas should not be cement amended, if practical, or accommodations should be planned for drainage and planting. Cement amendment should not be used if runoff during construction cannot be directed away from adjacent low-lying wet areas, active waterways and drainage paths.

When used for constructing pavement, staging or haul road subgrades, the amended surface should be protected from abrasion by placing a minimum 4-inch thickness of base rock material (Aggregate Base/Aggregate Subbase). To prevent strength loss during curing, cement-amended soil should be allowed to cure for a minimum of four days prior to placing the base rock. The base rock typically becomes contaminated with soil during construction. Contaminated base rock should be removed and replaced with clean base rock in pavement areas to meet the required thickness(es) in Section 7.0 to this report.

It is not possible to amend soil during heavy or continuous rainfall. Work should be completed during suitable weather conditions.

6.9. Structural Fill and Backfill

Structural areas include areas beneath pavements and any other areas intended to support structures or within the influence zone of structures.

Material used for structural fill should be free of debris, organic contaminants and rock fragments larger than 4 inches. The suitability of material for use as structural fill will depend on the gradation and moisture content of the soil. As the amount of fines increase, soil becomes increasingly sensitive to small changes in moisture content and achieving the required degree of compaction becomes more difficult or impossible. The following paragraphs summarize our recommendations for fill and backfill.

6.9.1. On-Site Soils

On-site soils consist of a range of soils from silt with trace clay, silt with varying amounts of sand and silty gravel with sand. The on-site soil is suitable for use as structural fill provided it meets the requirements for structural fill. An experienced geotechnical engineer from GeoEngineers should determine the suitability of on-site soil encountered during earthwork activities for reuse as structural fill.

6.9.2. Imported Select Structural Fill

Select imported granular material may be used as structural fill. The imported material should consist of pit or quarry run rock, crushed rock, or crushed gravel and sand that is fairly well-graded between coarse and fine sizes (approximately 25 to 65 percent passing the U.S. No. 4 sieve). It should have less than 5 percent passing the U.S. No. 200 sieve and have a minimum of 75 percent fractured particles according to AASHTO TP-61.

6.9.3. Aggregate Base

Aggregate Base material located under pavements, and retaining wall backfill should consist of imported clean, durable, crushed angular rock. Such rock should be well-graded, have a maximum particle size of 1 inch, have less than 5 percent passing the U.S. No. 200 sieve (3 percent for retaining walls) and meet the gradation requirements in Table 1. In addition, aggregate base shall have a minimum of 75 percent fractured particles according to AASHTO TP-61 and a sand equivalent of not less than 30 percent based on AASHTO T-176.

TABLE 1. RECOMMENDED GRADATION FOR AGGREGATE BASE

Sieve Size	Percent Passing (by weight)
1 inch	100
½ inch	50 to 65
No. 4	40 to 60
No. 40	5 to 15
No. 200	0 to 5

6.9.4. Aggregate Subbase

Aggregate Subbase material should consist of imported, clean, durable, crushed angular rock. Such rock should be well-graded, have a maximum particle size of 1½ inches, have less than 5 percent passing the U.S. No. 200 sieve and meet the gradation requirements in Oregon Department of Transportation (ODOT) Standard Section 00331. In addition, aggregate base shall have a minimum of 75 percent fractured particles according to AASHTO TP-61 and a sand equivalent of not less than 30 percent based on AASHTO T-176.

6.9.5. 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.

6.10. 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 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 below. 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.

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**
On-site fine-grained soils	92	92	----
Imported Granular, maximum particle size < 1¼ inch	95	95	----
Imported Granular, maximum particle size 1¼ inch to 4 inches	n/a (proof-roll)	n/a (proof-roll)	----
Retaining Wall Backfill*	92	92	-----
Nonstructural Zones	90	90	90
Trench Backfill	95	90	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 and a jumping jack.

A representative from GeoEngineers should evaluate compaction of each lift of fill. Compaction should be evaluated by compaction testing, unless other methods are proposed for oversized materials and are approved by GeoEngineers prior to fill placement. These other methods typically involve procedural placement and compaction specifications together with verifying requirements such as proof-rolling or probing.

7.0 PAVEMENT RECOMMENDATIONS

Our interpretations of the subgrade resilient modulus are based on subsurface explorations, DCP testing on existing subgrade conducted as part of subsurface explorations on site for development of Lindburg Drive and Strong Road, and visual observations on site. Descriptions of our input parameters and the recommended pavement designs are summarized below.

- The pavement subgrades, fill subgrades and site earthwork used to establish road grades below the Aggregate Subbase and Aggregate Base materials have been prepared as described in Section 6.0 of this report.

- A resilient modulus of 20,000 psi has been estimated for compacted Aggregate Subbase and Aggregate Base materials.
- Based on the upper 1 foot of subgrade soils being recompacted, we estimate a resilient modulus of 4,000 psi or structural fill placed on recompacted in-place soils.
- Initial and terminal serviceability indices of 4.2 and 2.5, respectively.
- Reliability and standard deviations of 90 percent and 0.49, respectively.
- Structural coefficients of 0.41 and 0.10 for the new asphalt and base rock, respectively.
- A 20-year design life.
- The classification of proposed streets were unknown at the time this report was prepared. We provided pavement sections for estimated traffic for "Local Street" provided in the City of Salem Design Standards.

If any of the noted assumptions vary from project design use, our office should be contacted with the appropriate information so that the pavement designs can be revised or confirmed adequate.

TABLE 3. PAVEMENT SECTION RECOMMENDATIONS

Street Classification	Design ESALs	Minimum Asphalt Thickness (inches)	Minimum Aggregate Base Thickness (inches)	Minimum Aggregate Subbase Thickness (inches)
Local Street	100,000	4.0	12	NA
		4.0	4	10

Note:

ESALs = equivalent single axle loads

An alternate pavement section using Aggregate Subbase material is provided because it may be more applicable during wet-weather construction where a gravel haul road or working surface is needed to support construction traffic. Wet weather construction recommendations are provided in Section 6.0 of this report. The subbase material can be incorporated into the gravel working blankets and haul roads provided the material meets the minimum thickness in Table 4 and meets the specifications for Aggregate Subbase. Working blanket and haul road materials that pump excessively or have excessive fines from construction traffic should be removed and replaced with proper materials prior to constructing roadways over those areas.

TABLE 4. PAVEMENT SECTION RECOMMENDATIONS WITH CEMENT AMENDED SUB-BASE

Street Classification	Design ESALs	Minimum Asphalt Thickness (inches)	Minimum Aggregate Base Thickness (inches)	Minimum Cement Amended Subgrade Thickness (inches)
Local Street	100,000	3.5	6	12

Cement amendment may be used during site development, as described above, or to reduce the pavement section thickness. The exact design of the amount of cement to be used should be determined based on the condition of the subgrade at the time of construction and the prevailing weather conditions but should likely be between 3 and 6 percent. We recommend the minimum thickness of amendment be 12 inches. GeoEngineers can provide additional information regarding cement volumes at the time of construction. The minimum pavement sections, with a 12-inch-thick cement amended soil section, are provided in Table 4 above.

8.0 INFILTRATION TESTING

As requested, we conducted infiltration testing to assist in evaluating the site for design for stormwater infiltration. We conducted on-site infiltration tests near the south end of the site. In our opinion, the infiltration test results provided gives a general approximation of the in-situ infiltration rate but location specific tests should be performed to confirm infiltration rates due to the variability of the onsite soils observed.

We conducted infiltration testing in general accordance with the procedures outlined in "Division 004" of the COSDS at a depth of approximately 4 feet bgs marked as IT-1 and IT-2 in Figure 2. Testing was conducted using the open pit infiltration testing procedure described in "Division 004."

8.1. Testing Methods and Results

Infiltration test pits were 2 feet wide and 2 to 3 feet long with a testing depth of approximately 3 feet. Approximately 2 inches of clean rock was placed in the bottom of the test locations to help minimize disturbance of the fine-grained materials in the excavation while adding water. Between 12 and 18 inches of water was added to the test pits for a period of three hours to saturate the underlying soils.

After the saturation period, the test locations were filled with clean water to at least 1 foot above the bottom of the excavation. The drop-in water level was measured over a period of one hour after the soak period. In the case where the water level falls during the time-measured testing, infiltration rates diminish as a result of less head from the water column in the test. The field test results are summarized in Table 5.

TABLE 5. INFILTRATION RESULTS

Infiltration Test No.	USCS Material Type	Field Measured Infiltration Rate[1] (inches/hour)
IT-1	ML/MH	0.4
IT-2	ML	0.1[2]

Notes:

[1] Appropriate factors should be applied to the field-measured infiltration rate, based on the design methodology and specific system used.

[2] Measured rate very low over 3-hour test period. It is likely some higher amount of infiltration occurs over extended period. Based on observations, it estimated to be approximately 1/8 in/hr which will result in negligible design rates when divided by a factor of safety – effectively as low as 0 in/hr based on test methodology.

USCS = Unified Soil Classification System

The infiltration rates shown in Table 5 are field-measured infiltration rates. These represent a relatively short-term measured rate taken after the required saturation period, and factors of safety have not been applied for the type of infiltration system being considered, or for variability that may be present in the on-site soil. In our opinion, and consistent with the state of the practice, correction factors should be applied to this measured rate to reflect the small area of testing and the number of tests conducted.

Appropriate correction factors should also be applied by the project civil engineer to account for long-term infiltration parameters. From a geotechnical perspective, we recommend a factor of safety (correction factor) of at least 2 be applied to the infiltration values derived from field observations to account for potential soil variability with depth and location within the area tested. This will result in a recommended infiltration value of 2 to 3 inches per hour.

In addition, the stormwater system design engineer should determine and apply appropriate remaining correction factor values, or factors of safety, to account for repeated wetting and drying that occur in this area, degree of in-system filtration, frequency and type of system maintenance, vegetation, potential for siltation and bio-fouling, etc., as well as system design correction factors for overflow or redundancy and base and facility size. Siltation of the upper facility medium is a common problem in new facilities where fine-grained soils are present in uphill sites and can wash into the new facility limiting (at times to zero) the infiltration capacity of designed facilities.

The actual depths, lateral extent and estimated infiltration rates can vary from the values presented above. Field testing/confirmation during construction is often required in large or long systems or other situations where soil conditions may vary within the area where the system is constructed. The results of this field testing might necessitate that the infiltration locations be modified to achieve the design infiltration rate.

Also, infiltration flow rate of a focused stormwater system typically diminishes over time as suspended solids and precipitates in the stormwater further clog the void spaces between the soil particles or cake on the infiltration surface. The serviceable life of an infiltration media in a stormwater system can be extended by pre-filtering or with on-going accessible maintenance. Eventually, most systems will fail and will need to be replaced or have media regenerated or replaced. We recommend that infiltration systems include an overflow that is connected to a suitable discharge point. Also, infiltration systems can cause localized high groundwater levels and should not be located near basement walls, retaining walls, or other embedded structures unless these are specifically designed to account for the resulting hydrostatic pressure. Infiltration locations should not be located on sloping ground, unless it is approved by a geotechnical engineer, and should not be infiltrated at a location that allows for flow to travel laterally toward a slope face, such as a mounded water condition or too close to a slope face.

8.2. Suitability of Infiltration System

Successful design and implementation of stormwater infiltration systems and whether a system is suitable for a development depend on several site-specific factors. Stormwater infiltration systems are generally best suited for sites having sandy or gravelly soil with saturated hydraulic conductivities greater than 2 inches per hour. Sites with silty or clayey soil, are generally not well- suited for long-term stormwater infiltration or as a sole method of stormwater infiltration. Soils that have fine-grained matrices are susceptible to volumetric change and softening during wetting and drying cycles. Fine-grained soils also have large variations in the magnitude of infiltration rates because of bedding and stratification that occurs during alluvial deposition, and often have thin layers of less permeable or impermeable soil within a larger layer.

As a result of fine-grained soil conditions and relatively low field measured infiltration rates, we recommend infiltration of stormwater not be used in the upper soils as the sole method of stormwater management at this site unless those design factors can be otherwise accounted for by increasing infiltration area or coupling with other methods of stormwater disposal. At a minimum, an overflow method should be provided for the overall system.

9.0 LIMITATIONS

We have prepared this report for Ward Development, LLC for the proposed Strong Heights project at the former Fairview Hospital and Training Center at Old Strong Road SE and Reed Road SE in Salem, Oregon. Client may distribute copies of this report to the Oregon Department of Administrative Services and their authorized agents and regulatory agencies as may be required for the project.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of engineering geology and geotechnical engineering in this area at the time this report was prepared. The conclusions, recommendations, and opinions presented in this report are based on our professional knowledge, judgment and experience. No warranty or other conditions, express or implied, should be understood.

Please refer to the appendix titled “Report Limitations and Guidelines for Use” for additional information pertaining to use of this report.

10.0 REFERENCES

Bela, J.L. 1981. Geologic Map of the Rickreall and Salem West Quadrangles, Oregon: Oregon Department of Geology and Mineral Industries Geological Map Series GMS 18b. 1 plate, 1:24,000 scale.

Department of Geology and Mineral Industries (DOGAMI). 2019. DOGAMI web based Lidar Viewer accessed on December 17, 2019 at https://gis.dogami.oregon.gov/maps/lidarviewer/.

Federal Emergency Management Agency (FEMA). 2019. Flood Insurance Rate Map, Marion County Oregon and Incorporated Areas, Panel 344 of 1150. Map Number 41047C0344 H revised January 2, 2019. FEMA website search for 2250 Old Strong Road SE, Salem Oregon on December 17, 2019. https://msc.fema.gov/portal/search#searchresultsanchor

Harvey, A.F. and G.I. Peterson. 1998. Water Induced Landslide Hazards, Western Portion of the Salem Hills, Marion County, Oregon: Oregon Department of Geology and Mineral Industries, Interpretive Map Series IMS-6, 13p, 1 plate, 1:24,000 scale

Hofmeister, J.R and Y. Wang. 2000. Earthquake Induced Slope Instability: Relative Hazard Map Western Portion of the Salem Hills, Marion County, Oregon: Oregon Department of Geology and Mineral Industries Interpretive Map Series IMS-17, 1 plate, 1:24,000 scale

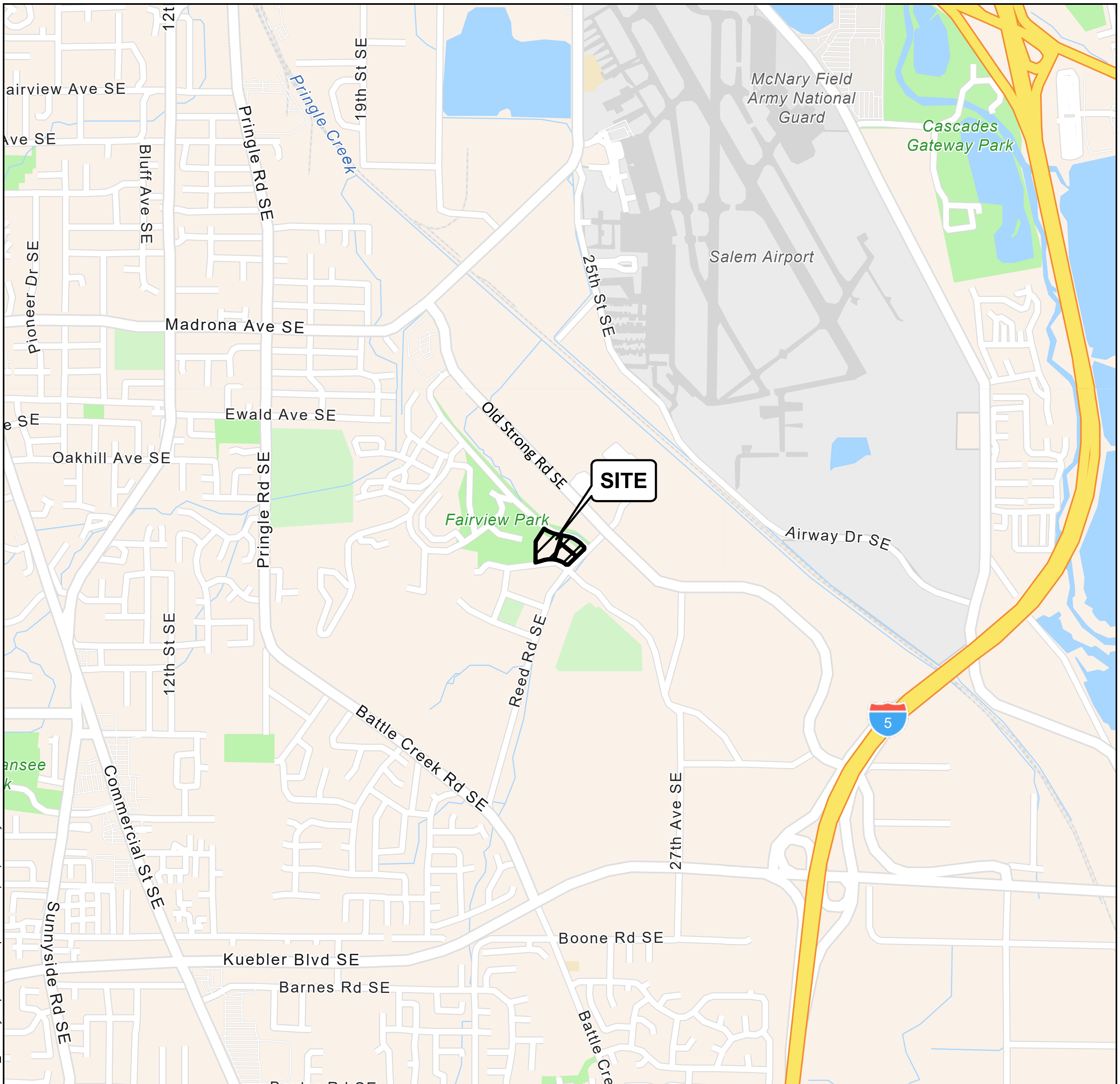
O'Connor, J.E., A. Sarna-Wojcick, K.C. Woznikak, D.J. Polette and R.J. Fleck. 2001. Origin, Extent, and Thickness of Quaternary Geologic Units in the Willamette Valley, Oregon; U.S. Geological Survey, Professional Paper 1620, 51 p., 1 plate, 1:100,000 scale.

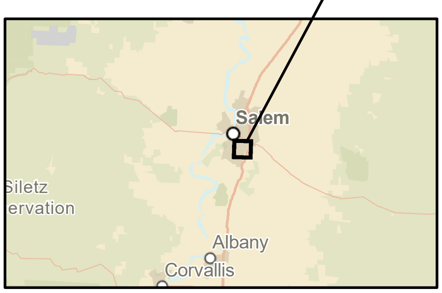
United States Geological Survey (USGS) Quaternary Fault and Fold Database of the United States accessed on December 17, 2019 at https://earthquake.usgs.gov/hazards/qfaults/.

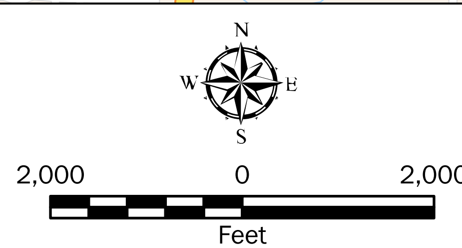
Wang, Y. and W.J. Leonard. 1996. Relative Earthquake Hazard Maps of the Salem East and Salem West Quadrangles, Marion and Polk Counties, Oregon: Oregon Department of Geology and Mineral Industries Geological Map Series GMS-105, 10p, 4 plates, 1:24,000 scale.

Williamson, D.A. 1994. Geotechnical Exploration – Drive Probe Method, in D.E. Hall, M.T. Long, and M.D. Rembolt, ed., Slope Stability Guide for National Forests in the United States, U.S. Department of Agriculture Forest Service Washington Office Engineering Staff Publication EM 7170-13, pp. 317-321.

FIGURES







Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: ESRI

Projection: NAD 1983 UTM Zone 10N

Vicinity Map

Strong Heights Subdivision
Salem, Oregon

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Figure 1

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Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: 2020 image from Google Earth Pro.
Roads from Marion County GIS.

Projection: NAD 1983 HARN StatePlane Oregon North FIPS 3601 Feet Intl

Legend

- Test Pit Number and Approximate Location
- Test Pit/Infiltration Test Number and Approximate Location
- DCP Number and Approximate Location
- Proposed Roadway

N
W E
S

60 0 60
Feet

Site Plan

Strong Heights Subdivision
Salem, Oregon

GEOENGINEERS

Figure 2

P:\24\24737002\GIS\24737002_Project\24737002_Project.aprx\2473700200_F02_SitePlan Date Exported: 04/19/21 by ccabrera

APPENDICES

GeoEngineers

APPENDIX A
Field Explorations and Laboratory Testing

APPENDIX A
FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

Shallow soil and groundwater conditions at the site were explored on April 14, 2021, by excavating 11 test pits at the approximate locations shown in Figure 2. The test pits were excavated by Cat 305 and Cat 315 tracked excavators owned and operated by K&E Excavating of Salem, Oregon to depths ranging from about 4 to 10½ feet below ground surface (bgs).

A staff certified engineering geologist (CEG) from our office observed the excavation of the test pits and maintained detailed logs of the borings and soundings, visually classified the soils encountered, and obtained representative soil samples from the borings. The same CEG performed a field reconnaissance on the same day.

Recovered soil samples were visually classified in the field in general accordance with ASTM International (ASTM) Standard Practices Test Method D2488 and the classification chart listed in Key to Exploration Logs, Figure A-1. Logs of the borings are presented in Figures A-2 through A-12. 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 might actually be gradual.

Laboratory Testing

Soil samples obtained from the explorations were visually classified in the field and in our laboratory using the Unified Soil Classification System (USCS) and ASTM classification methods. ASTM Test Method D2488 was used to visually classify the soil samples.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS
				SP	POORLY-GRADED SANDS, GRAVELLY SAND
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% PASSING NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

Sampler Symbol Descriptions

- 2.4-inch I.D. split barrel
- Standard Penetration Test (SPT)
- Shelby tube
- Piston
- Direct-Push
- Bulk or grab
- Continuous Coring

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

"P" indicates sampler pushed using the weight of the drill rig.

"WOH" indicates sampler pushed using the weight of the hammer.

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	AC	Asphalt Concrete
	CC	Cement Concrete
	CR	Crushed Rock/ Quarry Spalls
	SOD	Sod/Forest Duff
	TS	Topsoil

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Laboratory / Field Tests

%F	Percent fines
%G	Percent gravel
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DD	Dry density
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
Mohs	Mohs hardness scale
OC	Organic content
PM	Permeability or hydraulic conductivity
PI	Plasticity index
PL	Point load test
PP	Pocket penetrometer
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

Key to Exploration Logs

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Figure A-1

Date Excavated	4/14/2021	Total Depth (ft)	4	Logged By	JLL	Excavator Equipment	KRE Excavating CAT 305E2 rubber-tired excavator	Groundwater not observed
				Checked By				Caving not observed

Surface Elevation (ft) Vertical Datum	225	Latitude Longitude	44.897001 -123.010399	Coordinate System Horizontal Datum	Decimal Degrees WGS84

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
224	1				ML	Dark brown silt, occasional gravel and sand, fine roots to 6 inches (soft, moist) (fill)			
223	2				GM	Gray-brown silty gravel with sand, occasional asphalt fragments and angular basalt gravel to 4 inches (medium dense, moist)			
222	3				ML/MH	Gray-brown silt to elastic silt, red-brown mottling, low to moderate plasticity (soft to medium stiff, moist) (alluvium)			
221	4		1						PP = 1.0 IT=1 at 4 feet

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Log of Test Pit TP-1/IT-1

GeoEngineers

Project: Strong Heights Subdivision
Project Location: Salem, Oregon
Project Number: 24737-002-00

Date Excavated	4/14/2021	Total Depth (ft)	4	Logged By	JLL	Excavator	KRE Excavating	Groundwater not observed
				Checked By		Equipment	CAT 305E2 rubber-tired excavator	Caving not observed

Surface Elevation (ft) Vertical Datum	222	Latitude Longitude	44.896888 -123.010009	Coordinate System Horizontal Datum	Decimal Degrees WGS84

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
221, 220	1, 2				GM	Dark brown silty gravel with sand, round to angular gravel to 4 inches, occasional asphalt fragments (medium dense, moist) (fill)			
219, 218	3, 4		1		ML	Gray-brown silt, low plasticity, red-brown mottling (medium stiff, moist) (alluvium)			PP = 1.5 IT-2 at 4 feet

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Log of Test Pit TP-2/IT-2

GeoEngineers

Project: Strong Heights Subdivision
Project Location: Salem, Oregon
Project Number: 24737-002-00

Figure A-3
Sheet 1 of 1

Date Excavated	4/14/2021	Total Depth (ft)	8.5	Logged By	JLL	Excavator	KRE Excavating	Groundwater not observed
				Checked By		Equipment	CAT 305E2 rubber-tired excavator	Caving not observed

Surface Elevation (ft) Vertical Datum	233	Latitude Longitude	44.896853 -123.010689	Coordinate System Horizontal Datum	Decimal Degrees WGS84

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
					ML	Dark brown silt, low plasticity, fine roots 8 to 10 inches, occasional fine sand and small asphaltic gravel (stiff, moist) (alluvium)			
232	1		1						PP = 3.0
						Grades to brown, no sand or gravel, occasional large roots			
231	2								
									PP = 2.8
230	3		2						PP = 3.0
229	4					Grades to yellow-brown, weak red-brown and light gray mottling			
228	5		3						
227	6								
226	7								
225	8								

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Log of Test Pit TP-3

GeoEngineers

Project: Strong Heights Subdivision
Project Location: Salem, Oregon
Project Number: 24737-002-00

Figure A-4
Sheet 1 of 1

Date Excavated	4/14/2021	Total Depth (ft)	8	Logged By	JLL	Excavator	KRE Excavating	Groundwater not observed
				Checked By		Equipment	CAT 305E2 rubber-tired excavator	Caving not observed

Surface Elevation (ft) Vertical Datum	244	Latitude Longitude	44.896533 -123.011248	Coordinate System Horizontal Datum	Decimal Degrees WGS84

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
244	0				ML	Brown silt, low plasticity, roots to 6 to 8 inches, occasional fine sand (stiff, moist) (alluvium)			PP = 3.0
243	1								
242	2		1			Grades to yellow-brown, medium stiff to stiff			PP = 2.5-1.5
241	3								PP = 2.0
240	4								
239	5								
238	6								
237	7								
236	8								

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Log of Test Pit TP-4

GEOENGINEERS

Project: Strong Heights Subdivision
Project Location: Salem, Oregon
Project Number: 24737-002-00

Date Excavated	4/14/2021	Total Depth (ft)	7	Logged By	JLL	Excavator Equipment	KRE Excavating CAT 305E2 rubber-tired excavator	Groundwater not observed
				Checked By				Caving not observed

Surface Elevation (ft) Vertical Datum	254	Latitude Longitude	44.896678 -123.01222	Coordinate System Horizontal Datum	Decimal Degrees WGS84

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
					GM	Brown silty gravel, red rock and concrete fragments with silt and sand (medium dense, dry) (fill)			
					ML	Brown silt, low plasticity, trace fine sand (stiff, moist) (alluvium)			PP = 3.5
253	1								
									PP = 2.0
252	2		1						
						Grades to yellow-brown			PP = 3.0
251	3								
									PP = 3.0
250	4								
249	5								
248	6								
247	7								

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Log of Test Pit TP-5

GeoEngineers

Project: Strong Heights Subdivision
Project Location: Salem, Oregon
Project Number: 24737-002-00

Date Excavated	4/14/2021	Total Depth (ft)	8	Logged By	JLL	Excavator Equipment	KRE Excavating CAT 305E2 rubber-tired excavator	Groundwater not observed
				Checked By				Caving not observed

Surface Elevation (ft) Vertical Datum	249	Latitude Longitude	44.897163 -123.012162	Coordinate System Horizontal Datum	Decimal Degrees WGS84

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
248 – 246	0 – 2.5				GM	Brown silty gravel, round to angular gravel, occasional concrete fragments and debris with concrete, a metal pipe to 2 to 3 feet maximum (dense, dry) (debris fill)			
246	2.5				ML	Gray silt, low plasticity, trace sand, occasional spall gravel (hard, moist) (alluvium)			PP = 4.0+
245	3 – 4	▯	1			Grades to brown, stiff			PP = 3.0
244 – 242	5 – 7					Grades to yellow-brown, medium stiff			
241	8								

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Log of Test Pit TP-6

GeoEngineers

Project: Strong Heights Subdivision
Project Location: Salem, Oregon
Project Number: 24737-002-00

Date Excavated	4/14/2021	Total Depth (ft)	10.5	Logged By	JLL	Excavator	KRE Excavating	Groundwater not observed
				Checked By		Equipment	CAT 305E2 rubber-tired excavator	Caving not observed

Surface Elevation (ft) / Vertical Datum	237	Latitude / Longitude	44.897255 / -123.01166	Coordinate System / Horizontal Datum	Decimal Degrees / WGS84

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
					ML	Gray, brown, occasional black silt with sand, gravel, occasional debris (geotextile, angular ballast rock to 4 to 6 inches) (soft to stiff, moist) (fill)			PP = 3.5
236	1		1						
235	2								PP = 1.0
						Brick at 2½ feet			PP = 1.5
234	3								
233	4								
232	5								
231	6								
230	7					Metal pipe at 7 feet			
229	8								
228	9					Styrofoam at 9 feet			
					ML	Yellow-brown silt, low plasticity (soft to medium stiff, moist) (alluvium)			PP = 0.75
227	10		2						

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Log of Test Pit TP-7

GeoEngineers

Project: Strong Heights Subdivision
Project Location: Salem, Oregon
Project Number: 24737-002-00

Date:4/15/21 Path:P:\24\24737002\GINT\2473700200.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_TESTPIT_1P_GEOTEC_%F

Date Excavated	4/14/2021	Total Depth (ft)	9	Logged By	JLL	Excavator	KRE Excavating	Groundwater not observed
				Checked By		Equipment	CAT 305E2 rubber-tired excavator	Caving not observed

Surface Elevation (ft) Vertical Datum	242	Latitude Longitude	44.897075 -123.011788	Coordinate System Horizontal Datum	Decimal Degrees WGS84

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
242	0				GM/SM	Brown silty gravel with sand to silty sand with gravel, occasional concrete fragments to 4 to 6 inches (medium dense, moist) (fill)			
241	1								
240	2								
239	3								
238	4				ML/OL	Black silt with organic matter to organic silt, low plasticity, roots and sticks (soft, moist) (alluvium)			
237	5		1						
236	6				ML	Grades to yellow-brown silt, low plasticity (medium stiff, moist)			
235	7								
234	8								
233	9								

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Log of Test Pit TP-8

GeoEngineers

Project: Strong Heights Subdivision
Project Location: Salem, Oregon
Project Number: 24737-002-00

Figure A-9
Sheet 1 of 1

Date Excavated	4/14/2021	Total Depth (ft)	6	Logged By	JLL	Excavator Equipment	KRE Excavating CAT 305E2 rubber-tired excavator	Groundwater not observed
				Checked By				Caving not observed

Surface Elevation (ft) Vertical Datum	245	Latitude Longitude	44.896848 -123.011877	Coordinate System Horizontal Datum	Decimal Degrees WGS84

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
244	1				DEBRIS	Mix of concrete fragments, reinforced steel, organic matter and sod (loose, dry to moist) (fill)			
243	2				ML	Brown silt, low plasticity, trace fine sand (medium stiff, moist) (alluvium)			PP = 2.0
242	3								PP = 1.5
241	4								
240	5								
239	6								

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Log of Test Pit TP-9

GEOENGINEERS

Project: Strong Heights Subdivision
Project Location: Salem, Oregon
Project Number: 24737-002-00

Date Excavated 4/14/2021	Total Depth (ft) 8	Logged By JLL Checked By	Excavator KRE Excavating Equipment CAT 305E2 rubber-tired excavator	Groundwater not observed Caving not observed
Surface Elevation (ft) 241 Vertical Datum	Latitude 44.896959 Longitude -123.011345	Coordinate System Decimal Degrees Horizontal Datum WGS84		

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
241	0				ML	Brown silt with sand and gravel, low plasticity, round to angular gravel, occasional concrete fragments (medium stiff, moist) (fill)			
240	1								
239	2								
238	3		1						
237	4								
236	5								
235.5	5.5				ML	Brown silt, low plasticity, trace to occasional organic matter (soft, moist) (alluvium)			PP = 0.75
235	6								
234.5	6.5					Grades to yellow-brown, medium stiff			
234	7								
233	8								

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Date:4/15/21 Path:P:\24\24737002\GINT\2473700200.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_TESTPIT_1P_GEOTEC_%F

Log of Test Pit TP-10

GeoEngineers

Project: Strong Heights Subdivision
Project Location: Salem, Oregon
Project Number: 24737-002-00

Date Excavated	4/14/2021	Total Depth (ft)	8	Logged By	JLL	Excavator Equipment	KRE Excavating CAT 305E2 rubber-tired excavator	Groundwater not observed
				Checked By				Caving not observed

Surface Elevation (ft) Vertical Datum	237	Latitude Longitude	44.896902 -123.011009	Coordinate System Horizontal Datum	Decimal Degrees WGS84

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
					ML	Brown silt with sand and gravel, occasional chunks of concrete to 2 to 3 inches (medium stiff, moist) (fill)			PP = 1.5
236	1								
235	2					Becomes soft			PP = 0.5
234	3		1		OL	Black organic silt, occasional branches, duff and organic fine sand (soft, moist) (fill?)			
233	4								
232	5				ML	Brown silt, low plasticity (medium stiff, moist) (alluvium)			
231	6								
230	7								
229	8								

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Log of Test Pit TP-11

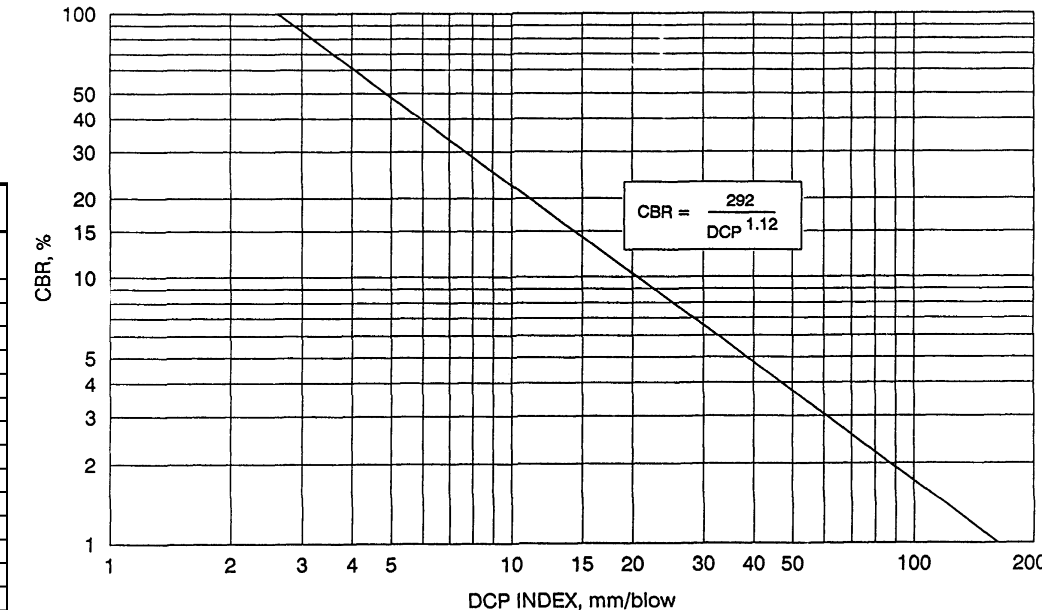
GeoEngineers

Project: Strong Heights Subdivision
Project Location: Salem, Oregon
Project Number: 24737-002-00

Location:	Old Strong Road	Date:	4/14/2021	Test Hole Number:	DCP-1 (near TP-4)
Depth to bottom:	36"	Pilot Hole Depth		Test Method:	Dynamic Cone Penetration
Tester's Name:	John Lawes			GeoEngineers Job:	24747-002-00
Tester's Company:	GeoEngineers, Inc.	Tester's Contact No:	971-409-7390	Project Name	Ward - Strong Heights
Notes:					

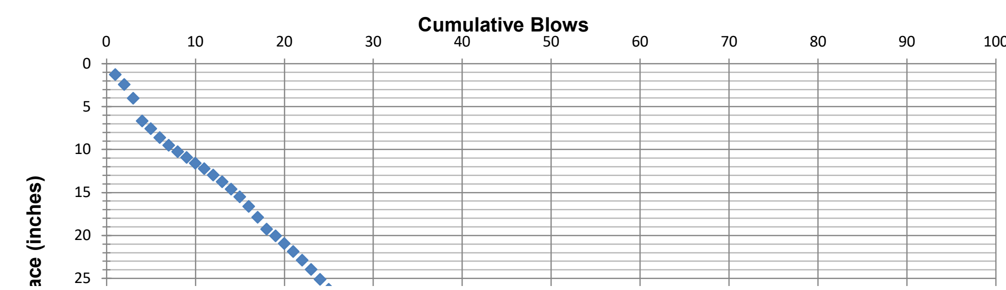
Depth	Soil Texture

Test increment	Number of blows	Cumulative blows	Depth Below Ground Surface	Penetration per increment	Cumulative penetration	Cummulative Penetration	Penetration per blow set	Penetration per blow	Hammer blow factor	DCP Index	DCP Index	CBR	M_R
#	#	#	(in)	(mm)	(mm)	(in)	(in)	(in)	1 for 8-kg 2 for 4.6-kg hammer	in/blow	mm/blow	%	psi
1	1	1	1.3	32.0	32.0	1.3	1.3	1.26	1	1.26	32.00	6	4441
2	1	2	2.4	30.0	62.0	2.4	1.2	1.18	1	1.18	30.00	6	4554
3	1	3	4.0	40.0	102.0	4.0	1.6	1.57	1	1.57	40.00	5	4071
4	1	4	6.7	67.0	169.0	6.7	2.6	2.64	1	2.64	67.00	3	3329
5	1	5	7.6	23.0	192.0	7.6	0.9	0.91	1	0.91	23.00	9	5051
6	1	6	8.6	27.0	219.0	8.6	1.1	1.06	1	1.06	27.00	7	4745
7	1	7	9.5	22.0	241.0	9.5	0.9	0.87	1	0.87	22.00	9	5140
8	1	8	10.2	19.0	260.0	10.2	0.7	0.75	1	0.75	19.00	11	5442
9	1	9	10.9	17.0	277.0	10.9	0.7	0.67	1	0.67	17.00	12	5683
10	1	10	11.6	17.0	294.0	11.6	0.7	0.67	1	0.67	17.00	12	5683
11	1	11	12.2	16.0	310.0	12.2	0.6	0.63	1	0.63	16.00	13	5819
12	1	12	13.0	19.0	329.0	13.0	0.7	0.75	1	0.75	19.00	11	5442
13	1	13	13.7	20.0	349.0	13.7	0.8	0.79	1	0.79	20.00	10	5334
14	1	14	14.6	22.0	371.0	14.6	0.9	0.87	1	0.87	22.00	9	5140
15	1	15	15.5	23.0	394.0	15.5	0.9	0.91	1	0.91	23.00	9	5051
16	1	16	16.6	28.0	422.0	16.6	1.1	1.10	1	1.10	28.00	7	4678
17	1	17	17.9	32.0	454.0	17.9	1.3	1.26	1	1.26	32.00	6	4441
18	1	18	19.3	35.0	489.0	19.3	1.4	1.38	1	1.38	35.00	5	4288
19	1	19	20.0	20.0	509.0	20.0	0.8	0.79	1	0.79	20.00	10	5334
20	1	20	20.9	23.0	532.0	20.9	0.9	0.91	1	0.91	23.00	9	5051
21	1	21	21.9	24.0	556.0	21.9	0.9	0.94	1	0.94	24.00	8	4968
22	1	22	22.9	25.0	581.0	22.9	1.0	0.98	1	0.98	25.00	8	4890
23	1	23	24.0	28.0	609.0	24.0	1.1	1.10	1	1.10	28.00	7	4678
24	1	24	25.1	29.0	638.0	25.1	1.1	1.14	1	1.14	29.00	7	4615
25	1	25	26.3	29.0	667.0	26.3	1.1	1.14	1	1.14	29.00	7	4615
26	1	26	27.4	29.0	696.0	27.4	1.1	1.14	1	1.14	29.00	7	4615
27	1	27	28.5	27.0	723.0	28.5	1.1	1.06	1	1.06	27.00	7	4745
28	1	28	29.6	28.0	751.0	29.6	1.1	1.10	1	1.10	28.00	7	4678
29	1	29	30.5	23.0	774.0	30.5	0.9	0.91	1	0.91	23.00	9	5051
30	1	30	31.5	25.0	799.0	31.5	1.0	0.98	1	0.98	25.00	8	4890
31	1	31	32.3	22.0	821.0	32.3	0.9	0.87	1	0.87	22.00	9	5140



(after Webster et al., 1992)

Webster, S. L., Grau, R. H., and Williams, T. P. (1992). Description and application of dual mass dynamic cone penetrometer. Department of the Army Waterways Equipment Station, No. GL-92-3.



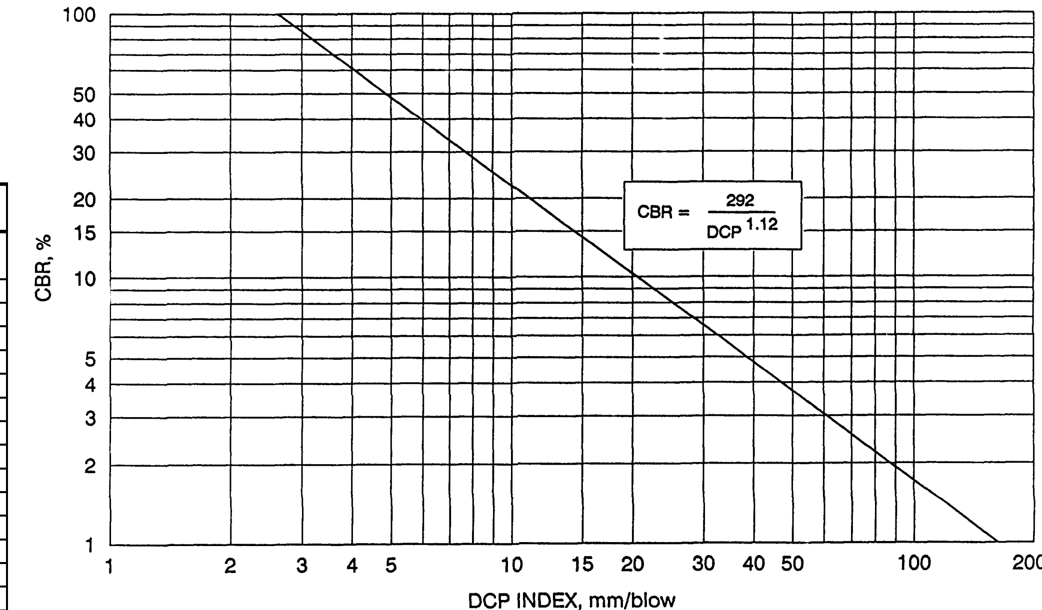
Location: Old Strong Road
Depth to bottom: 37"
Tester's Name: John Lawes
Tester's Company: GeoEngineers, Inc.
Notes:

Date: 4/14/2021
Pilot Hole Depth
Tester's Contact No: 971-409-7390

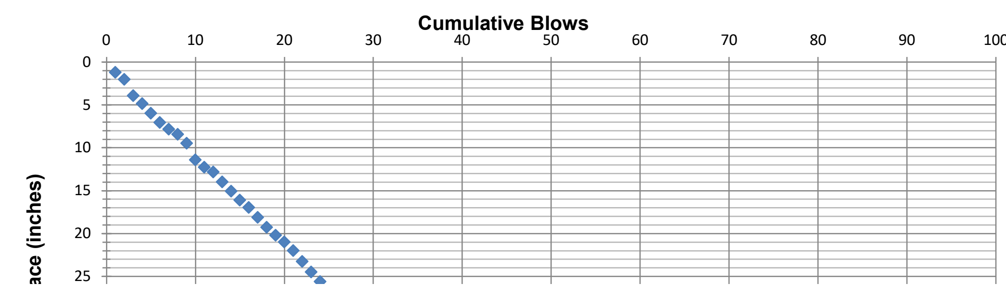
Test Hole Number: DCP-2 (near TP-3)
Test Method: Dynamic Cone Penetration
GeoEngineers Job: 24747-002-00
Project Name Ward - Strong Heights

Depth	Soil Texture

Test increment	Number of blows	Cumulative blows	Depth Below Ground Surface	Penetration per increment	Cumulative penetration	Cummulative Penetration	Penetration per blow set	Penetration per blow	Hammer blow factor	DCP Index	DCP Index	CBR	M_R
#	#	#	(in)	(mm)	(mm)	(in)	(in)	(in)	1 for 8-kg 2 for 4.6-kg hammer	in/blow	mm/blow	%	psi
1	1	1	1.2	30.0	30.0	1.2	1.2	1.18	1	1.18	30.00	6	4554
2	1	2	2.0	21.0	51.0	2.0	0.8	0.83	1	0.83	21.00	10	5234
3	1	3	3.9	48.0	99.0	3.9	1.9	1.89	1	1.89	48.00	4	3791
4	1	4	4.8	24.0	123.0	4.8	0.9	0.94	1	0.94	24.00	8	4968
5	1	5	5.9	28.0	151.0	5.9	1.1	1.10	1	1.10	28.00	7	4678
6	1	6	7.0	28.0	179.0	7.0	1.1	1.10	1	1.10	28.00	7	4678
7	1	7	7.8	20.0	199.0	7.8	0.8	0.79	1	0.79	20.00	10	5334
8	1	8	8.4	15.0	214.0	8.4	0.6	0.59	1	0.59	15.00	14	5967
9	1	9	9.4	26.0	240.0	9.4	1.0	1.02	1	1.02	26.00	8	4815
10	1	10	11.4	50.0	290.0	11.4	2.0	1.97	1	1.97	50.00	4	3731
11	1	11	12.2	21.0	311.0	12.2	0.8	0.83	1	0.83	21.00	10	5234
12	1	12	12.8	15.0	326.0	12.8	0.6	0.59	1	0.59	15.00	14	5967
13	1	13	14.0	29.0	355.0	14.0	1.1	1.14	1	1.14	29.00	7	4615
14	1	14	15.0	27.0	382.0	15.0	1.1	1.06	1	1.06	27.00	7	4745
15	1	15	16.1	27.0	409.0	16.1	1.1	1.06	1	1.06	27.00	7	4745
16	1	16	17.0	22.0	431.0	17.0	0.9	0.87	1	0.87	22.00	9	5140
17	1	17	18.1	29.0	460.0	18.1	1.1	1.14	1	1.14	29.00	7	4615
18	1	18	19.3	29.0	489.0	19.3	1.1	1.14	1	1.14	29.00	7	4615
19	1	19	20.2	24.0	513.0	20.2	0.9	0.94	1	0.94	24.00	8	4968
20	1	20	21.0	20.0	533.0	21.0	0.8	0.79	1	0.79	20.00	10	5334
21	1	21	22.0	25.0	558.0	22.0	1.0	0.98	1	0.98	25.00	8	4890
22	1	22	23.3	33.0	591.0	23.3	1.3	1.30	1	1.30	33.00	6	4388
23	1	23	24.5	31.0	622.0	24.5	1.2	1.22	1	1.22	31.00	6	4496
24	1	24	25.6	28.0	650.0	25.6	1.1	1.10	1	1.10	28.00	7	4678
25	1	25	27.0	35.0	685.0	27.0	1.4	1.38	1	1.38	35.00	5	4288
26	1	26	27.9	23.0	708.0	27.9	0.9	0.91	1	0.91	23.00	9	5051
27	1	27	28.7	22.0	730.0	28.7	0.9	0.87	1	0.87	22.00	9	5140
28	1	28	29.5	19.0	749.0	29.5	0.7	0.75	1	0.75	19.00	11	5442
29	1	29	30.6	28.0	777.0	30.6	1.1	1.10	1	1.10	28.00	7	4678
30	1	30	31.6	26.0	803.0	31.6	1.0	1.02	1	1.02	26.00	8	4815
31	1	31	32.9	32.0	835.0	32.9	1.3	1.26	1	1.26	32.00	6	4441



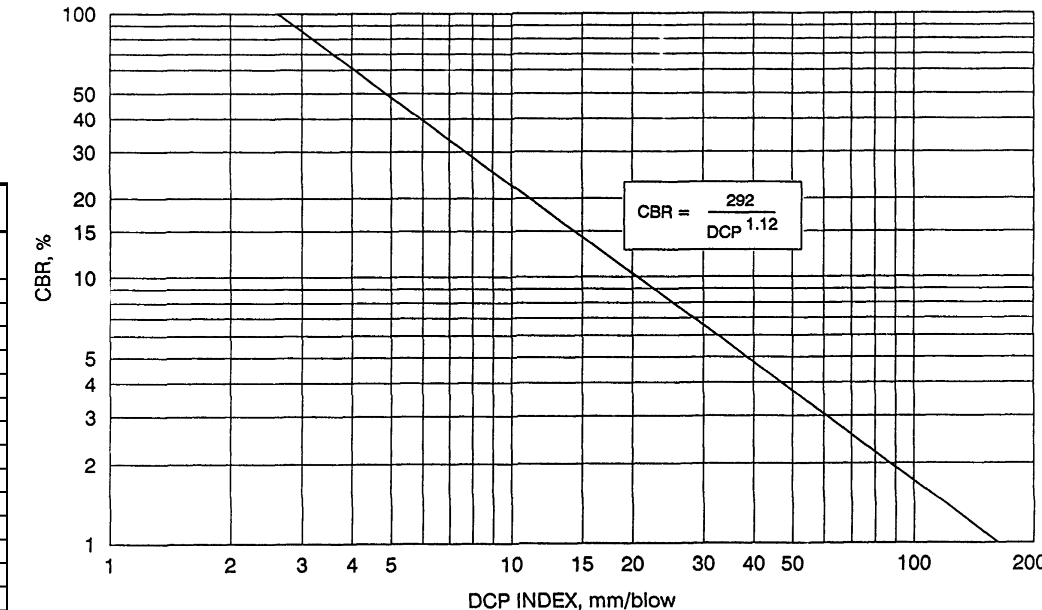
(after Webster et al., 1992)
Webster, S. L., Grau, R. H., and Williams, T. P. (1992). Description and application of dual mass dynamic cone penetrometer. Department of the Army Waterways Equipment Station, No. GL-92-3.



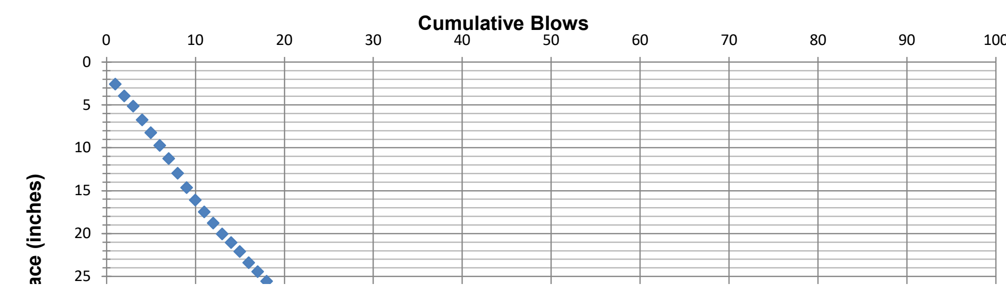
Location:	Old Strong Road	Date:	4/14/2021	Test Hole Number:	DCP-3 (near TP-5)
Depth to bottom:	36"	Pilot Hole Depth		Test Method:	Dynamic Cone Penetration
Tester's Name:	John Lawes			GeoEngineers Job:	24747-002-00
Tester's Company:	GeoEngineers, Inc.	Tester's Contact No:	971-409-7390	Project Name	Ward - Strong Heights
Notes:					

Depth	**Soil Texture**

Test increment	Number of blows	Cumulative blows	Depth Below Ground Surface	Penetration per increment	Cumulative penetration	Cummulative Penetration	Penetration per blow set	Penetration per blow	Hammer blow factor	DCP Index	DCP Index	CBR	M_R
#	#	#	(in)	(mm)	(mm)	(in)	(in)	(in)	1 for 8-kg 2 for 4.6-kg hammer	in/blow	mm/blow	%	psi
1	1	1	2.6	65.0	65.0	2.6	2.6	2.56	1	2.56	65.00	3	3368
2	1	2	3.9	35.0	100.0	3.9	1.4	1.38	1	1.38	35.00	5	4288
3	1	3	5.2	31.0	131.0	5.2	1.2	1.22	1	1.22	31.00	6	4496
4	1	4	6.7	40.0	171.0	6.7	1.6	1.57	1	1.57	40.00	5	4071
5	1	5	8.2	38.0	209.0	8.2	1.5	1.50	1	1.50	38.00	5	4153
6	1	6	9.7	38.0	247.0	9.7	1.5	1.50	1	1.50	38.00	5	4153
7	1	7	11.3	39.0	286.0	11.3	1.5	1.54	1	1.54	39.00	5	4111
8	1	8	13.0	43.0	329.0	13.0	1.7	1.69	1	1.69	43.00	4	3957
9	1	9	14.6	43.0	372.0	14.6	1.7	1.69	1	1.69	43.00	4	3957
10	1	10	16.1	37.0	409.0	16.1	1.5	1.46	1	1.46	37.00	5	4196
11	1	11	17.5	35.0	444.0	17.5	1.4	1.38	1	1.38	35.00	5	4288
12	1	12	18.8	33.0	477.0	18.8	1.3	1.30	1	1.30	33.00	6	4388
13	1	13	20.0	32.0	509.0	20.0	1.3	1.26	1	1.26	32.00	6	4441
14	1	14	21.1	26.0	535.0	21.1	1.0	1.02	1	1.02	26.00	8	4815
15	1	15	22.1	26.0	561.0	22.1	1.0	1.02	1	1.02	26.00	8	4815
16	1	16	23.4	33.0	594.0	23.4	1.3	1.30	1	1.30	33.00	6	4388
17	1	17	24.4	27.0	621.0	24.4	1.1	1.06	1	1.06	27.00	7	4745
18	1	18	25.6	28.0	649.0	25.6	1.1	1.10	1	1.10	28.00	7	4678
19	1	19	26.8	32.0	681.0	26.8	1.3	1.26	1	1.26	32.00	6	4441
20	1	20	28.1	32.0	713.0	28.1	1.3	1.26	1	1.26	32.00	6	4441
21	1	21	29.3	32.0	745.0	29.3	1.3	1.26	1	1.26	32.00	6	4441
22	1	22	30.5	29.0	774.0	30.5	1.1	1.14	1	1.14	29.00	7	4615
23	1	23	31.9	36.0	810.0	31.9	1.4	1.42	1	1.42	36.00	5	4241
24	1	24	33.3	37.0	847.0	33.3	1.5	1.46	1	1.46	37.00	5	4196
25	1	25	34.8	37.0	884.0	34.8	1.5	1.46	1	1.46	37.00	5	4196
26	1	26	36.1	34.0	918.0	36.1	1.3	1.34	1	1.34	34.00	6	4337



(after Webster et al., 1992)
Webster, S. L., Grau, R. H., and Williams, T. P. (1992). Description and application of dual mass dynamic cone penetrometer. Department of the Army Waterways Equipment Station, No. GL-92-3.



APPENDIX B
Report Limitations and Guidelines for Use

APPENDIX B
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This appendix provides information to help you manage your risks with respect to the use of this report.

Read These Provisions Closely

It is important to recognize that the geoscience practices (geotechnical engineering, geology and environmental science) rely on professional judgment and opinion to a greater extent than other engineering and natural science disciplines, where more precise and/or readily observable data may exist. To help clients better understand how this difference pertains to our services, GeoEngineers includes the following explanatory "limitations" provisions in its reports. Please confer with GeoEngineers if you need to know more how these "Report Limitations and Guidelines for Use" apply to your project or site.

Geotechnical Services Are Performed for Specific Purposes, Persons and Projects

This report has been prepared for Ward Development, LLC for the project specifically identified in the report. The information contained herein is not applicable to other sites or projects.

GeoEngineers structures its services to meet the specific needs of its clients. No party other than the party to whom this report is addressed may rely on the product of our services unless we agree to such reliance in advance and in writing. Within the limitations of the agreed scope of services for the Project, and its schedule and budget, our services have been executed in accordance with our Agreement with Ward Development dated March 31, 2020, and generally accepted geotechnical practices in this area at the time this report was prepared. We do not authorize, and will not be responsible for, the use of this report for any purposes or projects other than those identified in the report.

A Geotechnical Engineering or Geologic Report is based on a Unique Set of Project-Specific Factors

This report has been prepared for the proposed Strong Heights project at the former Fairview Hospital and Training Center Project in Salem, Oregon. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, it is important not to rely on this report if it was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

For example, changes that can affect the applicability of this report include those that affect:

- the function of the proposed structure;

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[1] Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org.

- elevation, configuration, location, orientation or weight of the proposed structure;
- composition of the design team; or
- project ownership.

If changes occur after the date of this report, GeoEngineers cannot be responsible for any consequences of such changes in relation to this report unless we have been given the opportunity to review our interpretations and recommendations. Based on that review, we can provide written modifications or confirmation, as appropriate.

Environmental Concerns Are Not Covered

Unless environmental services were specifically included in our scope of services, this report does not provide any environmental findings, conclusions, or recommendations, including but not limited to, the likelihood of encountering underground storage tanks or regulated contaminants.

Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by man-made events such as construction on or adjacent to the site, new information or technology that becomes available subsequent to the report date, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. If more than a few months have passed since issuance of our report or work product, or if any of the described events may have occurred, please contact GeoEngineers before applying this report for its intended purpose so that we may evaluate whether changed conditions affect the continued reliability or applicability of our conclusions and recommendations.

Geotechnical and Geologic Findings Are Professional Opinions

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies the specific subsurface conditions only at those points where subsurface tests are conducted, or samples are taken. GeoEngineers reviewed field and laboratory data and then applied its professional judgment to render an informed opinion about subsurface conditions at other locations. Actual subsurface conditions may differ, sometimes significantly, from the opinions presented in this report. Our report, conclusions and interpretations are not a warranty of the actual subsurface conditions.

Geotechnical Engineering Report Recommendations Are Not Final

We have developed the following recommendations based on data gathered from subsurface investigation(s). These investigations sample just a small percentage of a site to create a snapshot of the subsurface conditions elsewhere on the site. Such sampling on its own cannot provide a complete and accurate view of subsurface conditions for the entire site. Therefore, the recommendations included in this report are preliminary and should not be considered final. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for the recommendations in this report if we do not perform construction observation.

We recommend that you allow sufficient monitoring, testing and consultation during construction by GeoEngineers to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes if the conditions revealed during the work differ from those anticipated, and to evaluate whether earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective means of managing the risks associated with unanticipated conditions. If another party performs field observation and confirms our expectations, the other party must take full responsibility for both the observations and recommendations. Please note, however, that another party would lack our project-specific knowledge and resources.

A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

Misinterpretation of this report by members of the design team or by contractors can result in costly problems. GeoEngineers can help reduce the risks of misinterpretation by conferring with appropriate members of the design team after submitting the report, reviewing pertinent elements of the design team's plans and specifications, participating in pre-bid and preconstruction conferences, and providing construction observation.

Do Not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. The logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Photographic or electronic reproduction is acceptable but separating logs from the report can create a risk of misinterpretation.

Give Contractors a Complete Report and Guidance

To help reduce the risk of problems associated with unanticipated subsurface conditions, GeoEngineers recommends giving contractors the complete geotechnical engineering or geologic report, including these "Report Limitations and Guidelines for Use." When providing the report, you should preface it with a clearly written letter of transmittal that:

- advises contractors that the report was not prepared for purposes of bid development and that its accuracy is limited; and
- encourages contractors to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer.

Contractors Are Responsible for Site Safety on Their Own Construction Projects

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and adjacent properties.

Biological Pollutants

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings or conclusions regarding the detecting, assessing, preventing or abating of Biological Pollutants, and no conclusions or inferences should be drawn regarding Biological Pollutants as they may relate to this project. The term “Biological Pollutants” includes, but is not limited to, molds, fungi, spores, bacteria and viruses, and/or any of their byproducts. A Client that desires these specialized services is advised to obtain them from a consultant who offers services in this specialized field.

GeoEngineers