

Geotechnical Engineering Report

Proposed Parkland Village Addition

3121 NE Cumulus Avenue

McMinnville, Oregon

June 22, 2016

Terracon Project No. 82165034

Prepared for:

RJ Development

Olympia, WA

Prepared by:

Terracon Consultants, Inc.

Portland, Oregon

Offices Nationwide
Employee-Owned

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Geotechnical ■ Environmental ■ Construction Materials ■ Facilities

June 22, 2016



RJ Development
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Attn: Mr. Joshua Snodgrass
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Re: Geotechnical Engineering Report
Proposed Parkland Village Addition
3121 NE Cumulus Avenue.
McMinnville, Oregon
Terracon Project No. 82165034

Dear Mr. Snodgrass:

Terracon Consultants, Inc. (Terracon) has completed the geotechnical engineering services for the above referenced project. These services were performed in general accordance with Terracon's Proposal P82165031, dated March 23, 2016. This geotechnical engineering report presents the results of the subsurface exploration and provides geotechnical recommendations concerning earthwork and the design and construction of foundations, floor slabs, and pavements for the proposed project.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report, or if we may be of further service, please contact us.

Sincerely,
Terracon Consultants, Inc.

A blue ink signature of Brice W. Plouse.

Brice W. Plouse, EIT
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EXECUTIVE SUMMARY

Geotechnical explorations have been performed for the Proposed Parkland Addition located at 3121 NE Cumulus Avenue in McMinnville, Oregon. Terracon's geotechnical scope of work included the advancement of three geotechnical test borings to depths of up to 51½ feet below existing site grades (bgs) within the proposed development areas at the site.

The site appears suitable for the proposed construction based upon geotechnical conditions encountered in the borings and our current understanding of the proposed development. The following geotechnical considerations were identified:

- **Subsurface Conditions:** Geotechnical exploration borings B-1 through B-3 encountered native silt and sand soils throughout the depth of the borings. The native silt and sand soils are soft to stiff.
- **Structure Foundation Support:** Based on the subsurface conditions encountered at the site, the structures may be supported on conventional foundations bearing on a minimum of one foot of compacted select fill atop competent native soils. The compacted select fill is needed to limit static settlement.
- **Slope Stability:** Based on our analyses, the existing slope adjacent to the site is marginally stable. Therefore, in order to prevent adverse impacts to the existing slope and to protect the proposed development from potential slope instability, we recommend that the development incorporate a setback from the top of slope of at least 35 feet. In addition, due to the seismic risk of slope movement, the footings nearest the slope should be supported on a four foot thick geogrid-reinforced structural fill prism. If a reduced setback is desired, slope stabilization improvements would be necessary.
- Close monitoring of the construction operations discussed herein will be critical in achieving the design subgrade support. Therefore, we recommend that Terracon be retained to monitor this portion of the work.

This summary should be used in conjunction with the entire report for design purposes. It should be recognized that details were not included or fully developed in this section, and the report must be read in its entirety for a comprehensive understanding of the items contained herein. The section titled **GENERAL COMMENTS** should be read for an understanding of the report limitations.

**GEOTECHNICAL ENGINEERING REPORT
 PROPOSED PARKLAND VILLAGE ADDITION
 3121 NE CUMULUS AVENUE
 MCMINNVILLE, OREGON
 Terracon Project No. 82165034
 June 22, 2016**

1.0 INTRODUCTION

This report presents the results of our geotechnical engineering services performed for the Proposed Parkland Village Addition to be located at 3121 NE Cumulus Avenue in McMinnville, Oregon. Our geotechnical engineering scope of work for this project included the proposed advancement of three geotechnical test borings to a maximum depth of 51½ feet below existing site grades (bgs). The purpose of our services is to provide information and geotechnical engineering recommendations relative to:

- subsurface soil conditions
- foundation settlement
- earthwork
- pavement design parameters
- slope stability
- foundation design and construction
- floor slab design and construction
- seismic site classification
- lateral earth pressure

2.0 PROJECT INFORMATION

2.1 Project Description

ITEM	DESCRIPTION
Site layout	We were provided with a site layout showing an addition to the current Parkland Assisted Living development. The development is located on the north side of the existing structure.
Structures	We understand that the expansion is expected to be a one story development in height with wood- or light gage metal-framed, with concrete slab on-grade floors.
Finish floor elevation	Not known at this time, but assumed to be near existing grades.
Maximum loads, assumed	Columns Footings: 50 to 75 kips maximum total loads (assumed) Walls: 1 to 4 kips/lf maximum total loads (assumed) Floor Slabs: 150 psf (assumed)
Maximum allowable settlement	Total: 1 inch over entire building shell footprint (assumed) Differential: ½ inch over 30 feet (assumed)

ITEM	DESCRIPTION
Grading	Undetermined at this time, but assumed to remain near existing grade.
Cut and fill slopes	None expected.
Pavements	Traffic loads undetermined, but we anticipate conventional asphalt concrete in the ground floor structure covered drive.

2.2 Site Location and Description

ITEM	DESCRIPTION
Location	The expansion site is located on the north side of the current development located at 3121 NE Cumulus Avenue in McMinnville, Oregon (Lat.: 45.203623, Long.: -123.156698).
Existing Improvements	<p>Site: Developed with a single-story Senior and Assisted Living Facility encompassing the central portion of site and asphalt pavements on the remaining southern portion.</p> <p>North: Sloped to Southern Yamhill River tributary</p> <p>South: Residential developments</p> <p>East: Residential developments, then Southern Yamhill River tributary</p> <p>West: Empty field (different proposed development), then residential developments further west</p>
Current ground cover	The ground is covered with grass and small trees.
Existing topography	The site is relatively flat. However, a steep slope approximately XX feet in overall relief is located immediately north and northeast of the site development. The slope is part of an overall drainage ravine for the Yamhill River.

3.0 SUBSURFACE CONDITIONS

A cursory review of historical aerial photographs from Google Earth shows the proposed development area has not been developed.

3.1 Site Geology

The Oregon Department of Geology and Mineral Industries (DOGAMI) published Oregon Geologic Data Compilation-Release 5 (2009) indicates the majority of the site is classified as the medium terrace Missoula Flood deposits (Qmt). Site geology is described as fine grained

sediments. Based on our findings in the subsurface explorations, the site soils encountered are consistent with the above described Missoula Flood channel deposits.

3.1.1 Geologic Hazards

We reviewed the *Statewide Landslide Information Database for Oregon (SLIDO)* published by the Oregon Department of Geology and Mineral Industries (DOGAMI) and updated in 2014. The map also overlays *The Statewide Landslide Susceptibility Overview Map of Oregon (O-16-02)* also published by DOGAMI in 2012. The latter publication presents the landslide susceptibility in low (landsliding unlikely), moderate (landsliding possible), high (landsliding likely), and very high (existing landslide). The slope immediately north of the development is mapped as “high” landslide susceptibility.

3.1.2 Seismic Hazards

Seismic hazards resulting from earthquake motions can include slope instability, liquefaction, and surface rupture due to faulting or lateral spreading. Liquefaction is the phenomenon wherein soil strength is dramatically reduced when subjected to vibration or shaking.

We reviewed the *Relative Earthquake Hazard Maps for Selected Urban Areas in Western Oregon: McMinnville-Dayton-Lafayette (IMS-9)* published by the Oregon Department of Geology and Mineral Industries (DOGAMI) in 2000. The map evaluates the overall earthquake hazard rating based on three earthquake hazards including ground shaking amplification, liquefaction, and slope instability. The mapped categories range from Zone A for the highest overall relative earthquake hazard to Zone D for the lowest rating. Zone A indicates two or more individual earthquake hazards have a high relative hazard rating. Sites mapped as Zone B have a high rating from a single individual earthquake hazard. The subject site is mapped in an area categorized as Zone B due to a mapped high relative liquefaction hazard.

3.2 Typical Subsurface Profile

Based on the results of the borings, subsurface conditions on the project site can be generalized as follows:

Description	Approximate Depth to Bottom of Stratum	Material Encountered	Consistency/Density
Stratum 1 (Topsoil)	4 in.	Grass and root zone	N/A
Stratum 2 (Silt and Sand)	Undetermined; all borings terminated within this stratum at the planned exploration depth (maximum explored depth of 51½ feet)	Silt and Sand Mixtures	Soft to stiff and medium dense

Conditions encountered at each boring location are indicated on the individual boring logs found in Appendix A of this report. Stratification boundaries on the boring logs represent the approximate location of changes in soil types; in-situ, the transition between materials may be gradual. A discussion of field sampling procedures is included in Appendix and laboratory testing procedures and test results are presented in Appendix B.

3.3 Groundwater

Groundwater was observed from 19 to 34 feet bgs in the borings at the time of drilling and 30½ to 41¼ at completion of drilling. Groundwater level fluctuations occur due to seasonal variations in the amount of rainfall, runoff and other factors not evident at the time the borings were completed. Therefore, groundwater levels during construction or at other times may be higher or lower than the levels indicated on the boring logs. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project.

4.0 RECOMMENDATIONS FOR DESIGN AND CONSTRUCTION

4.1 Geotechnical Considerations

The subsurface conditions at the site were evaluated to develop geotechnical related design and construction recommendations for site development. In our opinion, the site is feasible for the proposed development provided the recommendations in this report are followed. Due to the risk of slope instability, we recommend a development (structure and grading) minimum setback of

35 feet from the top of the slope and a geogrid-reinforced fill prism for structure support be incorporated into the project details. The remaining portions of the structure could be supported on conventional spread and continuous footings bearing directly on one foot of compacted structural fill on the native stiff silt or re-compacted native soils.

The near surface native soils at the site are fine-grained and very moisture sensitive. Therefore, these soils will be difficult to reuse if overly moist (as they are in their current state) and should not be planned to be reused as structural fill. In addition, they should not be reused within the upper foot underneath floor slabs and/or footings. Recommendations for backfill are provided in the **Fill Material Types** and **Compaction** sections of this report.

4.2 Slope Stability

The existing slope below the expansion area is at an inclination that varies from approximately 25 to 80 percent and consists of silt soils with sand and organics, which we interpret to be native alluvial soils. The proposed expansion is planned to be constructed as close to the top of the slope as possible.

We evaluated the stability of the proposed slopes using the computer program Slope/W, Version 7.14, by Geo-Slope International. The Morgenstern-Price method with a rotational failure mechanism was selected since factors of safety for this method satisfy both moment and force equilibrium. Input parameters for the analysis consisted of slope geometry, geology, and ground water conditions of the slope, interpreted from our explorations, and available published information. The soil properties used in the slope stability analysis employ the Mohr-Coulomb model and are also shown on the Slope/W results sheets in Appendix D. The soil properties are based on soil strength parameters from laboratory strength testing, correlations to the index tests, SPT blow counts obtained from the borings, and our experience with similar type soils. The slope geometry was developed from plan sheet 1 developed by Civil West Engineering Services, Inc and based on aerial photographic and topographic data available from Google Earth.

In general, the calculated factor of safety is the ratio between the available soil shear resistance and the gravitational forces that tend to produce a slide. When the soil strength is equal to the slide-producing forces, a factor of safety of 1.0 would exist, and the slope would be in a state of incipient failure. An acceptable factor of safety would depend on the level of risk deemed acceptable by the owner and municipality. Typically, a static factor of safety of at least 1.5 is desired from a design standpoint for conditions where a failure could impact occupied structures and is considered acceptable for all slopes. During short-term seismic loading, a dynamic factor of safety of 1.1 is generally considered acceptable.

Seismic slope stability analyses were conducted using a horizontal seismic coefficient of 0.24g. This seismic coefficient is equal to approximately one-half of the peak ground acceleration of 0.47g, as determined for the site using 2010 ASCE 7-10 methods for a maximum considered

earthquake return period of 2,475 years. The use of one-half of the site-specific peak ground acceleration (PGA) value is consistent with the standard of practice for evaluation of slope stability for non-liquefiable soils.

Our analyses indicate that the minimum factor of safety for significant slope failure landslides in the steep slopes area, occurring at or behind the assumed top of slope elevation of 96 feet extend beyond the top of the slope approximately 30 to 35 feet. Therefore, in order to not adversely impact the stability of the slope and to protect the structures from instability, we recommend all development remain a minimum of 35 feet setback from the top of the slope. In addition, we recommend incorporating a geogrid-reinforced fill prism underneath the footings closest to the top of the slope. This fill prism should consist of BX1200 geogrid (or equivalent biaxial strength geogrid) spaced 12 inches vertically within crushed aggregate base materials. The fill prism should have four layers of geogrid. The prism should extend at least 5 feet beyond the extents of the edge of footing in all directions.

Should the setback limits overly constrain site development and the client desire the development to extend closer to the top of the slope, then slope stabilization measures would need to be incorporated in the design of the development. These typically consist of buried piles extending through the potential slide failure plans and can be quite costly. The pile wall improvements would need to be designed to overcome the active or at-rest pressures, depending on foundation set back from improvements, during a static and seismic event.

4.3 Earthwork

The following sections present recommendations for site preparation, excavation, subgrade preparation, placement and compaction of structural fill, and grading. The recommendations presented for design and construction of earth supported elements are contingent upon following the recommendations outlined in this section.

4.3.1 Site Preparation

Site preparation and initial construction activities should be planned to reduce disturbance to the existing ground surface. Construction traffic should be restricted to dedicated driveway and laydown areas. Preparation should begin with procedures intended to drain ponded water and control surface water runoff.

Site preparation will require removing stripping and grubbing of the vegetative layer within the effective development areas. If existing facilities or utility lines are encountered during construction activities, existing features shall be removed within the building pad limits, they should be properly capped at the site perimeter, and the trenches should be backfilled in accordance with structural fill recommendations presented in Sections 4.3.3 and 4.3.4 of this report. If unexpected fills are encountered within proposed development areas, affected areas

should be removed and the excavation thoroughly cleaned prior to backfill placement and/or construction unless evaluated and tested by an authorized Terracon representative.

In the event the exposed subgrade becomes unstable, yielding, or disturbed, we recommend that the materials be removed to a sufficient depth in order to develop stable subgrade soils that can be compacted to the minimum recommended levels. The severity of construction problems will be dependent, in part, on the precautions that are taken by the contractor to protect the subgrade soils.

4.3.2 Subgrade Preparation

Strip and remove existing vegetation, existing fill, topsoil, pavements, and other deleterious materials from the proposed development areas. Existing fill soils may remain within the non-building areas provided they are prepared according the following sections. Stripping depths to remove existing vegetation within the expansion pad are anticipated to be an average of about 4 to 6 inches, but may vary across the site and could be deeper. Areas where loose or soft surface soils exist, they should be compacted or removed and replaced to the depth of the disturbance as subsequently recommended for structural fill.

Excavations for footings should be completed to expose medium stiff silt materials and should be covered with the recommended granular select fill to prevent significant drying. The excavations should be observed for visual classification and T-probing by a representative of Terracon to confirm suitable subgrades for bearing support of foundations.

The upper one foot of pavement subgrades should be scarified and re-compacted to levels described in the **Compaction Requirements** section of this report after cutting to design subgrade elevation. We also recommend testing include proof-rolling to aid in the identification of weak or unstable areas within the near surface soils at the exposed subgrade level. Proof-rolling should be performed using heavy rubber-tired equipment, such as a fully-loaded dump truck, having a minimum gross weight of about 20 tons. Unsuitable areas observed at this time which are soft, yielding, or unable to be compacted to the specified criteria should be over-excavated and replaced with satisfactory fill material later described in section 4.3.3 of this report.

Based on the outcome of the proof-rolling operations, some undercutting or subgrade stabilization may be expected, especially during wet periods of the year. Methods of stabilization, which are outlined below, could include scarification and re-compaction and/or removal of unstable materials and replacement with granular fill (with or without geotextiles). The most suitable method of stabilization, if required, will be dependent upon factors such as schedule, weather, size of area to be stabilized and the nature of the instability.

- **Scarification and Re-compaction** - It may be feasible to scarify, dry, and re-compact the exposed sand soils at the site during periods of dry weather. The success of this procedure would depend primarily upon the extent of the disturbed

area. Stable subgrades may not be achievable if the thickness of the soft soil is greater than about 1 to 1½ feet.

- **Granular Fill** - The use of crushed stone or gravel could be considered to improve subgrade stability. Typical undercut depths would range from about ½ foot to 2 feet. The use of high modulus geotextiles i.e., engineering fabric, should be limited to outside of the Building Ground Improvements area. The maximum particle size of granular material placed immediately over geotextile fabric or geogrid should not exceed 2 inches.
- **Chemical Stabilization** - Improvement of subgrades with Portland cement, lime kiln dust, or Class C fly ash could be considered for unstable and plastic soils. Chemical modification should be performed by a pre-qualified contractor having experience with successfully stabilizing subgrades in the project area on similar sized projects with similar soil conditions.

Over-excavations should be backfilled with structural fill material placed and compacted in accordance with sections 4.3.3 and 4.3.4 of this report. Subgrade preparation and selection, placement, and compaction of structural fill should be performed under engineering controlled conditions in accordance with the project specifications.

4.3.3 Fill Material Types

Engineered or structural fill should meet the following material property requirements:

Fill Type ¹	Specification	Acceptable for Placement
Common Fill ^{2,3}	2015 Oregon Standard Specification for Construction (OSSC) 00330.13 Selected General Backfill with the additional requirements of Liquid Limits < 40 and Plasticity Index < 10	All locations across the site, with the exception of floor and pavement base materials Dry Weather only.
Select Fill ²	OSSC 00330.14 Selected Granular Backfill with exception of no more than 8% passing the No. 200 sieve by weight and reclaimed glass is not acceptable	All locations across the site, Wet Weather and Dry Weather acceptable.
Crushed Aggregate Base (CAB)	OSSC 02630.10 Dense Graded Aggregate (2"-0 to ¾"-0) with exception of no more than 8% passing the No. 200 sieve by weight	All locations across the site. Recommended for finished base course materials for floor slabs and pavements. Wet Weather and Dry Weather acceptable.

Fill Type ¹	Specification	Acceptable for Placement
1.	Controlled, compacted fill should consist of approved materials that are free (free = less than 3% by weight) of organic matter and debris (i.e. wood sticks greater than ¾-inch in diameter). Frozen material should not be used, and fill should not be placed on a frozen subgrade. A sample of each material type should be submitted to the geotechnical engineer for evaluation.	
2.	Materials within 1-foot of floor slabs base, pavement base, and footings should have a maximum particle size of 3-inches.	

If open-graded materials with large void spaces, such as quarry spalls, are used we recommend that the materials be placed over a geotextile fabric separator to prevent fines migration as well as to stabilize the subgrade. The geotextile fabric should be a woven product (Mirafi 500XT or equivalent).

4.3.4 Compaction Requirements

The following compaction requirements are recommended for the prepared subgrade and structural fill expected to be placed for this site:

Item	Description
Fill Lift Thickness	Common Fill, Select Fill and CAB: 10-inches or less in loose thickness when heavy, compaction equipment is used.
Compaction Requirements ¹	Common Fill, Select Fill & CAB: 95% of the material's maximum Proctor dry density (ASTM D1557) below building pad and upper two feet of site pavements. 92% of the materials maximum Proctor dry density (ASTM D1557) elsewhere.
Moisture Content	Common Fill, Select Fill and CAB: Within ±2 percent of optimum moisture content as determined by ASTM D1557.

1. We recommend that fill be tested for moisture content and compaction during placement. Should the results of the in-place density tests indicate the specified moisture or compaction limits have not been met, the area represented by the test should be reworked and retested as required until the specified moisture and compaction requirements are achieved.

4.3.5 On-Site Soils

Our explorations indicated that the on-site soils will likely consist of silt and sand soils. At the time of our exploration, moisture contents in the upper soils zone were found to generally range from approximately 21 to 34 percent, which we infer to be well above their optimum moisture content. Therefore, most on-site soils will likely be reusable only during dry weather if they can be adequately dried, but they will be difficult or impossible to reuse during wet weather. Any zones containing significant amounts of wood, asphalt, or other waste products should be excluded from reuse as structural fill.

4.3.6 Wet-Weather Earthwork

As discussed above, the on-site fine-grained native soils would be difficult to reuse as structural fill during wet weather and are likely precluded from use within the building pad over-excavation.

Consequently, the project specifications should include provisions for using imported, clean, granular fill. As a general structural fill material, we recommend using the crushed aggregate base courses meeting the Oregon Standard Specifications section 02630.10, which are readily available in the region, although some local sources of pit-run or bank-run may be available. The use of high modulus geotextiles (i.e., engineering fabric such as Mirafi HP370) may be used to aid in stabilization of the subgrade. To reduce the potential for subgrade disturbance during wet-weather periods, contractor should install haul roads consisting of clean, crushed rock at a minimum depth of 18 inches. Haul roads install and intended to be incorporated into final pavement section shall be evaluated for conformance with sections 4.3.2 thru 4.3.4 prior to placement of crushed rock.

4.3.7 Construction Considerations

Native fine grained soils were encountered near the surface across the site and were observed to consist of silt and fine sands and in a moisture condition much greater than about 2 percent over optimum moisture content. Therefore, the fine grained site soils are considered to be moisture sensitive and will be difficult or impossible to compact as structural fill. Accordingly, the fine-grained soils from site excavations are not considered suitable as granular fill in footing areas, their use in non-footing areas will depend on their moisture content at the time of earthwork, the prevailing weather conditions when site grading activities take place, and the proposed location for reuse. The onsite granular soils may be suitable for reuse as structural fill in building areas if the material is in accordance with the **Fill Material Types** section of this report.

Even if stable subgrades are exposed during construction, unstable subgrade conditions could develop during general construction operations, particularly if the soils are wetted and/or subjected to repetitive construction traffic. The use of light construction equipment would aid in reducing subgrade disturbance. The use of remotely operated equipment, such as a backhoe, would be beneficial to perform cuts and reduce subgrade disturbance. If the subgrade should become frozen, desiccated, saturated, or disturbed, stabilization measures will need to be employed.

The contractor is responsible for designing and constructing stable, temporary excavations (including utility trenches) as required to maintain stability of both the excavation sides and bottom. Excavations should be sloped or shored in the interest of safety following local and federal regulations, including current OSHA excavation and trench safety standards. Care should be taken when excavating near adjacent structures or right-of-ways. If excavations will encroach below a 1H:1V plane below the foundations of adjacent structures or right-of-ways, the contractor should be prepared to provide temporary shoring designed to resist the structure or traffic surcharge loads.

The geotechnical engineer should be retained during the construction phase of the project to observe earthwork and to perform necessary tests and observations during subgrade preparation,

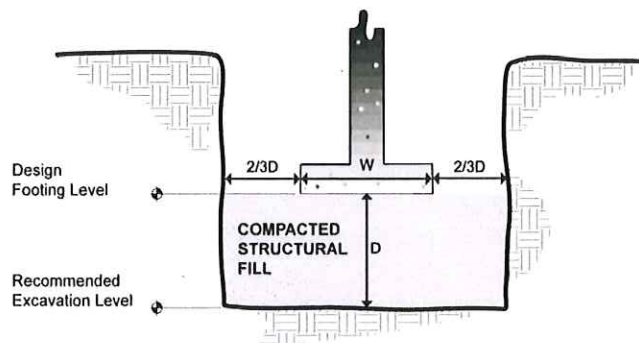
probing, placement and compaction of controlled compacted fills, and backfilling of excavations to the completed subgrade.

4.4 Foundations

The proposed structures may be supported by isolated spread footings and continuous footings bearing on one foot of granular structural fill (Select Fill) over the medium stiff native silts or re-compacted silt with sand subgrade soils. As discussed in the Slope Stability section of the report, the footings closest to the top of the slope should be supported on the geogrid-reinforced structural fill prism. Design recommendations for foundations for the proposed structures and related structural elements are presented in the following sections.

4.4.1 Footing Subgrade Preparation

Unsuitable bearing soils were encountered in the near surface of our explorations. The footing excavation should be extended one foot in depth and be replaced with compacted structural fill. The footings should bear on properly compacted structural backfill extending down to the stiff native soils or scarified and re-compacted subgrade soils to a depth of one foot. Foundations should not be supported on soft or loose soils or existing fill soils. Over-excavation for compacted backfill placement below footings should extend laterally beyond all edges of the footings at least 8 inches per foot of over-excavation depth below footing base elevation. Zones of loose, soft or otherwise unsuitable soil encountered in or below the footing subgrade should be over-excavated and replaced with properly compacted Select Fill.



Overexcavation / Backfill

NOTE: Excavations in sketches shown vertical for convenience. Excavations should be sloped as necessary for safety.

The compactive effort should be in accordance with recommendations provided in the 4.33 Earthwork section of this report.

4.4.2 Design Recommendations

DESCRIPTION	Column	Wall
Net allowable bearing pressure ¹ One foot of granular structural fill placed directly above the competent native soils	2,500 psf	2,500 psf
Minimum dimensions	2 feet	12 inches
Minimum embedment below finished grade for frost protection ²	12 inches	12 inches
Approximate total static settlement ³	<1 inch	<1 inch
Estimated differential settlement ³	<¾ inch between columns	<½ inch over 30 feet
Allowable passive pressure ⁴	230 psf/ft	
Allowable coefficient of sliding friction ⁴	0.33	

1. The recommended net allowable bearing pressure is the pressure in excess of the minimum surrounding overburden pressure at the footing base elevation. Assumes any unsuitable fill or soft soils, if encountered, will be undercut and replaced with structural fill. Assumes native soils will be undercut 1 foot and replaced with structural fill.
2. And to reduce the effects of seasonal moisture variations in the subgrade soils. For exterior footings and footings beneath unheated areas.
3. The foundation settlement will depend upon the variations within the subsurface soil profile, the structural loading conditions, the embedment depth of the footings, the thickness of compacted fill, and the quality of the earthwork operations. The above settlement estimates have assumed that the maximum footing size is 4 feet for column footings and 1.5 foot for continuous footings.
4. The value presented is an equivalent fluid pressure. The sides of the excavation for the spread footing foundation must be nearly vertical and the concrete should be placed neat against these vertical faces for the passive earth pressure values to be valid. Passive resistance in the upper 12 inches of the soil profile should be neglected.

The net allowable bearing pressures presented in the table above may be increased by one-third to resist transient, dynamic loads such as wind or seismic forces. Please note that lateral resistance to footings should be ignored in the upper 12-inches from finish grade.

4.3.2 Footing Drains

We recommend that footings drains be installed around the perimeter of the proposed building at the base of the foundations. Footing drains should consist of a minimum 4-inch diameter, Schedule 40, rigid, perforated PVC pipe placed at the base of the heel of the footing with the perforations facing down. The pipe should be surrounded by a minimum of 4 inches of clean free-draining granular material. We recommend enveloping the drain rock with a non-woven geotextile,

such as Mirafi 140N, or equivalent. Footing drains should be directed toward appropriate storm water drainage facilities. Water from downspouts and surface water should be independently collected and routed to a suitable discharge location.

4.5 Floor Slabs

We understand that the structures typically include construction of slabs-on-grade floors. The following design recommendations are provided for newly constructed concrete slabs.

4.5.1 Design Recommendations

ITEM	DESCRIPTION
Interior floor system	Concrete slab-on-grade.
Base / Capillary Break	6-inches of CAB material ($\frac{3}{4}$ "-0)
Modulus of subgrade reaction	125 pci for point load conditions

1. The concrete slab design should include a capillary break, comprised of free-draining, compacted, granular material, at least 6 inches thick. Free-draining granular material should have less than 5 percent fines (material passing the #200 sieve).

Where appropriate, saw-cut control joints should be placed in the slab to help control the location and extent of cracking. For additional recommendations refer to the ACI Design Manual. Joints or any cracks in pavement areas that develop should be sealed with a water-proof, non-extruding compressible compound specifically recommended for heavy duty concrete pavement and wet environments.

The use of a vapor retarder or barrier should be considered beneath concrete slabs on grade that will be covered with wood, tile, carpet or other moisture sensitive or impervious coverings, or when the slab will support equipment sensitive to moisture. When conditions warrant the use of a vapor retarder, the slab designer and slab contractor should refer to ACI 302 and ACI 360 for procedures and cautions regarding the use and placement of a vapor retarder/barrier.

4.5.2 Construction Considerations

On most project sites, the site grading is accomplished relatively early in the construction phase. Fills are placed and compacted in a uniform manner. However, as construction proceeds, excavations for utilities are made into these areas, rainfall and surface water saturates some areas, heavy traffic from concrete trucks and other delivery vehicles disturbs the subgrade and many surface irregularities are filled in with loose soils to improve trafficability temporarily. As a result, the floor slab subgrades, initially prepared early in the project, should be carefully re-evaluated as the time for floor construction approaches.

4.6 Pavements

4.6.1 Design Recommendations

Traffic patterns and anticipated loading conditions were not available at the time this report was prepared. We anticipate that traffic loads will be produced primarily by automobile traffic and occasional delivery trucks. The thickness of pavements subjected to heavy truck traffic should be determined using expected traffic volumes, vehicle types, and vehicle loads and should be in accordance with local, city or county ordinances.

Pavement thickness can be determined using AASHTO, Asphalt Institute and/or other methods if specific wheel loads, axle configurations, frequencies, and desired pavement life are provided. Terracon can provide thickness recommendations for pavements for loads other than personal vehicles and occasional delivery truck if provided.

Listed below are minimum pavement component thicknesses, which may be used as a guide for pavement systems at the site for typical commercial building traffic patterns. It should be noted that these systems were derived based on general characterization of the subgrade as predominantly fine-grained. No specific testing (such as CBR, resilient modulus test, etc.) was performed for this project to evaluate the support characteristics of the subgrade.

MINIMUM PAVEMENT THICKNESSES		
COMPONENT	Material Thickness, Inches	
	Automobile Parking Areas	Drive Lanes
Asphalt Concrete	3	4
Crushed Aggregate Base (CAB)	8	8

Prior to placement of the CAB the pavement subgrades should be prepared as per the recommendations in the **Earthwork** section of this report. Long term pavement performance will be dependent upon several factors, including maintaining subgrade moisture levels and providing for preventive maintenance. The following recommendations should be considered the minimum:

- The subgrade and the pavement surface have a minimum ¼-inch per foot slope to promote proper surface drainage;
- Consider appropriate edge drainage and pavement under drain systems;
- Install joint sealant and seal cracks immediately;
- Seal all landscaped areas in, or adjacent to pavements to minimize or prevent moisture migration to subgrade soils;
- Placing compacted, low permeability backfill against the exterior side of curb and gutter.

Preventive maintenance should be planned and provided for through an on-going pavement management program. Preventive maintenance activities are intended to slow the rate of pavement deterioration, and to preserve the pavement investment. Preventive maintenance consists of both localized maintenance (e.g. crack and joint sealing and patching) and global maintenance (e.g. surface sealing). Preventive maintenance is usually the first priority when implementing a planned pavement maintenance program and provides the highest return on investment for pavements. Prior to implementing any maintenance, additional engineering observation is recommended to determine the type and extent of preventive maintenance.

As previously stated, haul roads and laydown areas should be included in project planning to provide access to the building area during construction.

4.6.2 Asphalt and Base Course Materials

Specifications for manufacturing and placement of pavements and crushed aggregate base course should conform to specifications presented in Section 00745, of the 2015 Oregon Standard Specifications for Construction. All base course materials should be compacted to at least 95 percent of the maximum dry density determined in accordance with ASTM D1557. We recommend that all base courses be proofrolled with a loaded dump truck prior to placing the following lift of material. We recommend that asphalt be compacted to a minimum of 92 percent of the Rice (theoretical maximum) density.

4.6.3 Concrete Properties and Materials

Concrete pavement design recommendations are based on an assumed modulus of rupture of 580 psi and a minimum 28-day compressive strength of 4,000 psi for the concrete. It is our opinion that concrete pavements should be reinforced and have relatively closely spaced control joints on the order of 15 to 20 feet. We recommend that minimum reinforcement consist of 6x6-W2.0xW2.0 welded wire or equivalent. The welded wire reinforcement should be terminated 3 inches on either side of all construction, contraction and expansion joints. Construction Considerations

4.6.4 Pavement Construction Considerations

On most project sites, the site grading is accomplished relatively early in the construction phase. Fills are placed and compacted in a uniform manner. However, as construction proceeds, excavations are made into these areas, rainfall and surface water saturates some areas, heavy traffic from concrete trucks and other delivery vehicles disturbs the subgrade and many surface irregularities are filled in with loose soils to improve trafficability temporarily. As a result, the pavement subgrades, initially prepared early in the project, should be carefully evaluated as the time for pavement construction approaches.

We recommend the entire pavement subgrade should be scarified and re-compacted as recommended in 4.3 of this report to provide a uniform subgrade for pavement construction. Areas

that appear severely desiccated following site stripping may require further undercutting and moisture conditioning.

After scarification and re-compaction of subgrade soils, moisture content and density of the top 12 inches of the subgrade soils be evaluated and the pavement subgrades be proof-rolled prior to commencement of crushed aggregate base placement. Areas not in compliance with the required ranges of moisture or density should be moisture conditioned and re-compacted. Particular attention should be paid to high traffic areas that were rutted and disturbed earlier and to areas where backfilled trenches are located. Proof-roll testing should be performed by a qualified representative of Terracon at time of subgrade completion. Subgrade soils subjected to proof-roll testing should not exhibit pumping, yielding or deflection of greater than 1 inch in magnitude. Areas where unsuitable conditions are located should be repaired by removing and replacing the materials with properly compacted fills.

If a significant precipitation event occurs after the evaluation of subgrade soils or if the surface becomes disturbed, the subgrade should be reviewed by qualified personnel immediately prior to paving. The subgrade should be in its finished form at the time of the final review.

4.7 Seismic Considerations

DESCRIPTION	VALUE
2012 International Building Code Site Classification (IBC) ¹	D ²
Site Latitude	N 45.203623
Site Longitude	W 123.156698
S _s Spectral Acceleration for a Short Period	0.991
S ₁ Spectral Acceleration for a 1-Second Period	0.466
F _a site coefficient	1.104
F _v site coefficient	1.534
Peak Ground Acceleration (PGA)	0.452
Site Specific Coefficient (F _{PGA})	1.048

1. In general accordance with the *2012 International Building Code*, Table 1613.5.2. IBC Site Class is based on the average characteristics of the upper 100 feet of the subsurface profile.
2. The 2012 International Building Code (IBC) requires a site soil profile determination extending to a depth of 100 feet for seismic site classification. The current scope does not include the required 100 foot soil profile determination. Borings extended to a maximum depth of about 51½ feet, and this seismic site class definition considers that dense soil as noted on the published geologic mapping continues below the maximum depth of the subsurface exploration. Additional exploration to deeper depths would be required to confirm the conditions below the current depth of exploration. Therefore, we would interpret that site soils encountered at the site are representative of the soils to a depth of 100 feet.

Earthquake-Induced Soil Liquefaction

Liquefaction is the phenomenon wherein soil strength is dramatically reduced when subjected to vibration or shaking. Liquefaction generally occurs in saturated, loose sand and soft to medium stiff, low plasticity silt deposits. Based on the soft to medium stiff non-plastic silt with sand soils encountered from approximately 20 to 31½ feet bgs in the borings and depth to groundwater (between 19 and 34 feet), it is our opinion that the risk of liquefaction at the site is low due the moderate plasticity of the remaining site soils and we have therefore classified the site as a Site Class D.

5.0 GENERAL COMMENTS

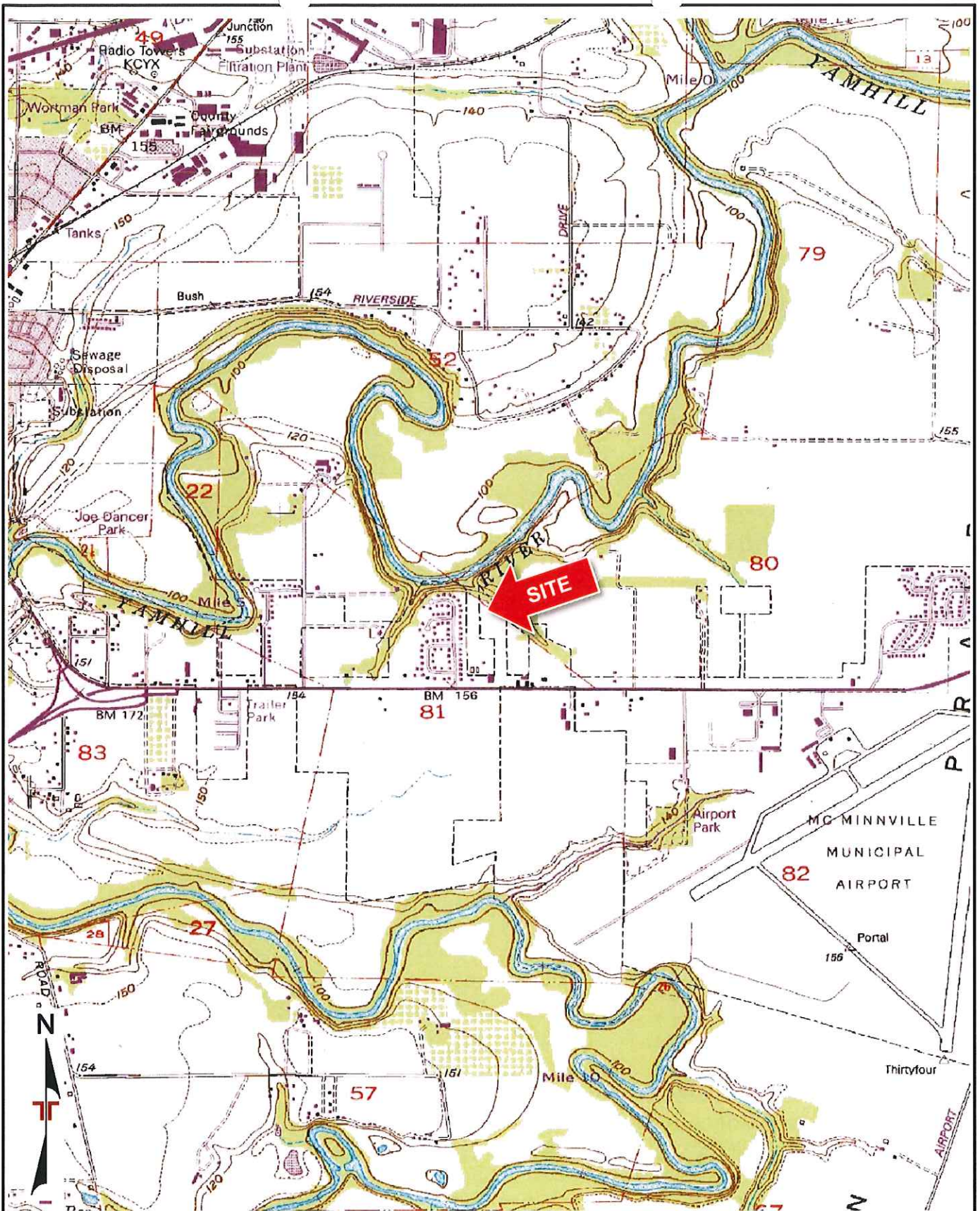
Terracon should be retained to review the final design plans and specifications so comments can be made regarding interpretation and implementation of our geotechnical recommendations in the design and specifications. Terracon also should be retained to provide observation and testing services during grading, excavation, foundation construction and other earth-related construction phases of the project.

The analysis and recommendations presented in this report are based upon the data obtained from the borings performed at the indicated locations and from other information discussed in this report. This report does not reflect variations that may occur between borings, across the site, or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. If variations appear, we should be immediately notified so that further evaluation and supplemental recommendations can be provided.

The scope of services for this project does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

This report has been prepared for the exclusive use of our client for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, either express or implied, are intended or made. Site safety, excavation support, and dewatering requirements are the responsibility of others. In the event that changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless Terracon reviews the changes and either verifies or modifies the conclusions of this report in writing.

APPENDIX A
FIELD EXPLORATION



Project Manager:
KTH

Drawn by:
JAE

Checked by:
BWP

Approved by:
KTH

Project No.
82165034

Scale:
N.T.S

File Name:
EXH. A-1

Date:
APRIL 2016

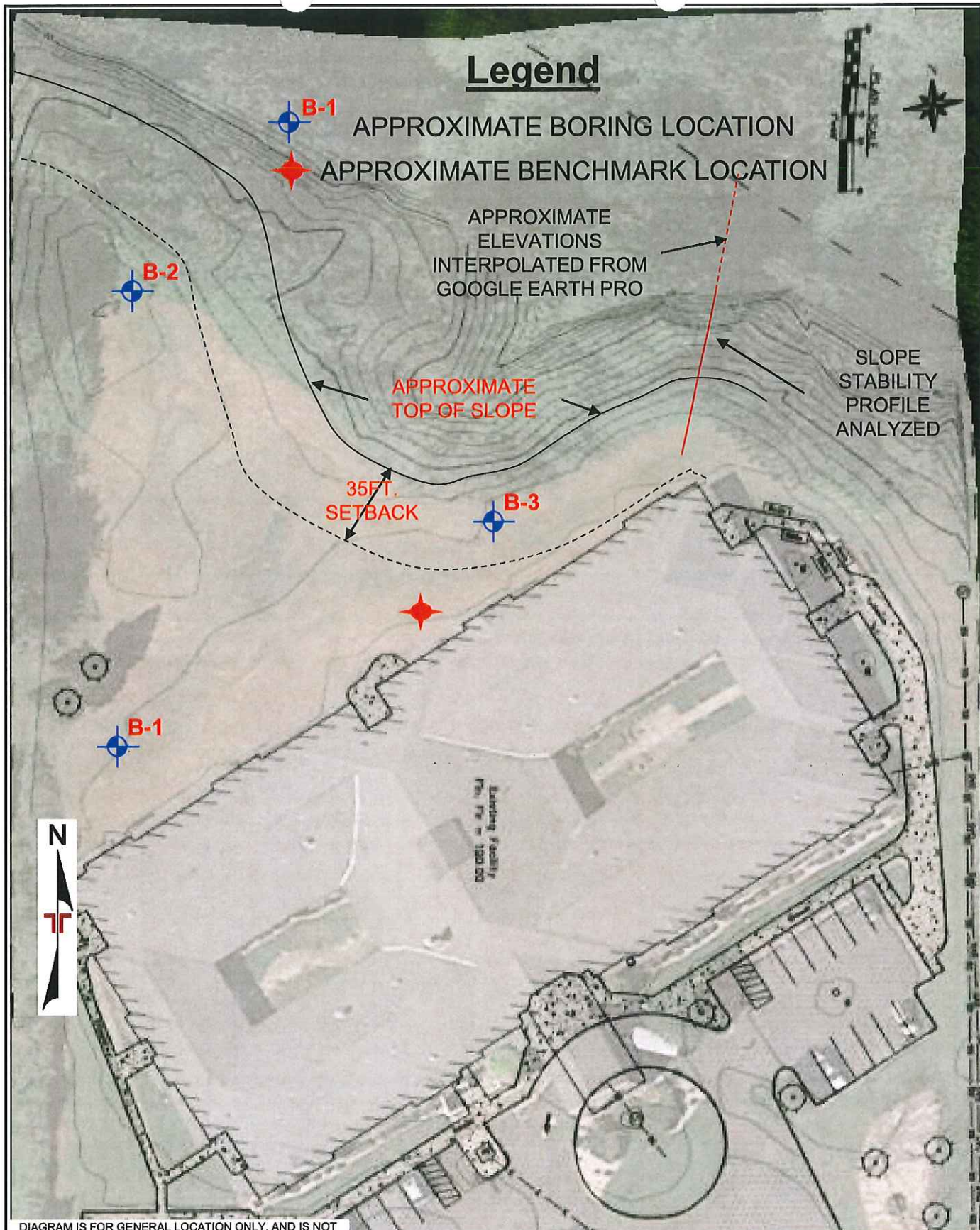
Terracon

4103 SE International Way, Suite 300
Portland, Oregon 97222

SITE LOCATION

PARKLAND EXPANSION
3123 NE CUMULUS AVENUE
MCMINNVILLE, OR

Exhibit
A-1



Project Manager: KTH
 Drawn by: JAE
 Checked by: BWP
 Approved by: KTH

Project No. 82165034
 Scale: N.T.S
 File Name: EXH. A-2
 Date: JUNE 2016

Terracon
 4103 SE International Way, Suite 300
 Portland, Oregon 97222

SITE AND EXPLORATION PLAN
 PARKLAND EXPANSION
 3123 NE CUMULUS AVENUE
 MCMINNVILLE, OR

Exhibit
A-2

Field Exploration Description

The boring locations were located in the field by Terracon personnel based on estimated dimensions from site features and the provided site plan by RJ Development. Terracon personnel estimated ground surface elevations of the borings (based on a site specific assumed elevation of 100 feet at the irrigation control valve on the north side of the property; see attached exhibit A-2) by using an engineer's level and rod. The locations and elevations of the borings should be considered accurate only to the degree implied by the means and methods used to define them.

The borings were drilled with a track mounted hollow-stem auger drill rig under subcontract to Terracon. A field engineer from our firm continuously observed the borings, logged the subsurface conditions, and obtained representative soil samples. Samples of the soil encountered in the borings were obtained using the split barrel and thin-walled tube sampling procedures. The samples were stored in moisture tight containers and transported to our laboratory for further visual classification and testing. After we logged each boring, the operator backfilled each boring in general conformance with local regulations and patched the surface to match the existing ground surface.

In the split-barrel sampling procedure, the number of blows required to advance a standard 2-inch O.D. split-barrel sampler the last 12 inches of the typical total 18-inch penetration by means of a 140-pound auto-hammer with a free fall of 30 inches, is the standard penetration resistance value (SPT-N). This value is used to estimate the in-situ relative density of cohesionless soils and consistency of cohesive soils. An automatic safety hammer used to advance the split-barrel sampler in the borings performed on this site.

In the thin-walled tube sampling procedure, a thin-walled, seamless steel tube with a sharp cutting edge is pushed hydraulically into the soil to obtain a relatively undisturbed sample. The samples were tagged for identification, sealed to reduce moisture loss, and taken to our laboratory for further examination, testing, and classification. Information provided on the boring logs attached to this report includes soil descriptions, consistency evaluations, boring depths, sampling intervals, and groundwater conditions.

A field log of each boring was prepared by the field engineer. These logs included visual classifications of the materials encountered during drilling as well as the driller's interpretation of the subsurface conditions between samples. Final boring logs included with this report represent the engineer's interpretation of the field logs and include modifications based on laboratory observation and tests of the samples.

BORING LOG NO. B-1

PROJECT: Parkland Assited Living Expansion

CLIENT:

SITE:

McMinnville, Oregon

GRAPHIC LOG	LOCATION See Exhibit A-2 Latitude: 45.20384° Longitude: -123.15731°	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (in.)	FIELD TEST RESULTS	STRENGTH TEST			WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
							TEST TYPE	COMPRESSIVE STRENGTH (psf)	STRAIN (%)				
0.3	TOPSOIL , 3-inch Grass and Root Zone			X	8	2-5-8 N=13				22			
2.5	SILT WITH SAND (ML) , brown with orange mottling, stiff			X	12	2-2-3 N=5				26			
	SANDY SILT (ML) , gray with orange mottling, medium stiff			X	15	3-4-6 N=10				34			
	stiff	5		X	18	4-5-6 N=11				32			
	brownish gray			X	18	3-4-5 N=9				34			
	light brown, medium stiff, trace mica	15		X	15	3-3-3 N=6				35			
		20	▽							35	86		
	medium stiff to stiff			X	15	2-2-6 N=8				37			
		25		X	15	2-3-4 N=7				43			
	Boring Terminated at 26.5 Feet												

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic SPT Hammer

Advancement Method:
Hollow stem auger

See Exhibit A-3 for description of field procedures.
See Appendix B for description of laboratory procedures and additional data (if any).

Notes:

Abandonment Method:
Borings backfilled with bentonite chips upon completion

See Appendix C for explanation of symbols and abbreviations.

WATER LEVEL OBSERVATIONS
▽ While drilling



Boring Started: 4/5/2016	Boring Completed: 4/5/2016
Drill Rig: D-50 track	Driller: Terracon
Project No.: 82165034	Exhibit: A-4

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 82165034 BORING LOGS.GPJ TERRACON2015.GDT 5/12/16

BORING LOG NO. B-2

PROJECT: Parkland Assited Living Expansion

CLIENT:

SITE:

McMinnville, Oregon

GRAPHIC LOG	LOCATION See Exhibit A-2 Latitude: 45.20433° Longitude: -123.1572°	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (in.)	FIELD TEST RESULTS	STRENGTH TEST			WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
							TEST TYPE	COMPRESSIVE STRENGTH (psf)	STRAIN (%)				
0.3	TOPSOIL , 3-inch Grass and Root Zone				8	1-1-2 N=3				28			
	SILT WITH SAND (ML) , brown, soft												
	medium stiff				15	1-2-3 N=5				28	32-23-9		
	soft, trace mica	5			10	1-1-2 N=3				35			
7.5	SANDY SILT (ML) , brown with orange mottling, medium stiff to stiff, low plasticity				18	2-3-5 N=8				37			
		10					UC	1032	0.9	36	84		
					15	2-2-4 N=6				37			
	soft	15			18	1-2-2 N=4				36			
		20			12	1-2-3 N=5				38			
	medium stiff	25			12	2-2-4 N=6				37			

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic SPT Hammer

Advancement Method:
Hollow stem auger

See Exhibit A-3 for description of field procedures.
See Appendix B for description of laboratory procedures and additional data (if any).
See Appendix C for explanation of symbols and abbreviations.

Notes:

Abandonment Method:
Borings backfilled with bentonite chips upon completion

WATER LEVEL OBSERVATIONS

- While drilling
- At completion of drilling



Boring Started: 4/4/2016

Boring Completed: 4/5/2016

Drill Rig: D-50 track

Driller: Terracon

Project No.: 82165034

Exhibit: A-5

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 82165034 BORING LOGS.GPJ TERRACON2015.GDT 5/12/16

BORING LOG NO. B-2

PROJECT: Parkland Assited Living Expansion

CLIENT:

SITE:

McMinville, Oregon

GRAPHIC LOG	LOCATION See Exhibit A-2 Latitude: 45.20433° Longitude: -123.1572°	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	STRENGTH TEST			WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
							TEST TYPE	COMPRESSIVE STRENGTH (psf)	STRAIN (%)				
31.5	SANDY SILT (ML) , brown with orange mottling, medium stiff to stiff, low plasticity (<i>continued</i>)	30											
31.5		31.5	▽	X	10	4-5-8 N=13			1743	13	35	87	NP
35	SILTY SAND (SM) , fine grained, gray, medium dense, trace mica	35		X		2-6-9 N=15					38		97
40		40	▽	X		3-4-6 N=10					39		
45	SANDY SILT (ML) , gray, stiff	45		X		2-4-5 N=9					36		
50	medium stiff to stiff	50		X		4-4-4 N=8					43		
51.5	Boring Terminated at 51.5 Feet	51.5											

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic SPT Hammer

Advancement Method:
Hollow stem auger

See Exhibit A-3 for description of field procedures.
See Appendix B for description of laboratory procedures and additional data (if any).
See Appendix C for explanation of symbols and abbreviations.

Notes:

Abandonment Method:
Borings backfilled with bentonite chips upon completion

WATER LEVEL OBSERVATIONS

- ▽ While drilling
- ▽ At completion of drilling



Boring Started: 4/4/2016

Boring Completed: 4/5/2016

Drill Rig: D-50 track

Driller: Terracon

Project No.: 82165034

Exhibit: A-5

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL_82165034 BORING LOGS.GPJ TERRACON2015.GDT 5/12/16

BORING LOG NO. B-3

PROJECT: Parkland Assited Living Expansion

CLIENT:

SITE:

McMinnville, Oregon

GRAPHIC LOG	LOCATION See Exhibit A-2 Latitude: 45.20404° Longitude: -123.15669°	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (in.)	FIELD TEST RESULTS	STRENGTH TEST			WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
							TEST TYPE	COMPRESSIVE STRENGTH (psf)	STRAIN (%)				
	0.3	TOPSOIL , 3-inch Grass and Root Zone SILT WITH SAND (ML) , brown, stiff				2-7-7 N=14			22				
		medium stiff				2-3-3 N=6			22				
		trace sand, brownish gray				1-2-3 N=5			25				
								UC					
	9.0	SANDY SILT (ML) , grayish brown, stiff				2-4-5 N=9			33				
	gray to brown, medium stiff, trace mica				2-2-3 N=5			39					
	gray, soft				3-2-2 N=4			35					
	grayish brown with orange mottling				2-2-3 N=5			37					

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic SPT Hammer

Advancement Method: Hollow stem auger	See Exhibit A-3 for description of field procedures. See Appendix B for description of laboratory procedures and additional data (if any). See Appendix C for explanation of symbols and abbreviations.	Notes:
Abandonment Method: Borings backfilled with bentonite chips upon completion		
WATER LEVEL OBSERVATIONS	4103 SE International Way Ste 300 Portland, OR	Boring Started: 4/4/2016
▽ While drilling		Boring Completed: 4/4/2016
▽ At completion of drilling		Driller: Terracon
		Project No.: 82165034
		Exhibit: A-6

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL 82165034 BORING LOGS.GPJ TERRACON2015.GDT 5/12/16

BORING LOG NO. B-3

PROJECT: Parkland Assited Living Expansion

CLIENT:

SITE:

McMinnville, Oregon

GRAPHIC LOG	LOCATION See Exhibit A-2 Latitude: 45.20404° Longitude: -123.15669°	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	STRENGTH TEST			WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
							TEST TYPE	COMPRESSIVE STRENGTH (psf)	STRAIN (%)				
	SANDY SILT (ML) , grayish brown, stiff <i>(continued)</i>												
		30	▽	X		2-2-3 N=5			36				
		35	▽	X		2-2-3 N=5			39				
	gray	40		X		2-2-3 N=5			37		39-25-14		
		45		X		4-7-6 N=13			34				
		50		X		5-5-9 N=14			36				
	SILTY SAND (SM) , fine grained, dark gray, medium dense												
	Boring Terminated at 51.5 Feet												

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic SPT Hammer

Advancement Method:
Hollow stem auger

See Exhibit A-3 for description of field procedures.
See Appendix B for description of laboratory procedures and additional data (if any).
See Appendix C for explanation of symbols and abbreviations.

Notes:

Abandonment Method:
Borings backfilled with bentonite chips upon completion

WATER LEVEL OBSERVATIONS

- ▽ While drilling
- ▽ At completion of drilling



Boring Started: 4/4/2016

Boring Completed: 4/4/2016

Drill Rig: D-50 track

Driller: Terracon

Project No.: 82165034

Exhibit: A-6

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL_ 82165034 BORING LOGS.GPJ TERRACON2015.GDT 5/12/16

APPENDIX B
LABORATORY TESTING

Laboratory Testing

As part of the testing program, all samples were examined in the laboratory by experienced personnel and classified in accordance with the attached General Notes and the Unified Soil Classification System based on the texture and plasticity of the soils. The group symbol for the Unified Soil Classification System is shown in the appropriate column on the boring logs and a brief description of the classification system is included with this report in the Appendix.

At that time, the field descriptions were confirmed or modified as necessary and an applicable laboratory testing program was formulated to determine engineering properties of the subsurface materials.

Laboratory tests were conducted on selected soil samples and the test results are presented in this appendix. The laboratory test results were used for the geotechnical engineering analyses, and the development of foundation and earthwork recommendations. Laboratory tests were performed in general accordance with the applicable ASTM, local or other accepted standards.

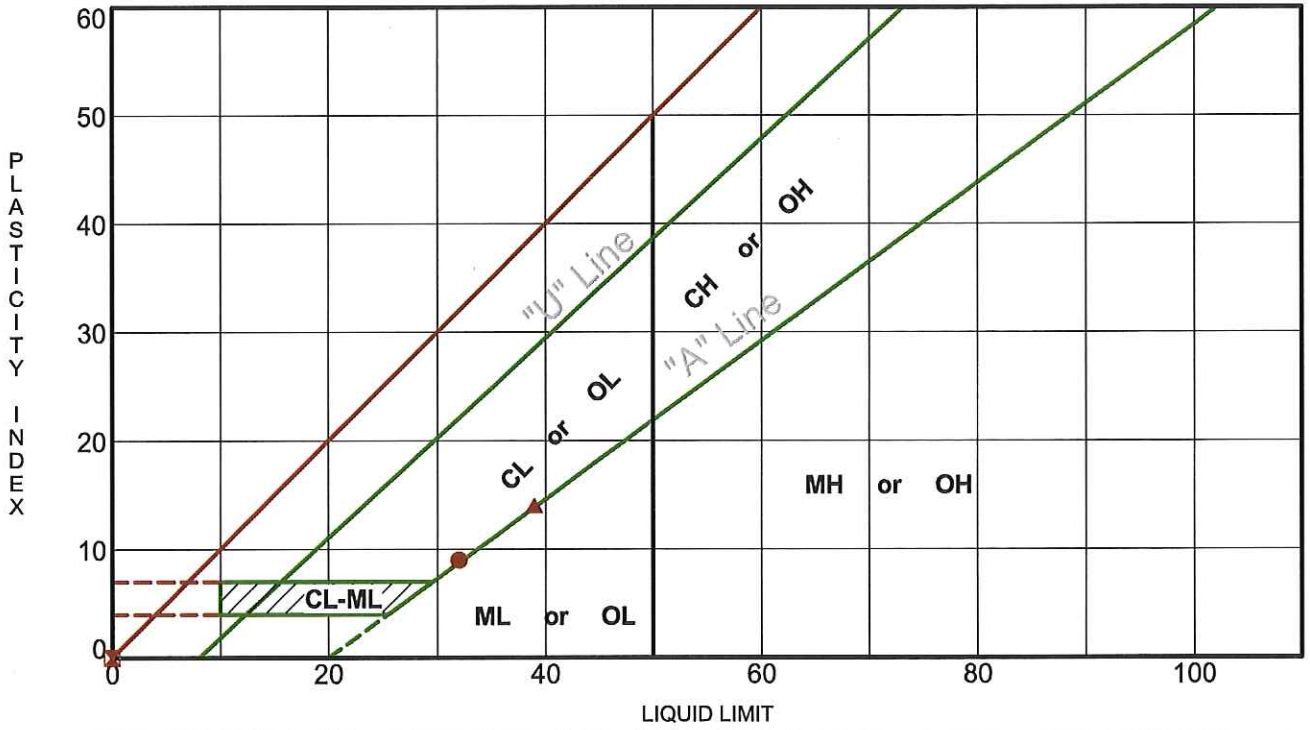
Selected soil samples obtained from the site were tested for the following engineering properties:

- In-situ Water Content (ASTM D 2216)
- Atterberg Limits (ASTM D 4318)
- Fines Content (passing No. 200 sieve) Determination (ASTM D 1140)
- Unconfined Compression Results (ASTM D 2166)
- Direct Shear Results (ASTM D 3080)

It is important to note that the site soils generally contain particles larger than 2 inches in diameter. Due to the sampling equipment being limited in diameter (1.8-inches), the grain size analyses are completed on materials that were able to be sampled. Therefore, the grain size analyses should be considered to be the materials passing a 2-inch sieve and not necessarily representative of the entire subsurface materials matrix.

ATTERBERG LIMITS RESULTS

ASTM D4318



LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. ATTERBERG LIMITS 82165034 BORING LOGS.GPJ TERRACON2015.GDT 5/12/16

Boring ID	Depth	LL	PL	PI	Fines	USCS	Description
● B-2	2.5 - 4	32	23	9			
☒ B-2	30 - 31.5	NP	NP	NP			
▲ B-3	40 - 41.5	39	25	14			

PROJECT: Parkland Assited Living Expansion

SITE:
McMinnville, Oregon



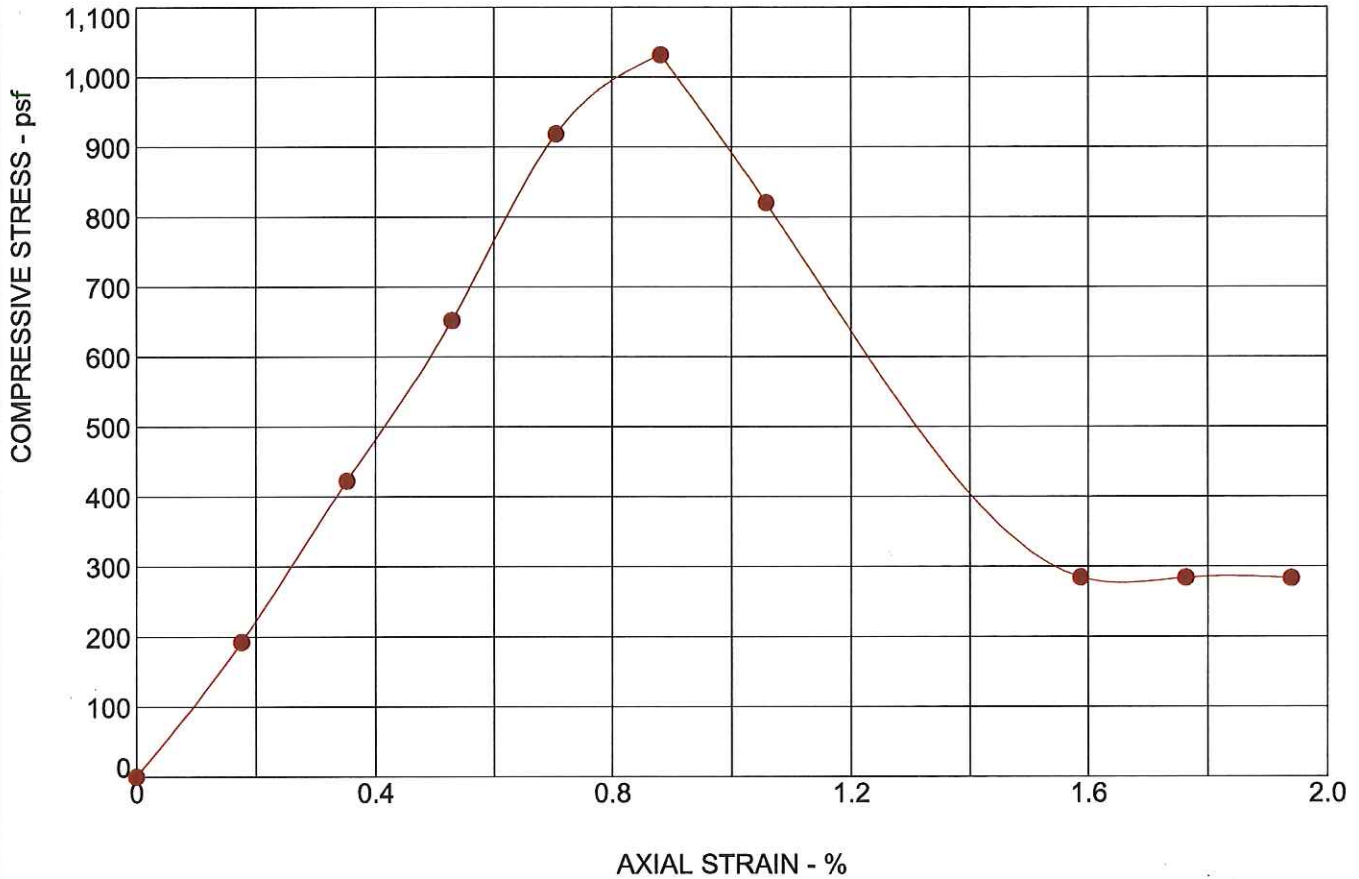
PROJECT NUMBER: 82165034

CLIENT:

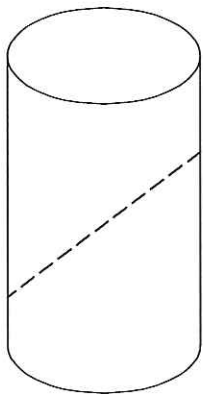
EXHIBIT: B-2

UNCONFINED COMPRESSION TEST

ASTM D2166



SPECIMEN FAILURE MODE



Failure Mode: Shear (dashed)

SPECIMEN TEST DATA

Moisture Content:	%	36
Dry Density:	pcf	83
Diameter:	in.	2.87
Height:	in.	5.67
Height / Diameter Ratio:		1.98
Calculated Saturation:	%	
Calculated Void Ratio:		
Assumed Specific Gravity:		
Failure Strain:	%	0.88
Unconfined Compressive Strength	(psf)	1032
Undrained Shear Strength:	(psf)	516
Strain Rate:	in/min	0.0800
Remarks:		

SAMPLE TYPE: Shelby Tube

SAMPLE LOCATION: B-2 @ 10 - 11.5 feet

DESCRIPTION:

LL

PL

PI

Percent < #200 Sieve

PROJECT: Parkland Assited Living Expansion

PROJECT NUMBER: 82165034

SITE:
McMinnville, Oregon

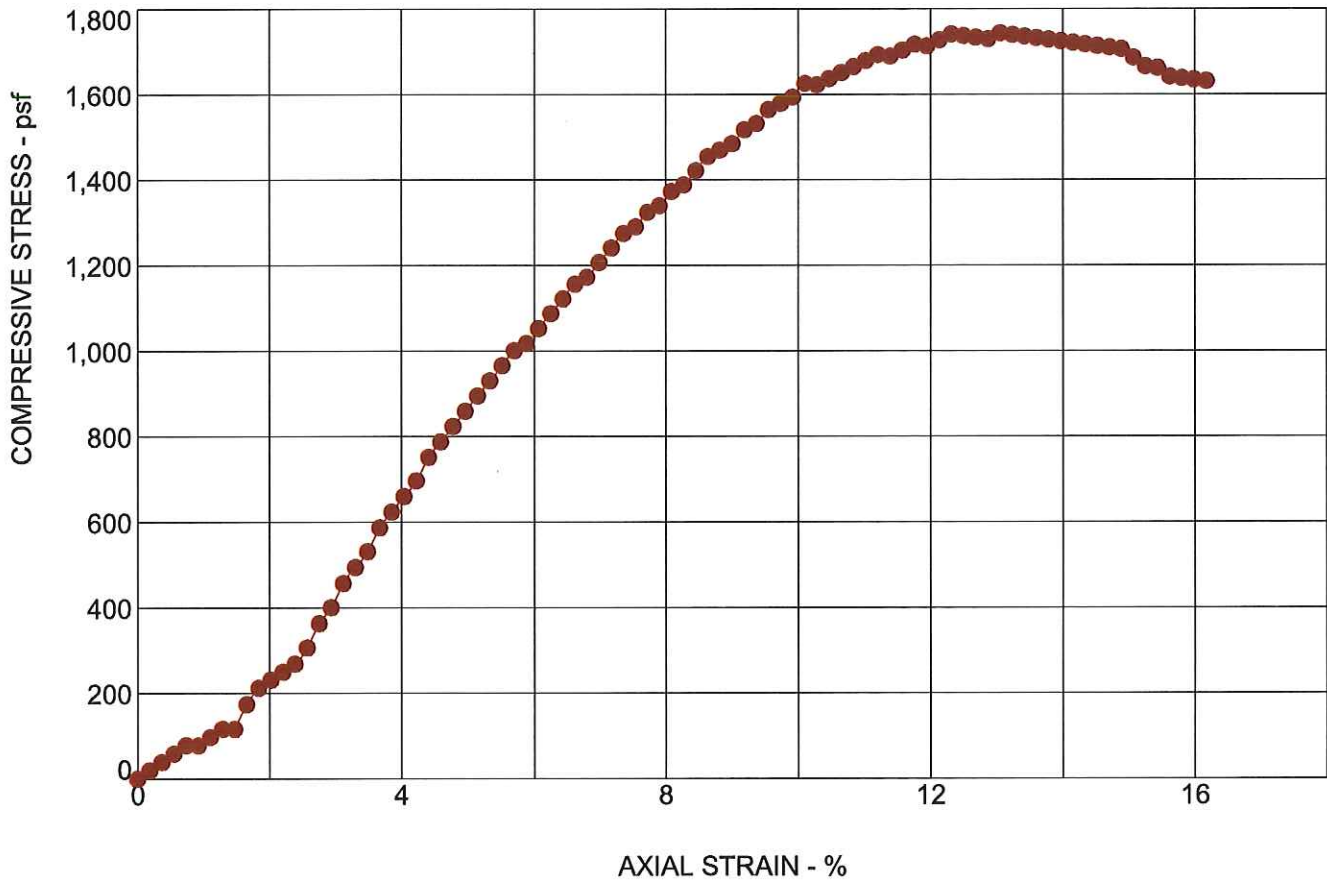
Terracon
4103 SE International Way Ste 300
Portland, OR

CLIENT:

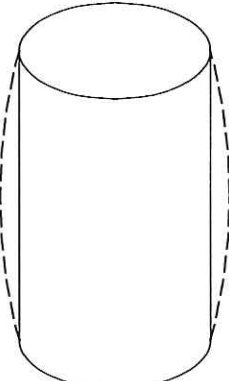
EXHIBIT: B-3

UNCONFINED COMPRESSION TEST

ASTM D2166



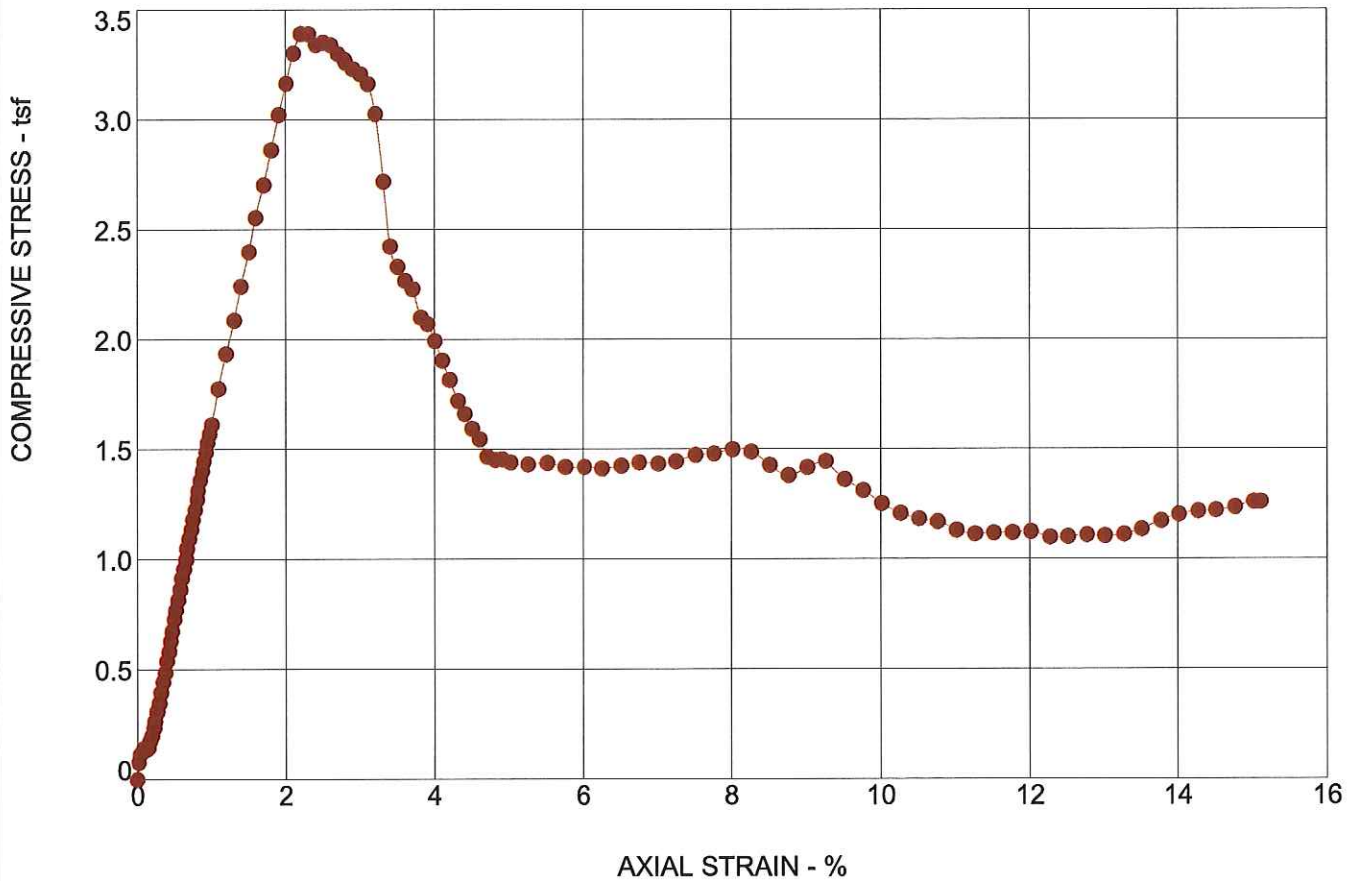
LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. UNCONFINED 82165034 BORING LOGS.GPJ TERRACON2012.GDT 5/12/16

SPECIMEN FAILURE MODE	SPECIMEN TEST DATA																																							
 <p style="text-align: center;">Failure Mode: Bulge (dashed)</p>	<table border="1" style="width: 100%; border-collapse: collapse;"> <tr><td>Moisture Content:</td><td style="text-align: right;">%</td><td style="text-align: right;">35</td></tr> <tr><td>Dry Density:</td><td style="text-align: right;">pcf</td><td style="text-align: right;">87</td></tr> <tr><td>Diameter:</td><td style="text-align: right;">in.</td><td style="text-align: right;">2.84</td></tr> <tr><td>Height:</td><td style="text-align: right;">in.</td><td style="text-align: right;">5.44</td></tr> <tr><td>Height / Diameter Ratio:</td><td></td><td style="text-align: right;">1.92</td></tr> <tr><td>Calculated Saturation:</td><td style="text-align: right;">%</td><td></td></tr> <tr><td>Calculated Void Ratio:</td><td></td><td></td></tr> <tr><td>Assumed Specific Gravity:</td><td></td><td></td></tr> <tr><td>Failure Strain:</td><td style="text-align: right;">%</td><td style="text-align: right;">13.05</td></tr> <tr><td>Unconfined Compressive Strength</td><td style="text-align: right;">(psf)</td><td style="text-align: right;">1743</td></tr> <tr><td>Undrained Shear Strength:</td><td style="text-align: right;">(psf)</td><td style="text-align: right;">872</td></tr> <tr><td>Strain Rate:</td><td style="text-align: right;">in/min</td><td style="text-align: right;">0.0857</td></tr> <tr><td>Remarks:</td><td></td><td></td></tr> </table>	Moisture Content:	%	35	Dry Density:	pcf	87	Diameter:	in.	2.84	Height:	in.	5.44	Height / Diameter Ratio:		1.92	Calculated Saturation:	%		Calculated Void Ratio:			Assumed Specific Gravity:			Failure Strain:	%	13.05	Unconfined Compressive Strength	(psf)	1743	Undrained Shear Strength:	(psf)	872	Strain Rate:	in/min	0.0857	Remarks:		
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Undrained Shear Strength:	(psf)	872																																						
Strain Rate:	in/min	0.0857																																						
Remarks:																																								

SAMPLE TYPE: Shelby Tube	SAMPLE LOCATION: B-2 @ 30 - 31.5 feet								
DESCRIPTION:	<table border="1" style="width: 100%; border-collapse: collapse;"> <tr> <td style="width: 25%;">LL</td> <td style="width: 25%;">PL</td> <td style="width: 25%;">PI</td> <td style="width: 25%;">Percent < #200 Sieve</td> </tr> <tr> <td>NP</td> <td>NP</td> <td>NP</td> <td></td> </tr> </table>	LL	PL	PI	Percent < #200 Sieve	NP	NP	NP	
LL	PL	PI	Percent < #200 Sieve						
NP	NP	NP							
PROJECT: Parkland Assited Living Expansion	PROJECT NUMBER: 82165034								
SITE: McMinnville, Oregon	CLIENT:								
 <p style="text-align: center;">4103 SE International Way Ste 300 Portland, OR</p>	EXHIBIT: B-4								

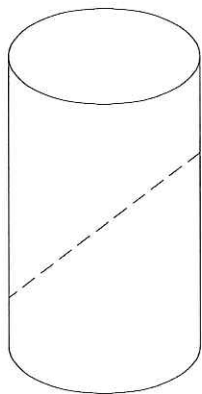
UNCONFINED COMPRESSION TEST

ASTM D2166



LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. UNCONFINED 82165034.GPJ TERRACON2012.GDT 5/3/16

SPECIMEN FAILURE MODE



Failure Mode: Shear (dashed)

SPECIMEN TEST DATA

Moisture Content:	%	31
Dry Density:	pcf	91
Diameter:	in.	2.86
Height:	in.	5.58
Height / Diameter Ratio:		1.95
Calculated Saturation:	%	96.68
Calculated Void Ratio:		0.88
Assumed Specific Gravity:		2.75
Failure Strain:	%	2.21
Unconfined Compressive Strength	(tsf)	3.39
Undrained Shear Strength:	(tsf)	1.70
Strain Rate:	in/min	0.0560
Remarks:		

SAMPLE TYPE: Shelby Tube

SAMPLE LOCATION: B-3 @ 7.5 - 9 feet

DESCRIPTION: Gray and Brown Silty Clay

LL

PL

PI

Percent < #200 Sieve

PROJECT: Parkland Expansion

PROJECT NUMBER: 82165034

SITE: McMinnville, OR

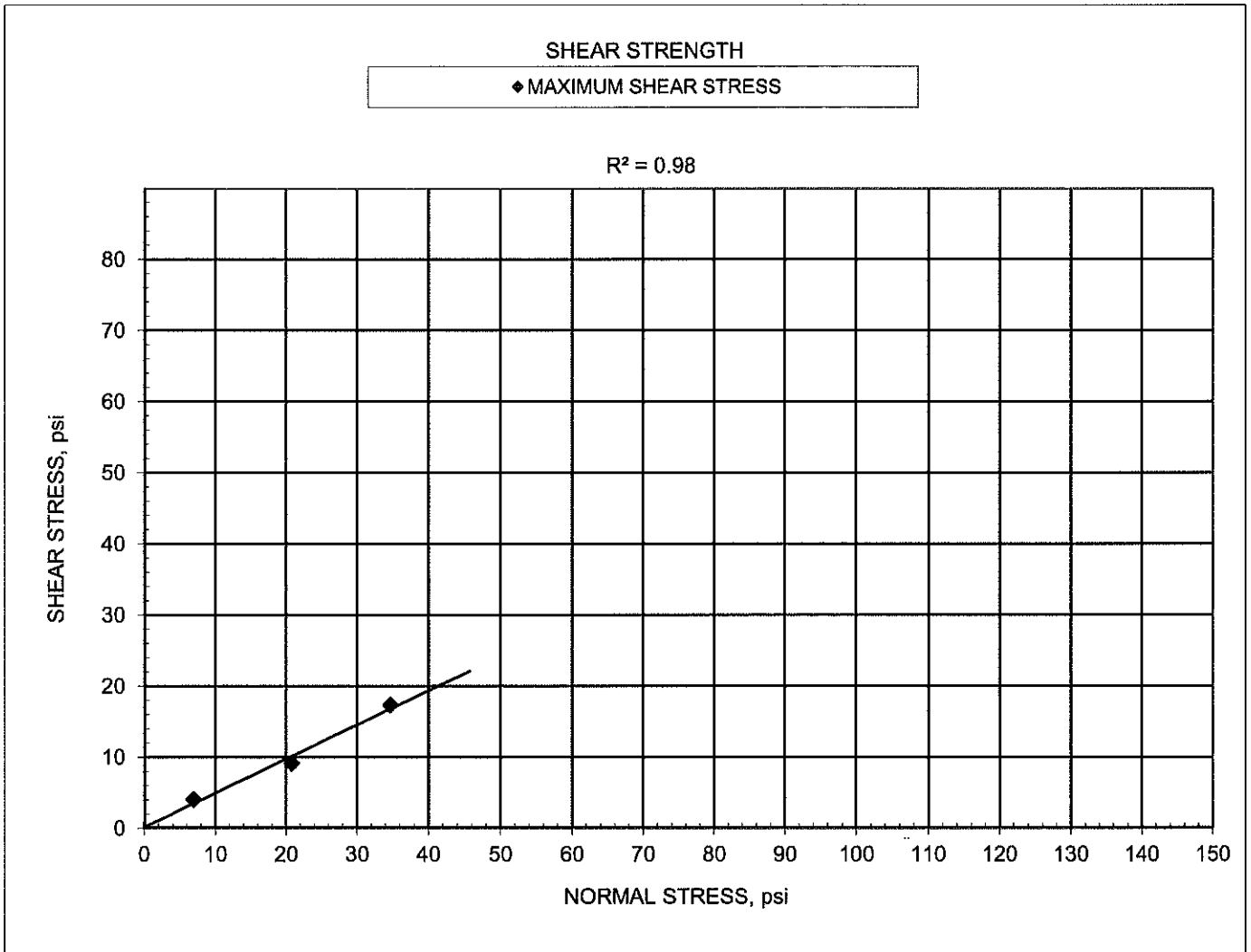
Terracon

CLIENT: RJ Development Services

51 Lost Mound Dr Ste 135
Chattanooga, TN

B-5

**DIRECT SHEAR TEST OF SOILS UNDER CONSOLIDATED DRAINED CONDITIONS
ASTM D3080**



The reported cohesion may be apparent cohesion.

		FRICITION ANGLE	COHESION	NORMAL STRESS, psi	NORMAL STRESS, psi	NORMAL STRESS, psi
AT MAXIMUM SHEAR STRESS		25.6 deg	0.2 psi	6.9	20.8	34.7
INITIAL AREA, mm ²	3166.9	INITIAL MOISTURE, %		33.8	34.3	32.9
INITIAL LENGTH, mm	25.40	INITIAL DRY DENSITY, pcf		83.2	78.5	85.6
SPECIFIC GRAVITY	2.70	INITIAL SATURATION, %		89	81	92
SG TESTED		INITIAL VOID RATIO		1.03	1.15	0.97
SG ASSUMED	X	FINAL MOISTURE, %		38.6	40.7	26.5
LIQUID LIMIT	X	FINAL SATURATION, %		100	99	99
PLASTIC LIMIT	X	FINAL VOID RATIO		1.04	1.11	0.72
PLASTICITY INDEX	X	MAXIMUM SHEAR STRESS, psi		4.03	9.12	17.34
SAMPLE TYPE	SHELBY TUBE	RATE OF LOADING, in/min		0.0018	0.0018	0.0018
DESCRIPTION	Gray and Brown Silty Clay					

PROJECT NAME: Parkland Expansion

BORING NO. B-3

LOCATION: McMinnville, OR

SAMPLE NO. S-4

JOB NO.: 82165034

DEPTH, feet 7.5 TO 9

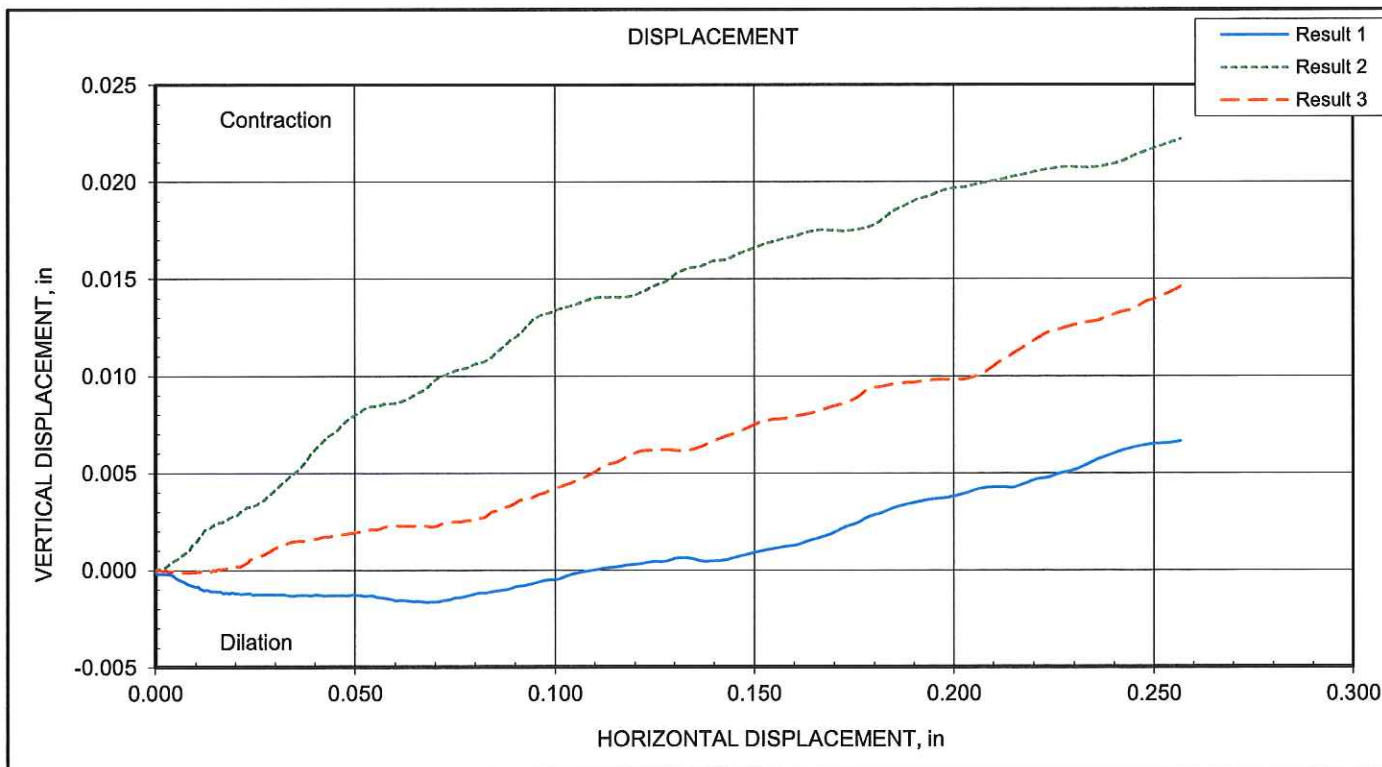
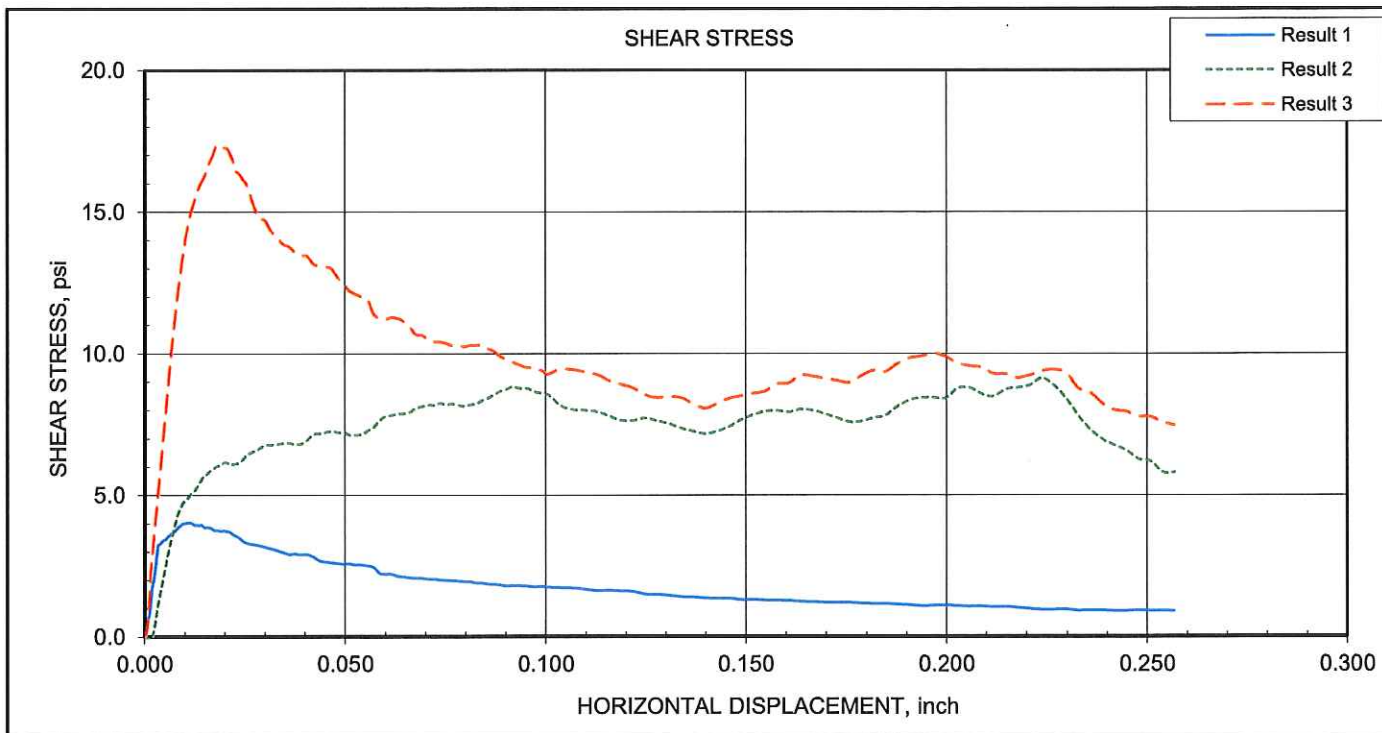
DATE: 5/3/2016

B-6

Terracon

Parkland Expansion
 McMinnville, OR
 82165034
 5/3/2016

BORING NO. B-3
 SAMPLE NO. S-4
 DEPTH, feet 7.5 TO 9



APPENDIX C
SUPPORTING DOCUMENT

GENERAL NOTES

DESCRIPTION OF SYMBOLS AND ABBREVIATIONS

SAMPLING	Shelby Tube Standard Penetration Test	WATER LEVEL	Water Initially Encountered Water Level After a Specified Period of Time Water Level After a Specified Period of Time	FIELD TESTS	N Standard Penetration Test Resistance (Blows/Ft.) (HP) Hand Penetrometer (T) Torvane (DCP) Dynamic Cone Penetrometer (PID) Photo-Ionization Detector (OVA) Organic Vapor Analyzer
			Water levels indicated on the soil boring logs are the levels measured in the borehole at the times indicated. Groundwater level variations will occur over time. In low permeability soils, accurate determination of groundwater levels is not possible with short term water level observations.		

DESCRIPTIVE SOIL CLASSIFICATION

Soil classification is based on the Unified Soil Classification System. Coarse Grained Soils have more than 50% of their dry weight retained on a #200 sieve; their principal descriptors are: boulders, cobbles, gravel or sand. Fine Grained Soils have less than 50% of their dry weight retained on a #200 sieve; they are principally described as clays if they are plastic, and silts if they are slightly plastic or non-plastic. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size. In addition to gradation, coarse-grained soils are defined on the basis of their in-place relative density and fine-grained soils on the basis of their consistency.

LOCATION AND ELEVATION NOTES

Unless otherwise noted, Latitude and Longitude are approximately determined using a hand-held GPS device. The accuracy of such devices is variable. Surface elevation data annotated with +/- indicates that no actual topographical survey was conducted to confirm the surface elevation. Instead, the surface elevation was approximately determined from topographic maps of the area.

STRENGTH TERMS	RELATIVE DENSITY OF COARSE-GRAINED SOILS		CONSISTENCY OF FINE-GRAINED SOILS		
	(More than 50% retained on No. 200 sieve.) Density determined by Standard Penetration Resistance		(50% or more passing the No. 200 sieve.) Consistency determined by laboratory shear strength testing, field visual-manual procedures or standard penetration resistance		
	Descriptive Term (Density)	Standard Penetration or N-Value Blows/Ft.	Descriptive Term (Consistency)	Unconfined Compressive Strength Qu, (psf)	Standard Penetration or N-Value Blows/Ft.
	Very Loose	0 - 3	Very Soft	less than 500	0 - 1
	Loose	4 - 9	Soft	500 to 1,000	2 - 4
	Medium Dense	10 - 29	Medium Stiff	1,000 to 2,000	4 - 8
	Dense	30 - 50	Stiff	2,000 to 4,000	8 - 15
Very Dense	> 50	Very Stiff	4,000 to 8,000	15 - 30	
		Hard	> 8,000	> 30	

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 15
With	15 - 29
Modifier	> 30

GRAIN SIZE TERMINOLOGY

Major Component of Sample	Particle Size
Boulders	Over 12 in. (300 mm)
Cobbles	12 in. to 3 in. (300mm to 75mm)
Gravel	3 in. to #4 sieve (75mm to 4.75 mm)
Sand	#4 to #200 sieve (4.75mm to 0.075mm)
Silt or Clay	Passing #200 sieve (0.075mm)

RELATIVE PROPORTIONS OF FINES

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 5
With	5 - 12
Modifier	> 12

PLASTICITY DESCRIPTION

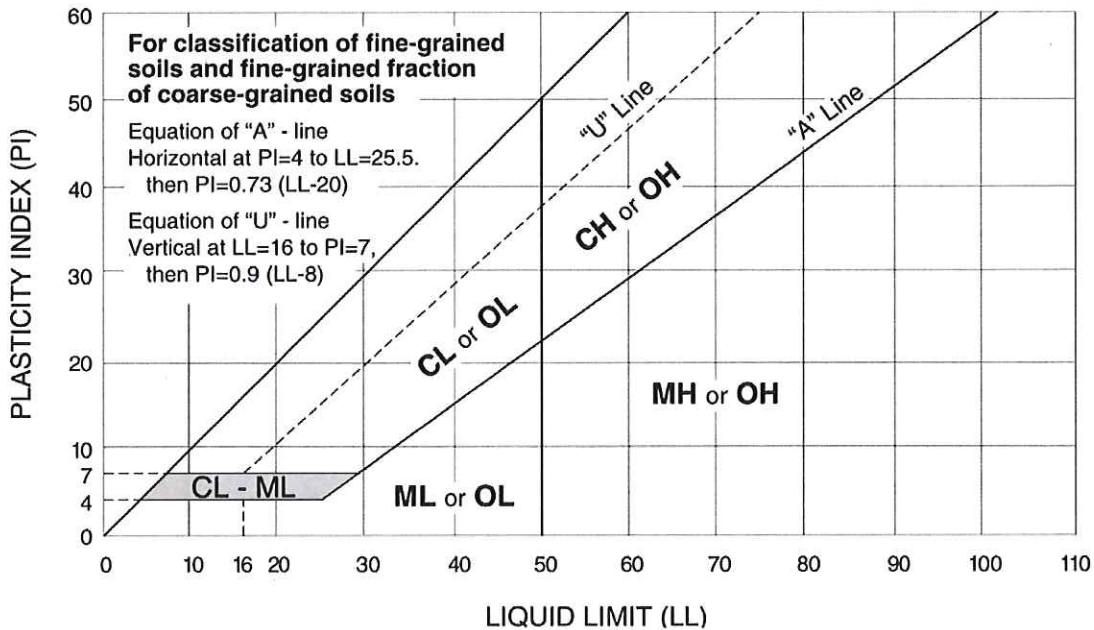
Term	Plasticity Index
Non-plastic	0
Low	1 - 10
Medium	11 - 30
High	> 30

UNIFIED SOIL CLASSIFICATION SYSTEM

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests ^A				Soil Classification		
				Group Symbol	Group Name ^B	
Coarse Grained Soils: More than 50% retained on No. 200 sieve	Gravels: More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels: Less than 5% fines ^C	$Cu \geq 4$ and $1 \leq Cc \leq 3$ ^E	GW	Well-graded gravel ^F	
			$Cu < 4$ and/or $1 > Cc > 3$ ^E	GP	Poorly graded gravel ^F	
		Gravels with Fines: More than 12% fines ^C	Fines classify as ML or MH	GM	Silty gravel ^{F,G,H}	
			Fines classify as CL or CH	GC	Clayey gravel ^{F,G,H}	
	Sands: 50% or more of coarse fraction passes No. 4 sieve	Clean Sands: Less than 5% fines ^D	$Cu \geq 6$ and $1 \leq Cc \leq 3$ ^E	SW	Well-graded sand ^I	
			$Cu < 6$ and/or $1 > Cc > 3$ ^E	SP	Poorly graded sand ^I	
		Sands with Fines: More than 12% fines ^D	Fines classify as ML or MH	SM	Silty sand ^{G,H,I}	
			Fines classify as CL or CH	SC	Clayey sand ^{G,H,I}	
Fine-Grained Soils: 50% or more passes the No. 200 sieve	Silts and Clays: Liquid limit less than 50	Inorganic:	$PI > 7$ and plots on or above "A" line ^J	CL	Lean clay ^{K,L,M}	
			$PI < 4$ or plots below "A" line ^J	ML	Silt ^{K,L,M}	
		Organic:	Liquid limit - oven dried	< 0.75	OL	Organic clay ^{K,L,M,N}
			Liquid limit - not dried		OH	Organic silt ^{K,L,M,O}
		Silts and Clays: Liquid limit 50 or more	Inorganic:	PI plots on or above "A" line	CH	Fat clay ^{K,L,M}
				PI plots below "A" line	MH	Elastic Silt ^{K,L,M}
	Organic:		Liquid limit - oven dried	< 0.75	OH	Organic clay ^{K,L,M,P}
			Liquid limit - not dried		OH	Organic silt ^{K,L,M,Q}
	Highly organic soils:	Primarily organic matter, dark in color, and organic odor			PT	Peat

- ^A Based on the material passing the 3-inch (75-mm) sieve
- ^B If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
- ^C Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.
- ^D Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay
- ^E $Cu = D_{60}/D_{10}$ $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$
- ^F If soil contains $\geq 15\%$ sand, add "with sand" to group name.
- ^G If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

- ^H If fines are organic, add "with organic fines" to group name.
- ^I If soil contains $\geq 15\%$ gravel, add "with gravel" to group name.
- ^J If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.
- ^K If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.
- ^L If soil contains $\geq 30\%$ plus No. 200 predominantly sand, add "sandy" to group name.
- ^M If soil contains $\geq 30\%$ plus No. 200, predominantly gravel, add "gravelly" to group name.
- ^N $PI \geq 4$ and plots on or above "A" line.
- ^O $PI < 4$ or plots below "A" line.
- ^P PI plots on or above "A" line.
- ^Q PI plots below "A" line.



DESCRIPTION OF ROCK PROPERTIES

WEATHERING

Term	Description
Unweathered	No visible sign of rock material weathering, perhaps slight discoloration on major discontinuity surfaces.
Slightly weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering and may be somewhat weaker externally than in its fresh condition.
Moderately weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a continuous framework or as corestones.
Highly weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.
Completely weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.
Residual soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

STRENGTH OR HARDNESS

Description	Field Identification	Uniaxial Compressive Strength, PSI (MPa)
Extremely weak	Indented by thumbnail	40-150 (0.3-1)
Very weak	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	150-700 (1-5)
Weak rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	700-4,000 (5-30)
Medium strong	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer	4,000-7,000 (30-50)
Strong rock	Specimen requires more than one blow of geological hammer to fracture it	7,000-15,000 (50-100)
Very strong	Specimen requires many blows of geological hammer to fracture it	15,000-36,000 (100-250)
Extremely strong	Specimen can only be chipped with geological hammer	>36,000 (>250)

DISCONTINUITY DESCRIPTION

Fracture Spacing (Joints, Faults, Other Fractures)		Bedding Spacing (May Include Foliation or Banding)	
Description	Spacing	Description	Spacing
Extremely close	< ¾ in (<19 mm)	Laminated	< ½ in (<12 mm)
Very close	¾ in – 2-1/2 in (19 - 60 mm)	Very thin	½ in – 2 in (12 – 50 mm)
Close	2-1/2 in – 8 in (60 – 200 mm)	Thin	2 in – 1 ft (50 – 300 mm)
Moderate	8 in – 2 ft (200 – 600 mm)	Medium	1 ft – 3 ft (300 – 900 mm)
Wide	2 ft – 6 ft (600 mm – 2.0 m)	Thick	3 ft – 10 ft (900 mm – 3 m)
Very Wide	6 ft – 20 ft (2.0 – 6 m)	Massive	> 10 ft (3 m)

Discontinuity Orientation (Angle): Measure the angle of discontinuity relative to a plane perpendicular to the longitudinal axis of the core. (For most cases, the core axis is vertical; therefore, the plane perpendicular to the core axis is horizontal.) For example, a horizontal bedding plane would have a 0 degree angle.

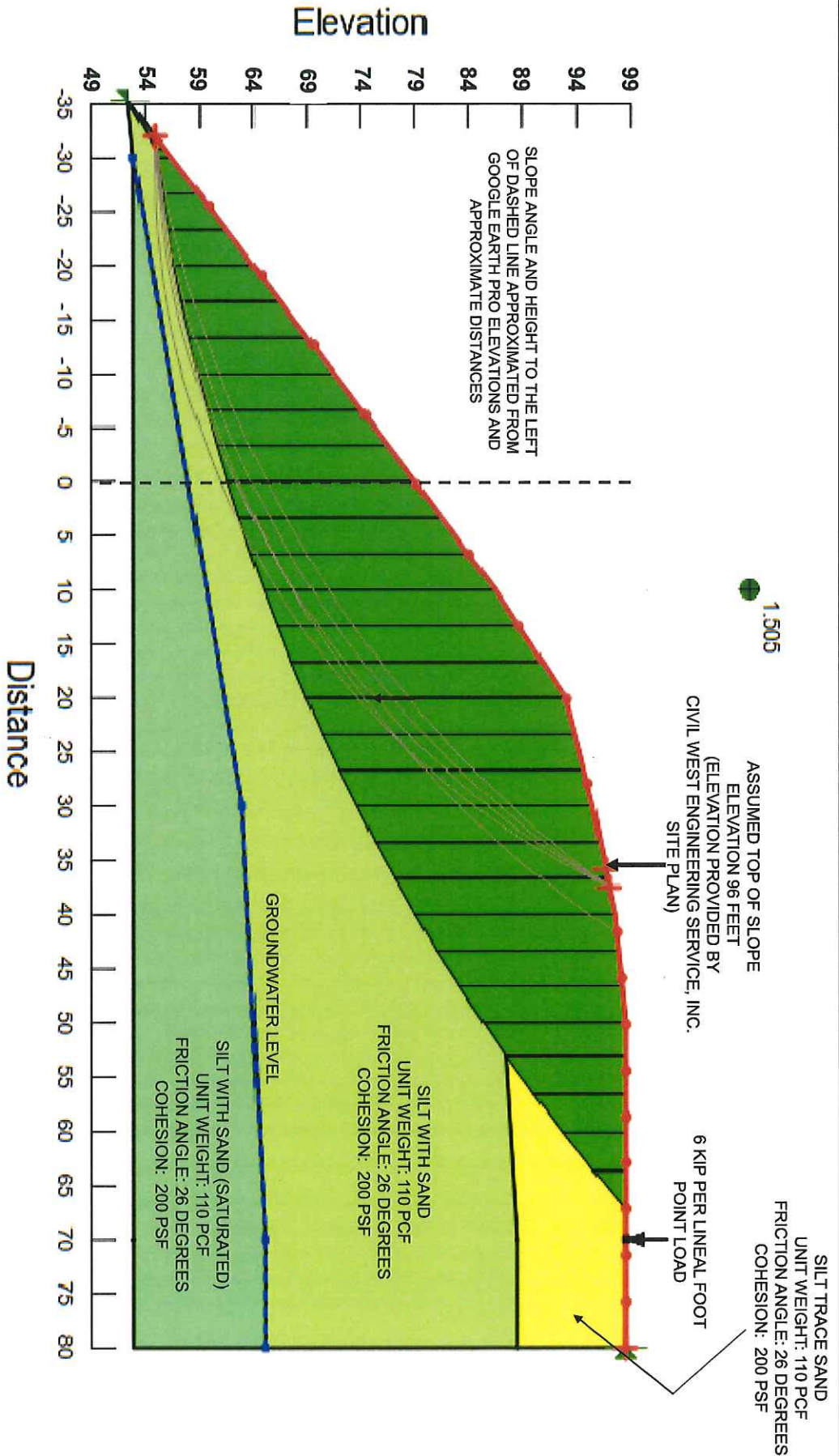
ROCK QUALITY DESIGNATION (RQD*)

Description	RQD Value (%)
Very Poor	0 - 25
Poor	25 - 50
Fair	50 - 75
Good	75 - 90
Excellent	90 - 100

*The combined length of all sound and intact core segments equal to or greater than 4 inches in length, expressed as a percentage of the total core run length.

Reference: U.S. Department of Transportation, Federal Highway Administration, Publication No FHWA-NHI-10-034, December 2009
Technical Manual for Design and Construction of Road Tunnels – Civil Elements

APPENDIX D
SLOPE STABILITY ANALYSES RESULTS



Project Manager: BVVP	Project No.: 821680934	Terracon 4103 SE International Way, Suite 300 Portland, Oregon 97222	SLOPE STABILITY ANALYSIS PARKLAND ASSISTED LIVING 13761 NE CLIMILUS AVENUE MCMINNVILLE, OREGON	Exhibit:
Drawn by: BVVP	Scale: AS SHOWN			D-1
Checked by: KTH	File Name: EXH D-1			
Approved by: KTH	Date: JULINE 2016			

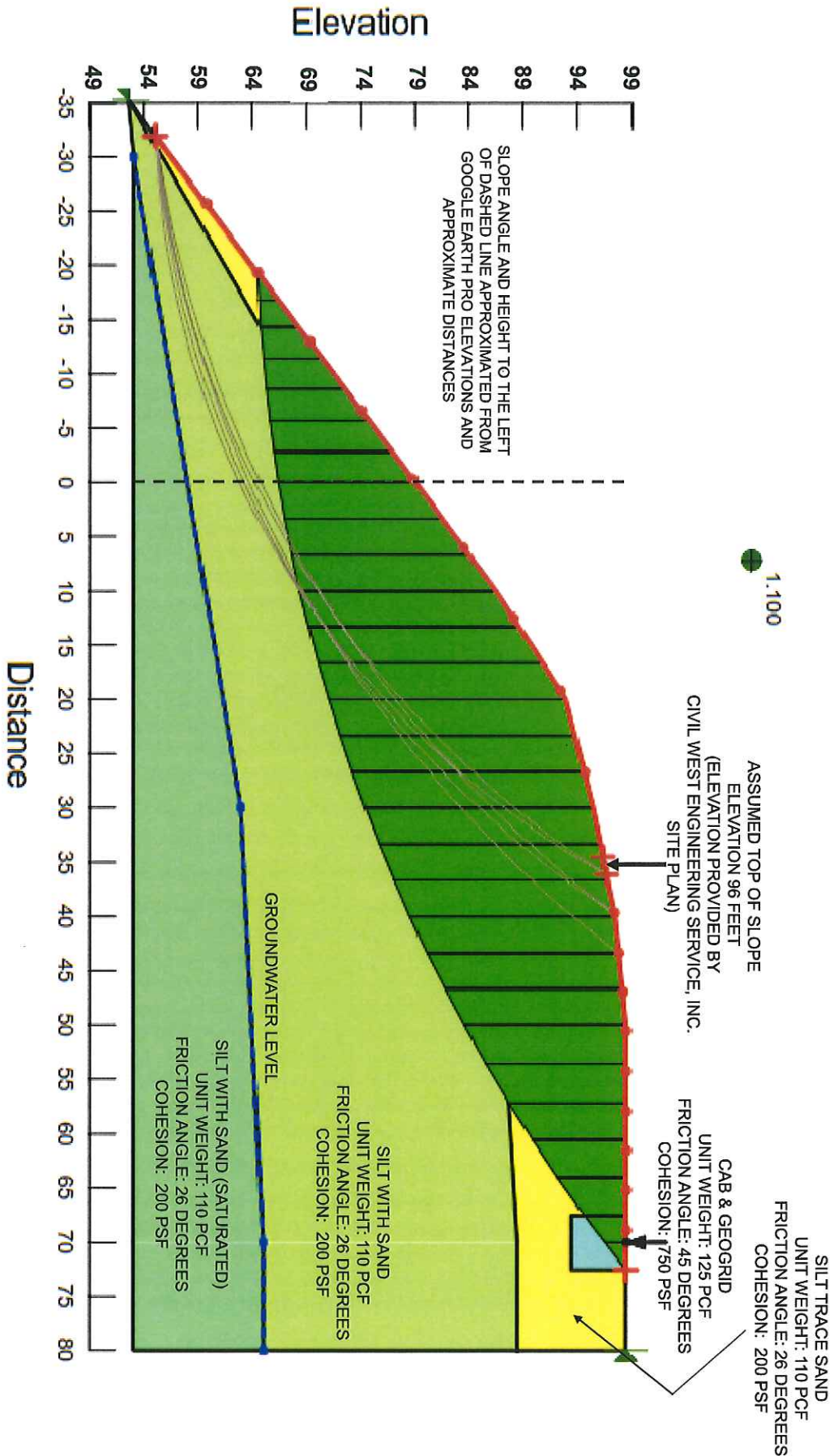


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES.

Project Manager:	BWP	Project No.:	82168934
Drawn by:	BWP	Scale:	AS SHOWN
Checked by:	KTH	File Name:	EXH D-1
Approved by:	KTH	Date:	JUNE 2016

Terracon
4103 SE International Way, Suite 300
Portland, Oregon 97222

SEISMIC SLOPE STABILITY ANALYSIS

PARKLAND ASSISTED LIVING
13361 NE CUMULUS AVENUE
MCMINNVILLE, OREGON

Exhibit
D-2



PROJECT STATISTICS

PROJECT SCOPE	APARTMENTS, 31 UNITS
PARCEL	40171C0408D
PROJECT LOCATION	3123 NE CUMULUS AVE MCMINNVILLE, OR 97128
SITE ZONING	CITY OF MCMINNVILLE
ZONING JURISDICTION	MEMORY CARE FACILITY
SITE USE	6'-0"
SET BACK - WEST:	20'-0"
SET BACK - NORTH:	15'-0"
SET BACK - SOUTH:	6'-0"
SET BACK - EAST:	60'-0"
BUILDING HEIGHT LIMIT	<35'-0"
MAX PROPOSED/EXISTING HEIGHT	

CODE ANALYSIS

CODE:	OSSC 2014, 2009 ICC A117.1-2009
OCCUPANCY:	I-2 INSTITUTIONAL
CONSTRUCTION TYPE:	V-A WOOD CONSTRUCTION

UNIT DENSITY	1,500 S.F. UNIT DENSITY PER ACRE
ASSISTED LIVING UNITS	1,500 S.F. PER UNIT
TOTAL UNITS	1,500 X 5.09 ACRES (221,853 S.F.) = 147 UNITS

PROPOSED DENSITY	
EXISTING ASSISTED LIVING UNITS	50 UNITS (50 BEDS)
PROPOSED MEMORY CARE UNITS	18 UNITS (31 BEDS)
TOTAL UNITS	68 UNITS (81 BEDS)

SITE AREA

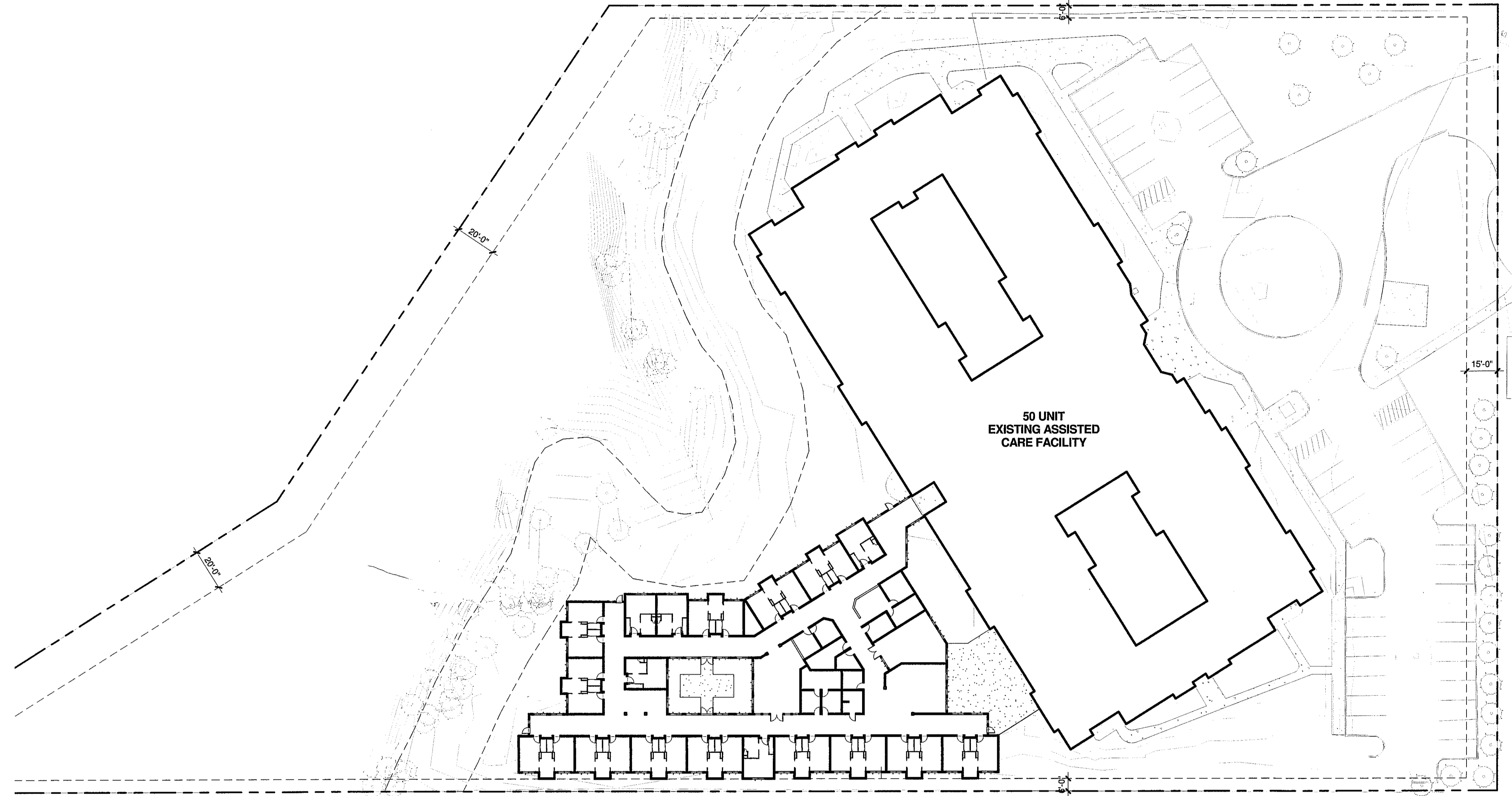
PROPERTY	AREA	AREA (SF)
TOTALS	5.09 acres	221,853.2 SF

BUILDING AREA

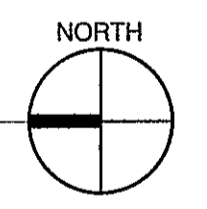
BUILDING	TOTAL AREA
FINISH FLOOR - L1	
PROPOSED ADDITION	18,246.92 SF
EXISTING FACILITY	36,961.52 SF
TOTAL	55,208.44 SF

PARKING PROVIDED

DESCRIPTION	TYPE	SPACE PROVIDED
VAN ACCESSIBLE	V	1
STANDAR	S	41
HANDICAP	H	3
TOTAL		45



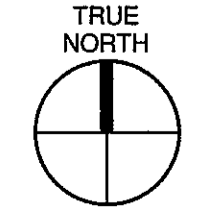
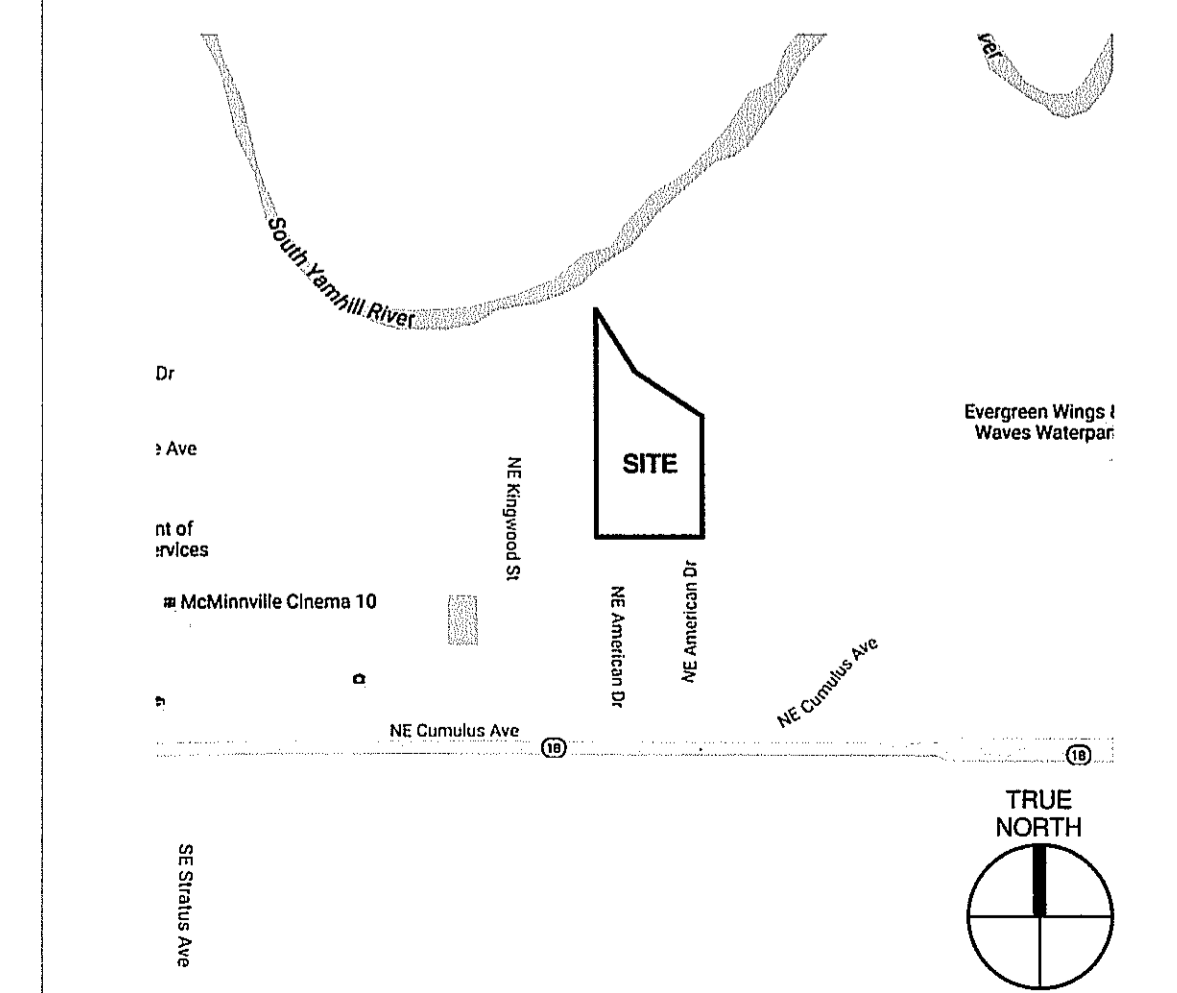
50 UNIT
EXISTING ASSISTED
CARE FACILITY



OVERALL SITE PLAN

1/32" = 1'-0"

VICINITY MAP

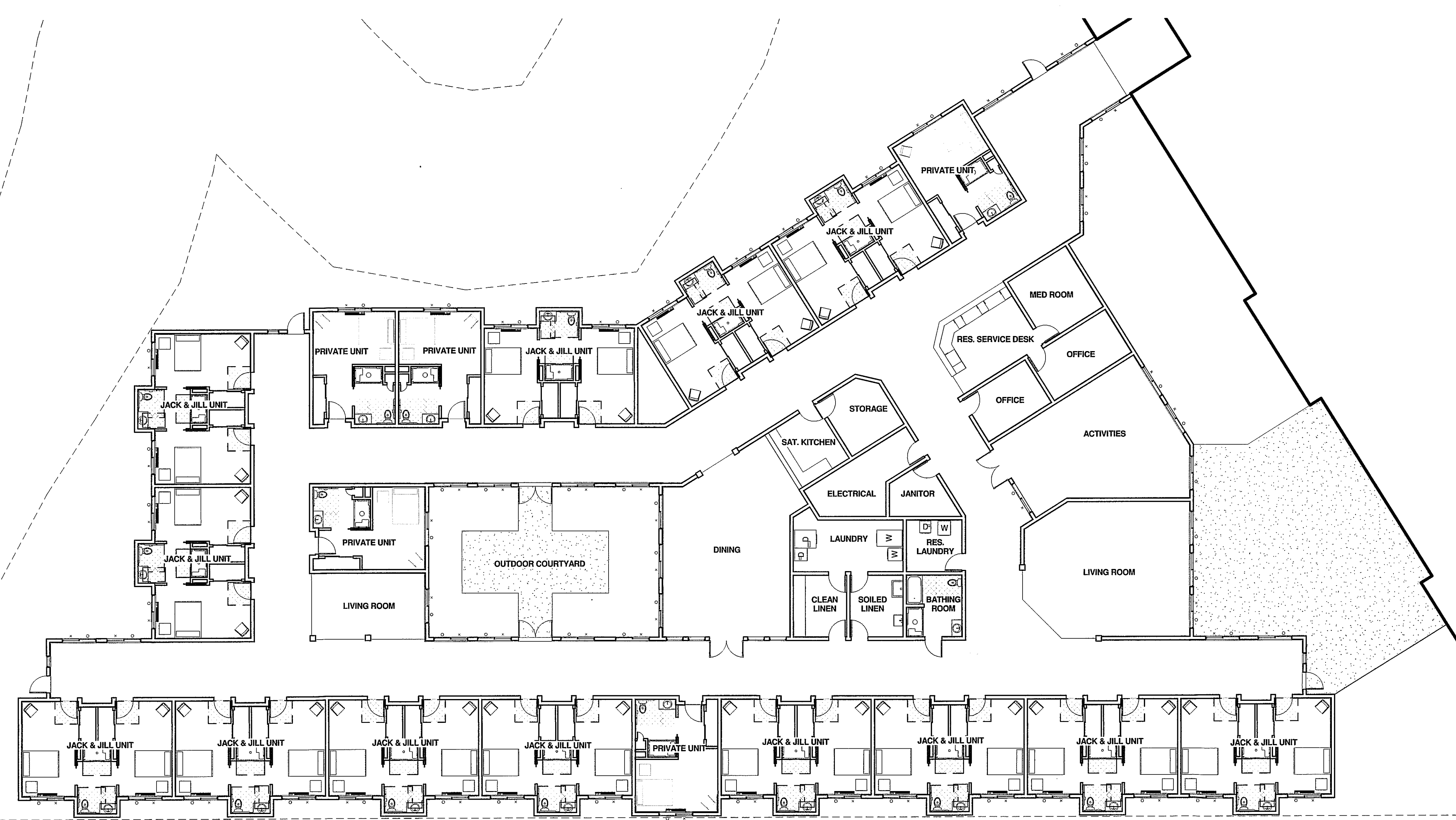


McMINNVILLE PARKLAND ASSISTED CARE
3123 NE CUMULUS AVE,
MCMINNVILLE, OREGON 97128
OVERALL SITE PLAN

PROJECT

DATE	DESC.	#

JOB NO.:	1631
ISSUE DATE:	10/31/2016
REVISED:	
DRAWN BY:	BM



NOTES

PROPOSED UNIT TYPES

ROOM NAME	UNIT COUNT	BED COUNT
JACK & JILL UNIT	13	26
PRIVATE UNIT	5	5
TOTALS	18	31

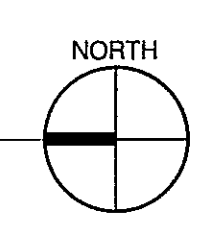
RDA

ROSS DECKMAN ARCHITECT
 207 FOURTH AVENUE SOUTHEAST,
 PUYALLUP, WASHINGTON 98372
 PHONE: 253.840.8405
 FAX: 253.840.9503
REGISTERED ARCHITECT
 ROSS E. DECKMAN
 PUYALLUP, WASHINGTON
 2875
STATE OF OREGON

PROJECT
McMINNVILLE PARKLAND ASSISTED CARE
 3123 NE CUMULUS AVE,
 MC MINNVILLE, OREGON 97128

FLOOR PLAN - LEVEL 1

FLOOR PLAN - LEVEL 1
 3/32" = 1'-0"



#	DATE	DESC.

JOB NO.: 1631
 ISSUE DATE: 10/31/2016
 REVISIONS:
 DRAWN BY: BM
 SHEET