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February 20, 2019 6173 GEOTECHNICAL AND PAVEMENT RPT

DRAFT

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Attention: David Brokaw, PE

SUBJECT: Geotechnical and Pavement Investigation
Garden Corner Curves Transportation Improvement Project
Tualatin, Oregon

At your request, GRI conducted a geotechnical and pavement investigation for the above-referenced project. The general location of the site is shown on the Vicinity Map, Figure 1. The purpose of the investigation was to evaluate subsurface conditions at the site and develop geotechnical and pavement recommendations for use in the design and construction of the proposed improvements. The investigation consisted of subsurface explorations, in-situ infiltration testing, falling weight deflectometer (FWD) deflection testing, laboratory testing, and engineering studies. This report describes the work accomplished and provides our conclusions and recommendations.

PROJECT DESCRIPTION

The project consists of the realignment of the Garden Corner Curves in Tualatin, Oregon. The Garden Corner Curves consists of portions of SW 105th Avenue, SW Blake Street, and SW 108th Avenue that provide one of the few continuous north-south routes within the City of Tualatin (City). The Site Plan, Figure 2, shows the project alignment. The project will realign the curves to accommodate a shared 12-ft-wide path to improve pedestrian and bicycle safety and reduce vehicle speeds along the road. An existing culvert, which carries Hedges Creek beneath SW 105th Avenue north of SW Blake Street, will be replaced with a pre-cast concrete box culvert. The new box culvert will be 15 ft wide and 9 ft tall with 10-ft wing walls. New asphalt concrete (AC) pavement, stormwater detention features, and retaining walls will also be constructed. The height of retaining walls are estimated to range from about 4 to 8 ft.

All elevations noted in this report are based on the National American Vertical Datum of 1988 (NAVD 88).

SITE DESCRIPTION

Topography and Surface Conditions

The Garden Corner Curves include the intersection of SW 108th Avenue and SW Blake Street, SW Blake Street, and the intersection of SW Blake Street and SW 105th Avenue in Tualatin, Oregon. Hedges Creek crosses beneath SE 105th Avenue north of SW Blake Street in a 42-in.-diameter, corrugated metal pipe culvert. The existing road has two lanes and is paved with AC. Mature trees line the majority of the alignment.

Our observations at the site and review of available topographic maps indicate the ground surface slopes down from the north end of the project towards Hedges Creek from about elevation 225 ft near SW Moratoc

Drive to about elevation 201 ft at the creek crossing. The east side of the road grade slopes down relatively steeply towards Hedges Creek in the vicinity of the intersection of SW Blake Street and SW 105th Avenue. The ground surface elevation increases south of the creek crossing to about elevation 257 ft at the southern end of the project area near SW Willow Street.

Geology

Fill is present in several locations along the alignment. Beneath the fill and/or pavement, the site is mantled by soils referred to as Willamette Silt. Willamette Silt caps the majority of the Willamette Valley from Portland to Eugene and is composed of fine-grained sand, silt, and clay. Stratification within the unit commonly consists of 4- to 6-in.-thick beds, although 3- to 4-ft-thick beds are present locally. In some areas, the silt is massive and bedding is indistinct or non-existent. Available geologic information indicates the silt and sand are underlain by weathered and unweathered basaltic flows of Columbia River Basalt (Ma et al. 2012). Review of available geologic literature indicates there are no known or mapped faults within 2.5 miles of the site.

SUBSURFACE CONDITIONS

General

Subsurface materials and conditions along the project alignment were investigated from January 3 through 17, 2019, with five machine-drilled borings, designated B-1 through B-5, and three hand-augered borings for infiltration testing, designated I-1 through I-3. The approximate locations of the borings are shown on Figure 2. The machine-drilled borings were advanced to depths of about 11.5 to 31.5 ft below the ground surface, and the hand-augered borings were advanced to depths of about 3 to 4 ft. Logs of the machine-drilled borings are provided on Figures 1A through 5A and logs of the hand-augered borings are provided on Figures 6A and 7A. Soil samples collected from the explorations were returned to our laboratory for further examination and physical testing. The field investigation and laboratory testing programs completed for this investigation are described in Appendix A. The terms used to describe the materials encountered in the borings are defined in Table 1A in Appendix A and on the attached legend. Table 2A in Appendix A provides a summary of the results of the laboratory testing completed.

Soils

The borings indicate the project site beneath the pavement is typically mantled with silt, sand, and clay of the Willamette Silt Formation. Fill was encountered beneath the pavement in several borings. For the purpose of discussion, the materials disclosed by the borings have been grouped into the following categories based on their physical characteristics and engineering properties:

1. **Asphalt Concrete PAVEMENT and Crushed-Rock BASE COURSE**
2. **FILL**
3. **SILT, SAND, and CLAY (Willamette Silt Formation)**

The following paragraphs provide a detailed description of these materials and a discussion of the groundwater conditions at the site.

1. Asphalt Concrete PAVEMENT and Crushed-Rock BASE COURSE. Within the project limits, the roadway is surfaced with dense-graded AC pavement. Borings B-1 through B-5 encountered about 3.75 to 8.5 in. of AC pavement at the ground surface underlain by about 4.75 to 11.5 in. of crushed-rock base (CRB) course.

Based on our observations, we estimate the relative density of the crushed rock ranges from medium dense to dense. Pavement cores ranged from good to poor condition. Full-depth cracking was observed in cores B-1 and B-3. Logs of the pavement cores are provided on Figures 9A through 13A. Photographs of the pavement cores are shown on Figures 14A through 18A. We characterized the pavement cores in general accordance with the information provided in the current Oregon Department of Transportation (ODOT) Pavement Design Guide.

2. FILL. Silt identified as possible fill was encountered beneath the pavement in borings B-1 and B-4. The material extends to a depth of about 5 ft below the ground surface. The silt contains varying percentages of sand and clay, ranging from a trace of sand to sandy and trace to some clay, and wood debris. Standard Penetration Test (SPT) N-values ranging from 2 to 6 blows/ft indicate the relative consistency of the silt ranges from soft to medium stiff. The natural moisture content of the silt ranges from 20 to 23%.

A 1-ft thickness of silty gravel fill was encountered at the ground surface in boring I-2. The gravel fill is angular and based on our observations, we estimate the relative density of the fill is loose to medium dense.

3. SILT, SAND, and CLAY (Willamette Silt Formation). Interbedded alluvial silt, sand, and clay of the Willamette Silt Formation were encountered beneath the pavement and fill in all the borings. The silt portions contain varying percentages of clay and fine- to coarse-grained sand ranging from a trace of clay to clayey and trace sand to sandy. Wood debris is present in the silt in boring B-4 between depths of about 7.5 and 12.5 ft. The silt is typically brown to brown mottled rust. SPT N-values ranging from 1 to 13 blows/ft indicate the relative consistency of the silt ranges from very soft to stiff and is typically medium stiff. The natural moisture content of the silt ranges from 21 to 33%. A moisture content of 73% at a depth of 7.5 ft in boring B-4 is likely associated with the presence of wood debris at that depth. Borings B-1, B-3, B-4, I-1, and I-3 were terminated in silt at depths ranging from about 3 to 11.5 ft.

Clay layers were encountered in boring B-5 beneath the pavement at a depth of about 1.2 to 7.5 ft and below a layer of silty sand from about 26 to 31.5 ft, and in boring I-2 beneath the fill at a depth of 1 to 3 ft. The clay contains varying percentages of silt and fine-grained sand, ranging from a trace of silt to silty and a trace of sand to sandy. Organics are present in the near-surface portion of the clay above a depth of about 7.5 ft. The clay is typically dark gray to brown. SPT N-values ranging from 0 to 10 blows/ft and Torvane shear-strength values of 0.35 to 0.80 tsf indicate the relative consistency of the clay ranges from very soft to stiff. The natural moisture content of the clay ranges from 31 to 37%. Borings B-5 and I-2 were terminated in clay at depths of 31.5 and 3 ft, respectively.

Silty sand was encountered between the depths of 10 and 11.5 ft in boring B-2, between 3 and 10 ft in boring B-3, and between 12.5 and 25 ft in boring B-5. The sand is fine grained and gray to brown. SPT N-values ranging from 3 to 9 blows/ft indicate the relative density of the sand ranges from very loose to loose. The natural moisture content of the sand ranges from 25 to 40%. Boring B-2 was terminated in sand at a depth of about 11.5 ft.

Falling Weight Deflectometer (FWD)

GRI performed non-destructive pavement deflection testing using our KUAB 2m Model 150 FWD that is compliant with provisions in ASTM International (ASTM) D4694. Testing was completed on January 18, 2019, and was conducted at approximately 100-ft intervals in the outer wheel path of the travel lanes in both directions. The purpose of FWD testing was to estimate the in-situ moduli (stiffness) of the pavement, CRB, and subgrade layers through use of a backcalculation analysis, which is discussed later in this report. Pavement surface deflections, expressed in mil units (1 mil = 0.001 in), were normalized to a 9,000-lb (9-kip) FWD load and temperature-adjusted to an AC mid-depth temperature of 68 °F. The deflection results are shown in Table 1B and plotted by test location on Figure 1B in Appendix B.

Groundwater

Borings B-1 through B-4 were completed using open-hole, hollow-stem auger drilling techniques, which allows for direct measurement of groundwater. However, groundwater was not observed. Boring B-5 was advanced using mud-rotary drilling methods, which do not permit the observation of groundwater conditions during drilling. Based on our experience in the area and review of Oregon Department of Water Resources well logs in the area surrounding the site, we anticipate the local groundwater level along the majority of the alignment typically occurs at a depth of 20 to 30 ft below the ground surface during the normally dry summer and fall months. Groundwater levels at the site will vary with precipitation and may approach the ground surface during the wet winter and spring months or during periods of heavy or prolonged precipitation. We anticipate groundwater levels in the vicinity of the creek crossing will be at or near the water level elevation in the creek and will fluctuate in response to water levels in the creek.

CONCLUSIONS AND RECOMMENDATIONS

General

The borings indicated the project alignment is underlain by silt, sand, and clay. The groundwater along the majority of the alignment typically occurs below a depth of about 20 ft and may approach the ground surface during the wet winter and spring months or during periods of heavy or prolonged precipitation. The groundwater level near the creek crossing will remain at or near the water level in the creek and will fluctuate in response to creek levels.

The primary geotechnical considerations associated with construction of the proposed improvements include the presence of fine-grained soils at the ground surface that are extremely sensitive to moisture content and the potential for shallow, perched groundwater conditions. In addition, a zone of silty sand was encountered below the groundwater at the culvert boring. Therefore, the potential for running sand will be an important consideration during excavation and construction of the new culvert.

The following sections of this report provide our conclusions and recommendations for use in the design and construction of the project.

Infiltration Testing

Falling head infiltration testing was completed at the site on January 3 and 9, 2019, in substantial conformance with the City of Portland 2016 Stormwater Management Manual (SMM) using the encased falling head method outlined in Section 2.3.6 of the manual. The infiltration tests were completed in hand-augered borings at a depth of about 3 ft at the locations provided by Wallis Engineering, PLLC. The test locations, designated I-1 through I-3, are shown on the Site Plan, Figure 2. The average unfactored, field-measured infiltration rates are tabulated below. Additional details of the infiltration testing methods are provided in Appendix A. Logs of the hand-augered borings are provided on Figures 6A and 7A and a summary of the laboratory test results for the hand-augered borings is provided in Table 2A in Appendix A.

Table 1: INFILTRATION TEST RESULTS

Boring	Depth of Infiltration Test, ft	Average Infiltration Rate, in./hour	Soil Classification	Fines Content (% Passing No. 200 Sieve)
I-1	3.1	0.2	SILT, trace to some fine-grained sand, trace clay	82%
I-2	3.5	0.0	Silty CLAY, some fine-grained sand	81%
I-3	3.5	0.0	Sandy SILT, fine-grained sand	64%

Water-level drop inside the auger was observed to be less than about 0.2 in. per hour during the infiltration testing, indicating a very low infiltration rate. We anticipate the infiltration rate within the silt soil mantling the other areas of the site will also be very low. In addition, groundwater could approach the ground surface during the periods of prolonged precipitation common during the typically wet winter months of the year. In our opinion, the observed low infiltration rates and potential for high groundwater conditions are not favorable to on-site infiltration of stormwater at this site.

CULVERT

General

As previously discussed, the existing 42-in.-diameter, corrugated metal pipe culvert will be replaced with a pre-cast reinforced concrete box culvert. In our opinion, the important geotechnical considerations associated with the culvert replacement includes maintaining positive control of groundwater during construction, providing stable excavation side slopes or shoring to support excavation sidewalls, adequately bedding the new culvert, properly backfilling the culvert walls to minimize post-construction settlement at the pavement surface, and providing long-term erosion protection at the inlet and outlet of the structure.

Groundwater Control. Positive control of water will be necessary to maintain stable excavation sides and bottom. Groundwater levels are expected to be consistent with creek levels, and problems associated with controlling water can be reduced by scheduling construction of the replacement culvert during the typically driest months of the year in late summer and early fall. The appropriate method of control will depend on actual water levels at the time of construction. The use of temporary cofferdams such as sand bags, supersacks, earthen berms or sheetpiles may be necessary to help control the flow of water. Regardless of the method used by the contractor, any proposed dewatering system should be capable of maintaining groundwater levels a minimum of 2 ft below the base of the excavation to maintain a stable excavation subgrade. GRI should review the contractor's proposed dewatering system prior to mobilization to the site.

Excavation and Shoring. We anticipate the excavations necessary to remove the old culvert and construct the new culvert can be made by sloping or shoring the excavation sidewalls. The maximum depth of the excavation necessary to construct the new culvert appears to be about 15 ft. The method of excavation and the design of the excavation support is typically the responsibility of the contractor and should conform to applicable local, state, and federal regulations. The information provided below is for the use of our client and should not be interpreted to mean we are assuming responsibility for the contractor's actions or site safety.

The inclination of temporary excavation slopes will depend on the groundwater conditions encountered at the time of construction and the soil type. Our borings indicate loose silty sand below a depth of about 7.5

ft in the vicinity of the culvert that should be classified as Type C soil according to the most recent Occupational Safety and Health Administration (OSHA) regulations. OSHA requirements for excavations in soils classified as Type C limit excavations sloped at $1\frac{1}{2}$ H:1V (Horizontal to Vertical) to a depth of 20 ft unless the slope is designed by an engineer. Heavy surcharge loads should not be allowed within about 15 ft of the top of the cut without further analysis of slope stability. The flatter slope angle or the installation of sheetpiles to retain the slope may be necessary if significant seepage or running soil conditions develop on the side slopes of the excavation.

Other measures that should be considered to reduce the risk of localized failures of temporary slopes include: (1) use of a geotextile fabric or plastic sheeting to protect the exposed cut slopes from surface erosion due to precipitation; (2) providing positive drainage away from the top and bottom of the cut slopes; (3) construction and backfill of the culvert as soon as practical after completing the excavation; and (4) periodically monitoring of the area around the top of the excavation for evidence of ground cracking.

The lateral earth pressure criteria shown on Figures 3 and 4 can be used for the design of shoring systems. The guidelines provided on Figure 3 for cantilevered shoring assume the shoring can be allowed to yield somewhat into the excavation during construction, while the guidelines provided on Figure 4 for braced shoring assume yielding will be minimized.

Excavation Bottom Stabilization and Culvert Bedding. In our opinion, due to the probability of water seepage within the bottom of the excavations, we recommend overexcavation of the subgrade to install granular structural fill. It has been our experience that up to 2 ft of overexcavation will likely be needed; however, the actual depth of overexcavation would be best established based on observations at the time of construction. Backfill for the overexcavation can consist of granular material, such as sand, sandy gravel, or fragmental rock, with a maximum size of up to about 3 in. and conforming to the requirements in Section 00330.14 of the 2018 ODOT Oregon Standard Specifications for Construction (ODOT SSC). The subgrade should be blanketed with a non-woven geotextile fabric prior to placement of the backfill. Trench-bottom stabilization material should be placed in a single lift and compacted with vibratory equipment until well-keyed. We anticipate pumping from temporary sumps installed within or below the trench-bottom stabilization material can be used to help control groundwater.

We recommend a minimum 6-in. thickness of ¾-in.-minus granular aggregate be provided over the excavation bottom stabilization material to serve as a leveling course and "choke" the surface of the coarser rock. This material is also suitable for use as bedding for the culvert. It may be prudent to bed precast structures on fluid grout, controlled-density fill, or a similar type of material at the time of placement to reduce the potential for any voids or the development of a preferential path of seepage beneath the structure. In addition, to limit the potential for piping beneath the structure, the floor of the structure should be provided with cut-off walls at the inlet and outlet. The cut-off walls should extend a minimum of 3 ft below the bottom of the culvert. We anticipate the cut-off walls will be constructed by excavating a trench and placing the concrete directly against the sidewalls. Depending somewhat on the materials and groundwater encountered in the excavation, it may be necessary to place the concrete for the cut-off walls using tremie methods.

Culvert Backfill. Backfill along the culvert should consist of sand or well-graded crushed rock with a maximum size of about 3 in. and meeting the requirements of Section 00330.14 of the ODOT SSC

However, the backfill within 2 ft of the walls and roof of the culvert should be limited to a maximum size of 1 in. We recommend the backfill material be placed in lifts and compacted until well-keyed using a medium-weight, smooth, steel-wheeled vibratory roller or a hoe-mounted vibratory plate compactor. Lift thicknesses should be proportioned to be appropriate with the type of compaction equipment used. We recommend limiting lift thicknesses prior to compaction to 12 in. for vibratory, smooth-drum rollers and 24 in. for trackhoe-mounted vibratory plates. Backfill should be placed in lifts and compacted with vibratory equipment to at least 95% of the maximum density as determined by ASTM D698. Care should be taken to raise the level of the backfill equally on both sides of the box culvert during the backfilling. We recommend finished roadway embankment slopes at the inlets and outlets be no steeper than 2H:1V.

Settlement. Post-construction settlement of the ground surface can be reduced by backfilling the excavation with clean, granular structural fill as previously recommended. Inadequate removal of disturbed, soft, or loosened materials prior to installation of the stabilization/bedding material beneath the culvert may result in post-construction settlement of the culvert and overlying pavement surface. Subgrade disturbance could be caused by improper excavation, insufficient groundwater control, or trafficking of an exposed and unprotected subgrade.

Erosion Protection and Piping Potential

We recommend erosion protection be provided at the inlet and outlet of the new structure. The slopes in these areas should be protected with a blanket of rip rap material placed directly over stone embankment material. The class of rip rap should be sized for estimated maximum stream-flow velocities. A non-woven geotextile fabric should be placed over the subgrade prior to placement of the stone embankment material. To provide protection against scour, we recommend rip rap material also be provided at the channel bottom at the inlet and outlet cut-off walls. The thickness of the rip rap should be consistent with the rip rap class and extend a minimum horizontal distance of 10 ft beyond the culvert inlet and outlet. Other areas of newly exposed soil should be mulched and seeded to promote the growth of new vegetation.

Earth Pressures

We anticipate groundwater levels will closely parallel creek levels and the walls of the new box culverts will be relatively rigid and non-yielding. Therefore, we recommend the wall of the box culvert be designed for the at-rest earth pressure case using a hydrostatic pressure based on an equivalent fluid unit weight of 55 pcf. Relatively rigid and non-yielding wing walls retaining a back slope of up to 2H:1V can be designed using a hydrostatic pressure based on an equivalent fluid unit weight of 85 pcf with fully drained backfill. Additional lateral earth pressure due to surcharge loads, such as traffic or construction equipment, may be estimated based on a rectangular distribution of 200 psf. The pressure on the roof of the box culvert due to the weight of the fill and pavement section over the roof may be estimated assuming a bulk unit weight of about 130 pcf. Additional live load pressures due to traffic should also be included in the structural design of the top of the box culvert.

Seismic Considerations

The proposed structure is considered a buried structure by the American Association of State Highway and Transportation Officials (AASHTO); therefore, in accordance with Sections 3.10.1 and 12.6.1 of the 2017 AASHTO Load Resistance Factor Design (LRFD) Bridge Design Specifications (BDS), we anticipate earthquake loads need only be considered if the structure crosses an active fault. The geology evaluation did not identify any faults that are active or otherwise in the immediate project vicinity. Therefore, evaluation of seismic mitigation was not performed for the project.

ROAD WIDENING AND REALIGNMENT

General

We anticipate relatively minor amounts of fill will be required for the widening and realignment. We anticipate fills will be constructed of imported granular material or material generated from on-site cuts. We anticipate the fill slopes will be graded at 2H:1V or flatter.

Site Preparation and Grading

The ground surface within all areas to receive new structural fill or pavement should be stripped of vegetation and surface organics. Stripping in grassy areas should generally be accomplished to a nominal depth of about 4 in.; however, deeper grubbing will be required locally to remove heavy brush or tree roots. In our opinion, strippings should be removed from the site or stockpiled on site for use in landscaped areas. Site preparation and grading activities should be performed in a manner that minimizes disturbance to exposed, moisture-sensitive, fine-grained soil subgrades. Site preparation and grading should conform to Section 00330 of the ODOT SSC.

Due to the moisture-sensitive nature of the fine-grained soils that mantle the project area, we recommend the site preparation and earthwork be accomplished during the dry summer months, typically extending from late June to mid-October of any given year. During periods of wet weather and/or wet-ground conditions, the moisture content of the fine-grained soils is generally well above the optimum moisture content for effective compaction. As a result, the use of on-site materials within the structural fill will not be feasible during wet periods of the year. When working in wet conditions, the contractor must employ construction techniques that prevent or minimize disturbance and softening of the subgrade soils. For example, the use of scrapers for stripping and earthwork during periods of wet weather or in areas of wet ground will likely result in subgrade disturbance. During these conditions, it may be necessary to use a bulldozer or trackhoe excavator equipped with a smooth-edged bucket for stripping and general subgrade preparation activities.

To reduce disturbance and softening of the fine-grained subgrade soils during wet weather or in areas of wet-ground conditions, the movement of construction traffic should be limited to granular haul roads and work pads. In general, about 1½ to 2 ft of relatively clean, granular material is typically required to support concentrated construction traffic, such as dump trucks and concrete trucks, and protect the fine-grained subgrade. A 12-in.-thick granular work pad should be sufficient to support occasional truck traffic and light construction operations. A geotextile stabilization/separation fabric should be used between the granular work pad/haul road materials and the underlying fine-grained subgrade soils as a filter to prevent the movement of fines into the rock.

We recommend permanent cut and fill slopes of silty material be constructed no steeper than 2H:1V and temporary cut and fill slopes no steeper than 1H:1V. In areas where space constraints preclude construction of 2H:1V or flatter slopes or where it is desired to limit encroachment of fill slopes into the right-of-way, fill slopes constructed with fragmental rock and a finished slope of 1.5H:1V can be considered, depending on the height of the slope. Fills placed for the new roadway should be keyed into the existing fill slopes in accordance with the ODOT Standard Embankment Construction Detail (Detail 2100) and/or ODOT Sliver Fill Benching Detail (Detail 2101). In general, structural fills should extend a minimum horizontal distance of 5 ft beyond the limits of the new improvements, such as the edge of new pavement. If the final configuration includes retaining walls, the minimum setback should be measured as the horizontal distance from the toe of the wall footing to the exposed face of the slope. The following section provides additional recommendations for structural fill materials and placement methods.

Structural Fill

In our opinion, organic-free, fine- or coarse-grained soils are suitable for use in structural fills. However, as previously mentioned, fine-grained soils are sensitive to moisture content and can be placed and adequately compacted only during the dry summer months. For construction during the wet winter and spring months, fills should be constructed using imported, relatively clean, granular materials.

In general, approved organic-free, fine-grained soils used to construct structural fills should be placed in 9-in.-thick lifts (loose) and compacted using medium to large segmented-pad rollers to a density not less than 95% of the maximum dry density as determined by ASTM D698. Fill placed in landscaped areas should be compacted to a minimum of about 90% of the afore-mentioned standard. In our opinion, the moisture content of fine-grained soils at the time of compaction should be controlled to within 3% of optimum. Some aeration and drying of fine-grained soils should be anticipated to achieve the compaction criteria for fine-grained soils.

Granular material used to construct structural fills or work pads during wet weather can consist of sand, sandy gravel and cobbles, or fragmental rock with a maximum size of up to about 6 in. and not more than about 5% passing the No. 200 sieve (washed analysis). If necessary due to space constraints, embankment fill slopes consisting of hard, durable, angular, fragmental rock can be constructed at 1.5H:1V. Material conforming to Section 00330.16(a) of the ODOT SSC for stone embankments would be suitable for construction of slopes at 1.5H:1V. The first lift of granular fill material placed over the silt subgrade should be in the range of 12 to 18 in. thick (loose). Subsequent lifts should be placed 12 in. thick (loose). All lifts should be compacted with a medium-weight, smooth, steel-wheeled, vibratory roller until well compacted.

Placement of new fill will induce consolidation of the underlying silt and settlement at the ground surface. We estimate approximately 0.25 to 0.75 in. of primary settlement will occur for fills 2 to 5 ft in height, respectively, assuming granular fill material with a minimum width of 10 ft. Taller fills will induce greater settlement and should be evaluated on a case-by-case basis when final configurations are established. We estimate the majority of the settlement will occur within about 2 months of fill placement. We recommend fill soils be placed as early in the construction schedule as possible.

UTILITIES

The method of excavation and design of trench support are the responsibilities of the contractor and subject to applicable local, state, and federal safety regulations, including the current OSHA excavation and trench safety standards. The means, methods, and sequencing of construction operations and site safety are also the responsibilities of the contractor.

All backfill placed in utility trench excavations within the limits of the roadway improvements, including sidewalks, hardscape, etc., should consist of crushed rock with a maximum size of up to 1 in. that meets the requirements of Section 00405.14(b) of the ODOT SSC. In our opinion, the granular backfill should be placed in maximum 6-in.-thick lifts (loose) if using hand-operated vibratory plate compactors or tamping units. If heavier compaction equipment (e.g., a hoepack) is used, thicker lifts may be appropriate to prevent damage to newly placed conduits. The granular backfill should be compacted to at least 95% of the maximum dry density as determined by ASTM D698. Flooding or jetting the backfilled trenches with water to achieve the recommended compaction should not be permitted.

RETAINING WALLS

Retaining wall types are still being developed; however, we anticipate the walls will be either cantilever or gravity type. For areas where the ground line in front of the wall will be nearly horizontal, we recommend the toe of the wall be provided with a minimum 1 ft of embedment. Sloping ground in front of the wall will require additional embedment and can be evaluated when wall cross section details become available. The bottom of the wall footings should also be established behind or below a plane extending upward at 1½H:1V from the toe of the creek bank.

Our recommendations assume the walls will be founded in firm, undisturbed silty soils. If the near-surface soils in those areas are deemed unsuitable for foundation support, it will be necessary to overexcavate these materials and replace with compacted crushed rock. To provide more uniform support, walls should be founded on a minimum 12-in. thickness of compacted crushed rock. To evaluate sliding, we recommend an allowable value of 0.40 for the coefficient of friction for wall footings established in accordance with the above criteria. Wall footings established in accordance with the above criteria can be designed to impose an allowable soil bearing pressure of up to 1,500 psf. This value applies to the total of dead load plus frequently and/or permanently applied live loads and can be increased by one-third for the total of all loads: dead, live, and wind and/or seismic.

Design lateral earth pressures for embedded walls depend on the type of construction, i.e., the ability of the wall to yield. The two possible conditions regarding the ability of the wall to yield include the at-rest and the active earth pressure cases. The at-rest earth pressure case is applicable to a wall considered to be relatively rigid and laterally supported at the top and bottom and therefore unable to yield. The active earth pressure case is applicable to a wall capable of yielding slightly away from the backfill by either sliding or rotating about its base. A conventional cantilevered retaining wall is an example of a wall that develops the active earth pressure case by yielding. A basement wall where the top is restrained from deflecting is an example of a non-yielding wall.

Yielding and non-yielding walls can be designed using lateral earth pressures based on equivalent fluids having unit weights of 35 and 55 pcf, respectively. These design lateral earth pressures assume the wall backfill is completely drained and the grade behind the wall is horizontal. We recommend placing a minimum 1-ft-thick zone of ¾- to ¼-in. crushed rock with less than 2% passing the No. 200 sieve directly behind the wall to create a drained condition. The drain rock should be separated from silty soil by a non-woven geotextile filter fabric. A 4-in.-diameter perforated drain pipe (footing drain) should be placed near the bottom of the open-graded crushed-rock backfill and sloped to drain. General wall backfill should consist of granular structural fill material conforming to Section 00405.14(b) of the ODOT SSC. The backfill should be compacted to 95% of the maximum dry density as determined by ASTM D698, and backfill within 5 ft of the wall should be compacted to about 93% of the afore-mentioned standard. Compaction close to the walls should be accomplished with hand-operated vibratory plate compactors. Overcompaction of backfill could significantly increase lateral earth pressures behind walls and should be avoided.

Seismic loading on retaining walls depends on the type of wall and construction techniques. The Agusti and Sitar (2013) method was used to develop the seismically induced lateral earth pressures. The method applies a triangular lateral earth pressure distribution with a pressure of 0H (psf) at the ground surface and a maximum pressure of 8H and 18H (psf) for yielding and non-yielding walls, respectively, at the base of the wall, where H is the height of the wall. These pressures assume the backfill behind the structure is horizontal. The resultant force acts at a point above the base of the wall equal to one-third the wall height.

Additional lateral pressures due to surcharge loadings in the backfill area are in addition to the earth pressures. Additional lateral pressures induced by surcharge loads can be estimated using the guidelines provided on Figure 5. We recommend a minimum uniform vertical surcharge of 200 psf to account for construction equipment and vehicle traffic in locations where construction equipment and traffic will operate within 10 ft of the wall.

PAVEMENT DESIGN

GRI prepared recommendations for rehabilitation of existing pavements and new construction of a 12-ft-wide shared-use path along portions of SW 108th Avenue, SW Blake Street, and SW 105th Avenue.

Existing Pavement Condition

Based on a visual survey of the existing pavement conducted by GRI engineering staff, we observed medium- to high-severity fatigue cracking in localized areas throughout the project limits, including the intersection of SW 105th Avenue and SW Blake Street, as well as the intersection of SW Blake Street and SW 108th Avenue. We also observed full-lane maintenance or "skin" patches in both lanes south of SW Paulina Drive. We recommend areas of medium- and high-severity fatigue cracking be repaired prior to pavement rehabilitation. Extensive high-severity, load-associated cracking is present in both lanes between Stations 14+74 and 15+58 and therefore, full-depth reconstruction is warranted between these stations. The locations requiring localized repair are summarized in Table 3 below and Figures 19A and 20A in Appendix A.

Traffic Loading

As directed by the project team, we assumed there is minimal truck traffic loading. Therefore, the traffic-loading estimate used in the analysis is based on the recommended minimum cumulative 18-kip Equivalent Single Axle Load (ESAL) repetitions of 50,000 for a low-volume road, shown in the 1993 AASHTO Guide for Design of Pavement Structures for a given performance period.

Backcalculation

The FWD deflection data were analyzed to backcalculate the equivalent elastic moduli of the AC, base materials, and subgrade soils at the FWD test locations. The backcalculation analysis procedure and results are summarized in Table 1C in Appendix C.

Subgrade Design Values

The design modulus for new pavement construction was developed from the backcalculated subgrade moduli and represents the minimum from either the average or 2.3-percentile modulus divided by 0.70 subgrade modulus for the critical analysis unit. A discussion regarding this methodology is provided in Appendix C. Based on these results, a design modulus of 3,100 psi was used in the evaluation.

Pavement Design Analysis

The pavement design procedures utilized for this project are based on those outlined in the 1993 AASHTO Guide for Design of Pavement Structures and are also consistent with provisions in the ODOT Pavement Design Guide. Input parameters for pavement design analysis are outlined below in Table 2.

Table 2: AASHTO DESIGN PARAMETERS FOR FLEXIBLE PAVEMENT

Parameter	Design Values
Design Period, Years	20
Traffic Loading Case, ESALs	50,000
Design Reliability Level	80%
Initial Serviceability	4.2
Terminal Serviceability	2.5
Standard Deviation	0.49
Asphalt Concrete Layer Coefficient	0.42
Crushed-Rock Base Resilient Modulus, psi	
Existing CRB	Backcalculated at FWD tests
New Construction CRB	20,000
Subgrade Resilient Modulus, psi	
Existing SG	Backcalculated at FWD tests
New Construction	3,100

Rehabilitation

Based on the results of our analysis, we found that strengthening is warranted on SW Blake Street and SW 105th Street. The analysis indicates strengthening is not required on SW 108th Street, but surface rehabilitation is necessary due to low-severity fatigue and random cracking distress. Therefore, for the entire length of the project, we recommend cold plane milling to a depth of 2 in. followed by a 3-in. AC overlay, which will raise the grade by 1 in. and provide the required structural strengthening.

As noted earlier herein, there is a significant amount of low-severity fatigue and random cracking distress and a few areas where the road has been "skin patched," presumably due to underlying cracking distress. Therefore, where new AC (i.e., overlay or inlay) is placed over existing cracked AC, there is potential for reflective cracking (i.e., underlying cracks reflecting up through the new AC). In our opinion, the above recommendation (i.e., 2-in. mill and 3-in. overlay) should provide relatively good protection against premature reflective cracking. Milling has the advantage of not only providing a "roughened" surface that will assist with bonding the overlay to the underlying pavement, but it typically reduces the crack width by removal of surface spalling, which reduces the potential for the crack to reflect through the overlay. The time until significant reflecting cracking occurs is a function of the traffic loading, pavement stiffness and subgrade support conditions, environmental factors, and the properties of the AC mix. Typically, 3 in. of new AC should provide on the order of 8 to 10 years until significant reflective cracking occurs. However, if the City would like to remove the risk of reflective cracking, then the pavement should be reconstructed in order to remove the entire thickness of existing (cracked) AC.

Since there are grade constraints on SW 108th Street due to the curb and gutter, we recommend performing a taper mill on this street (i.e., transitioning the milling depth from 2 to 3 in.) in order to match grade at the gutter. Based on our core exploration on SW 108th (boring B-4), the AC thickness is only 3.75 in. thick.

Therefore, there is a potential that the entire thickness of AC will be removed due to the milling operation on this street (particularly near the edges where the milling depth is increased to 3 in). If there are areas where milling removes the entire thickness of AC, we recommend the existing aggregate be recompacted and a leveling course placed prior to placing the new AC.

New Construction

Design recommendations for new construction should be applied to areas of widening and localized digout repair listed in Table 3. We developed two design alternatives for new pavement construction: an aggregate stabilization with geotextile and a compacted subgrade section. The aggregate-stabilization design alternative was developed using the Giroud and Han procedures (Giroud and Han, 2004) for geosynthetic reinforcement above an undisturbed subgrade. For both design alternatives, a geotextile material is recommended as a separation layer between the subgrade soils and CRB to reduce the risk of contaminating the CRB layer from the underlying subgrade soils, which is a low-cost assurance of subgrade and base layer preservation. Design details are provided in Appendix C.

Localized Digout Repairs

Prior to rehabilitation, the areas of localized distress shown in Table 3 should be repaired using Alternative 2 given in the design recommendations for new pavement construction in areas with traffic loading provided below.

Table 3: GARDEN CORNER CURVES LOCALIZED DIGOUT REPAIR AREAS

Repair		Station			
No.	Direction	From	To	Width[1], ft	Area, sy
1	NB	11+83	11+99	6	1.25
2	NB	14+27	14+30	6	0.25
3	NB & SB	14+74	15+58	24	68.5
4	SB	19+90	20+15	6	7.5
5	SB	20+15	21+15	6	14.75
6	SB	23+53	23+59	6	0.75
7	SB	25+05	25+61	6	8.5
8	SB	25+09	25+11	6	0.5
9	SB	25+44	25+49	12	1.5
10	NB	29+42	29+54	6	1.5
				Total:	105

Note: [1]Width of localized repairs should either be the width of half the lane (i.e., 6 ft) or the entire width of the lane (i.e., 12 ft) as appropriate.

Pavement Design Recommendations

GRI recommends the following strategies for rehabilitation of existing pavements and construction of new pavements in areas with and without traffic loading.

Pavement Areas with Traffic Loading

Rehabilitation

- 3.0-in.-thick AC, Level 2, ½-in-size AC Performance Grade (PG) 64-22 Wearing Course
- 2.0-in.-thick Cold Plane Pavement Removal

New Construction Alternative 1 – Compacted Subgrade

- 3.0-in.-thick, Level 2, ½ in.-size AC PG 64-22 Wearing Course
- 2.0-in.-thick, Level 2, ½ in.-size AC PG 64-22 Base Course
- 8.0-in.-thick, 1-in.- or ¾-in.- size CRB
- Unwoven Geotextile
- 12-in.-thick Compacted Subgrade (Compacted to 95% AASHTO T99)

New Construction Alternative 2 – Aggregate Stabilization

- 3.0-in.-thick, Level 2, ½ in.-size AC PG 64-22 Wearing Course
- 2.0-in.-thick, Level 2, ½ in.-size AC PG 64-22 Base Course
- 14-in.-thick, 1-in.- or ¾-in.- size CRB
- Unwoven Geotextile
- Undisturbed Subgrade

Widening Areas without Traffic Loading

New Construction Alternative 1 – Compacted Subgrade

- 3.0-in.-thick, Level 2, ½-in.-size AC PG 64-22 Wearing Course
- 8.0-in.-thick, 1-in.- or ¾-in.- size CRB
- Unwoven Geotextile
- 12-in.-thick Compacted Subgrade (Compacted to 95% AASHTO T99)

New Construction Alternative 2 – Aggregate Stabilization

- 3.0-in.-thick, Level 2, ½ in.-size AC PG 64-22 Wearing Course
- 14-in.-thick, 1-in.- or ¾-in.- size CRB Base
- Unwoven Geotextile
- Undisturbed Subgrade

Recommendations for Materials and Construction. Construction materials and procedures should comply with the applicable sections of the 2018 ODOT SSC given below in Table 4.

Table 4: ODOT SPECIFICATIONS FOR CONSTRUCTION FOR GARDEN CORNER CURVES PROJECT

Materials/Activity	Specification
Cold Plane Pavement Removal	Standard Specification 00620 – Traffic should not be allowed to traffic the milled surface prior to placing overlay.
Asphalt Concrete Pavement Repair	Standard Specification 00748.
Asphalt Concrete Mix Design	Standard Specification 00744 – Use PG 64-22, Level 2 AC, Wearing & Base Course 1/2-in.-Size Leveling Course 3/8-in.-Size
Crushed-Rock Base	Standard Specification 00641 – Use1-in.-0 or 3/4-in.-0 Size.
Subgrade Compaction	Standard Specification 00330.
Geotextile Reinforcement	Standard Specification 00350.

Pavement Construction Considerations. The pavement section recommendations provided above assume pavement construction will be accomplished during the summer months or during warm, dry conditions and all workmanship and materials will conform to applicable ODOT specifications. A member of GRI's engineering staff should evaluate the exposed subgrade conditions during construction. During periods of wet weather or when wet ground conditions exist, it will likely be necessary to increase the thickness of the aggregate-base backfill to support construction equipment and protect the moisture-sensitive subgrade soils from disturbance. For wet-weather construction or if very soft subgrade conditions are encountered during construction, we anticipate the thickness of aggregate base, shown above, will need to be increased to provide a total of 24 in. of crushed rock. However, extremely soft subgrade or unsuitable materials may require additional thickness of aggregate base backfill to support construction traffic, which should be evaluated by GRI on a case-by-case basis.

DESIGN REVIEW AND CONSTRUCTION SERVICES

GRI should review geotechnical aspects of construction plans and specifications for this project as they are being developed. In addition, to observe compliance with the intent of our recommendations, design concepts, and the plans and specifications, we are of the opinion that a representative from GRI should observe construction operations dealing with earthwork. Our construction-phase services will allow for timely design changes if site conditions are encountered that differ from those described in this report. If we do not have the opportunity to confirm our interpretations, assumptions, and analyses during construction, we cannot be responsible for the application of our recommendations to subsurface conditions that are different from those described in this report.

LIMITATIONS

This report has been prepared to assist the owner and engineer in the design of this project. The scope is limited to the specific project and location described herein. Our description of the project represents our understanding of the significant aspects of the project relevant to the design and construction of the replacement culverts. In the event that any changes in the design and location of the modifications as outlined in this report are planned, we should be given the opportunity to review the changes and to modify or reaffirm the conclusions and recommendations of this report in writing.

The analyses and recommendations submitted in this report are based on the data obtained from the subsurface explorations made at the locations shown on Figure 2 and from the other sources of information discussed in this report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged that variations in soil conditions may exist between exploration locations and that groundwater levels will fluctuate with time. This report does not reflect any variations that may occur between these explorations. The nature and extent of variations may not become evident until construction. If, during construction, subsurface conditions differ from those described in this report or appear to be present beneath or beyond the limits of earthwork, we should be advised at once so that we can observe these conditions and reconsider our recommendations where necessary.

Submitted for GRI,

A. Wesley Spang, PhD, PE, GE	Lindsi A. Hammond, PE,	Tamara G. Kimball, PE, GE
Principal	Associate	Senior Engineer

This document has been submitted electronically.

References

AASHTO, 2017, LRFD BDS, 8th Edition.

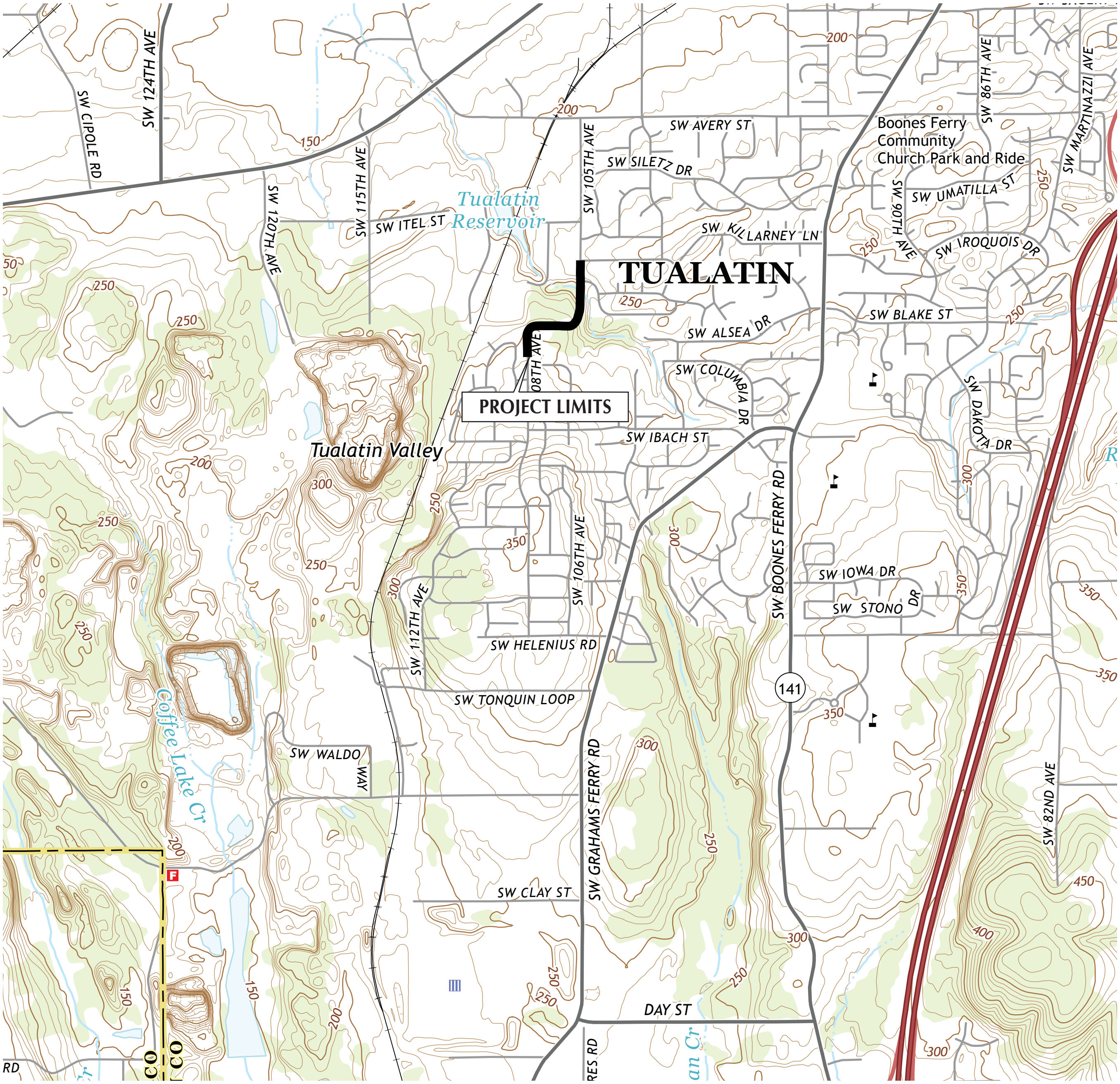
Agusti, G. C., and Sitar, N., 2013, Seismic earth pressures on retaining structures in cohesive soils, University of California, Berkeley, UCB GT 13-02.

Giroud, J. P., and Han, J., 2004, Design method for geogrid-reinforced unpaved roads. I. Development of design method and Design method for geogrid-reinforced unpaved roads. II. Calibration and applications, J. Geotech., Eng. Div., Am. Soc. Civ. Eng., 130(8), pp. 775-797.

Ma, L., Madin, I.P., Duplantis, S., and Williams, K.J., 2012, Lidar-Based Surficial Geologic Map and Database of the Greater Portland Area, Clackamas, Columbia, Marion, Multnomah, Washington, and Yamhill Counties, Oregon and Clark County, Washington, Oregon Department of Geology and Mineral Industries, Open-File Report O-12-02.

Oregon Department of Transportation, 2018, Standard specifications for highway construction.

U.S. Geological Survey (USGS), 2017, Quaternary fault and fold database of the United States, accessed 1/10/19 from USGS website: https://earthquake.usgs.gov/hazards/qfaults/.



USGS TOPOGRAPHIC MAP
SHERWOOD, OREG. (2017)

North

0 1/2 1 MILE

GRI

WALLIS ENGINEERING
GARDEN CORNER CURVES

VICINITY MAP

FEB. 2019 JOB NO. 6173 FIG. 1

SITE PLAN FROM FILE BY WALLIS ENGINEERING

INFILTRATION TEST COMPLETED BY GRI
(JANUARY 3-9, 2019)

BORING COMPLETED BY GRI
(JANUARY 16-17, 2019)

North

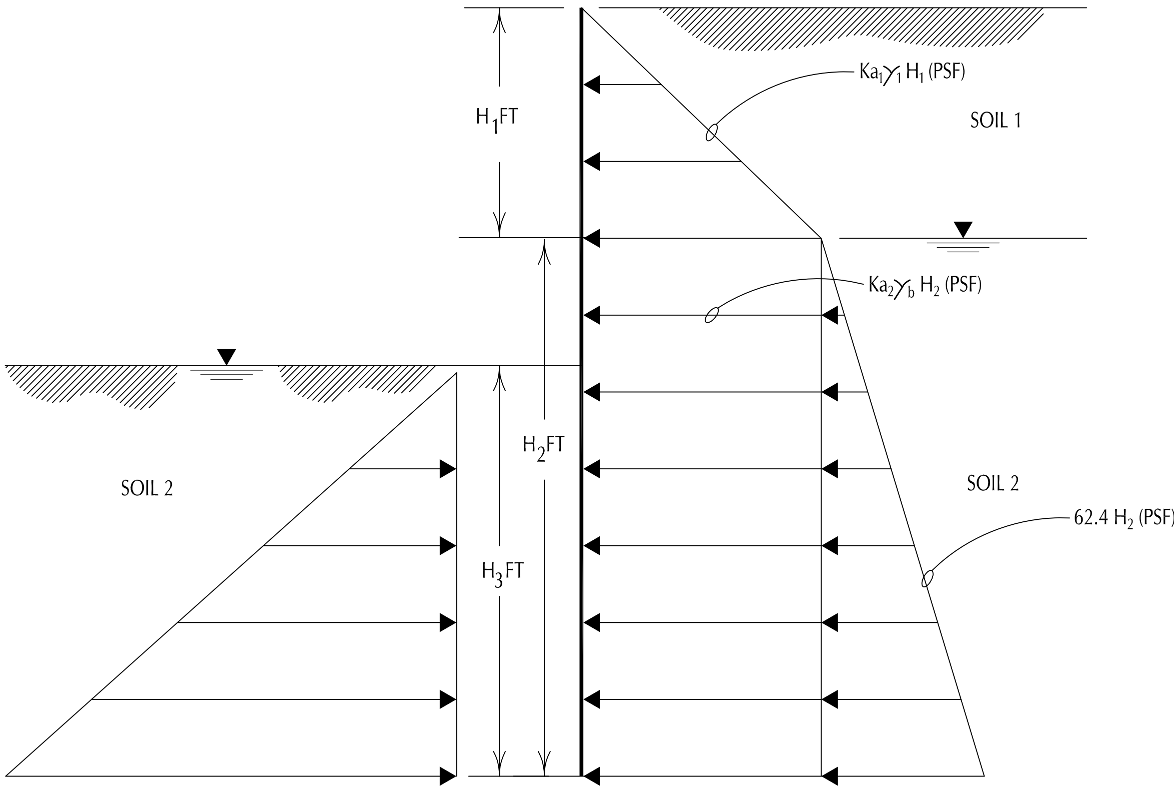
0 100 200 FT

GRI

WALLIS ENGINEERING
GARDEN CORNER CURVES

SITE PLAN

FEB. 2019 JOB NO. 6173 FIG. 2



Ka_1 = 0.4 (COEFFICIENT OF ACTIVE EARTH PRESSURE SOIL 1)
Ka_2 = 0.3 (COEFFICIENT OF ACTIVE EARTH PRESSURE SOIL 2)
Kp = 3.5 (COEFFICIENT OF PASSIVE EARTH PRESSURE, SOIL 2; INCLUDING F5 = 1.3)
γ_1 = 125 PCF (TOTAL UNIT WEIGHT SOIL 1)
γ_b = 55 PCF (TOTAL UNIT WEIGHT SOIL 2)

NOTES:

1) SURCHARGE EFFECTS FROM TRAFFIC, CONSTRUCTION EQUIPMENT, ETC., SHOULD BE ADDED TO THE ABOVE DESIGN PRESSURES. THE ACTUAL AMOUNT OF THIS SURCHARGE WILL DEPEND ON THE CONTRACTOR'S APPROACH TO THE WORK; HOWEVER, WE RECOMMEND A MINIMUM ADDITIONAL PRESSURE OF 200 PSF BE ADDED BEHIND THE WALL.

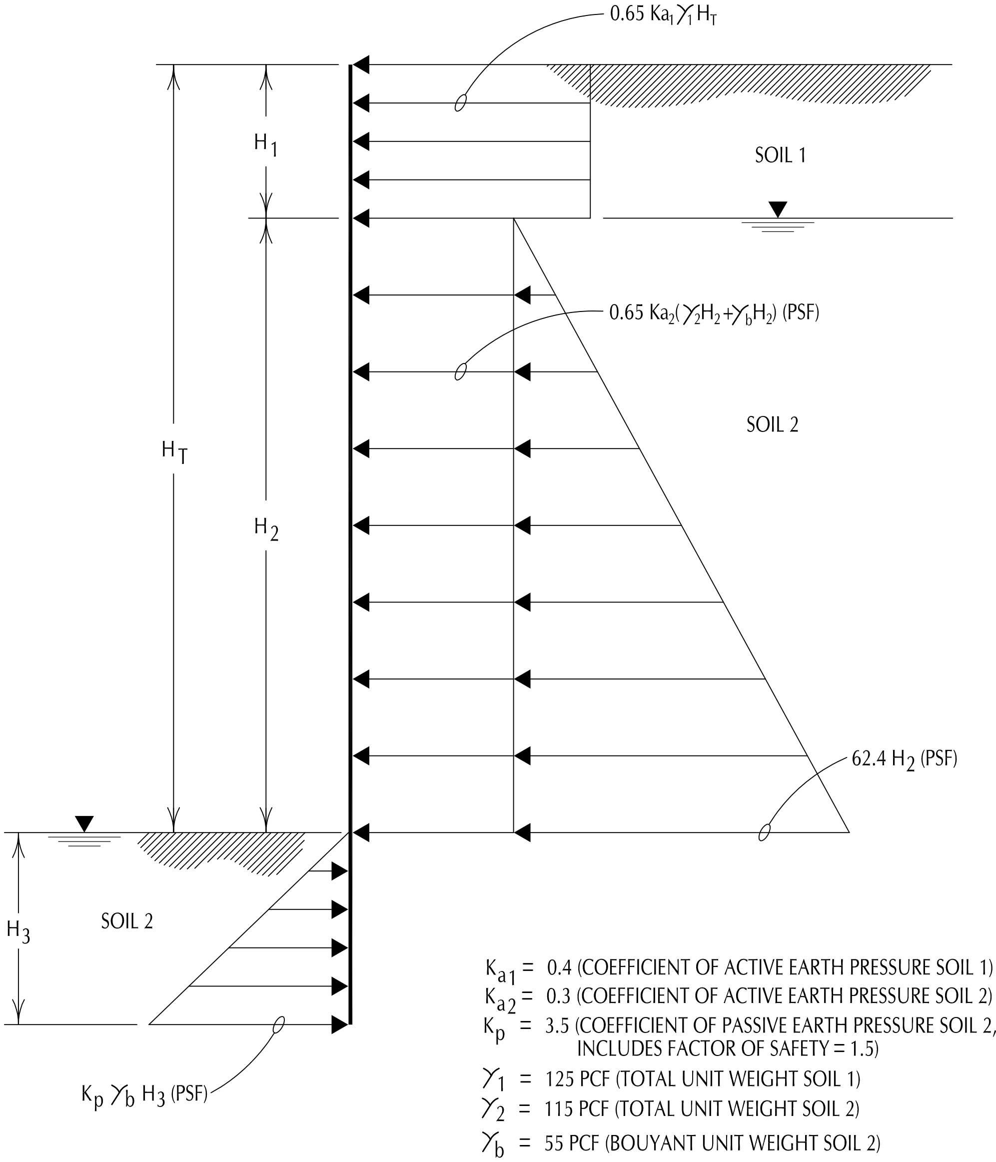
2) THE PRESSURES ACT OVER THE SURFACE AREAS OF THE SHEETS.

3) FOR CANTILEVERED SOLDIER PILES WITH LAGGING, BELOW THE BOTTOM OF THE EXCAVATION, PASSIVE AND ACTIVE PRESSURES ACT OVER TWO PILE DIAMETERS.

WALLIS ENGINEERING
GARDEN CORNER CURVES

EARTH PRESSURES FOR CANTILEVER SHORING

FEB. 2019 | JOB NO. 6173 | FIG. 3



NOTES:

1) SURCHARGE EFFECTS FROM TRAFFIC, CONSTRUCTION EQUIPMENT, ETC., SHOULD BE ADDED TO THE ABOVE DESIGN PRESSURES. THE ACTUAL AMOUNT OF THIS SURCHARGE WILL DEPEND ON THE CONTRACTOR'S APPROACH TO THE WORK; HOWEVER, WE RECOMMEND USING A MINIMUM UNIFORM PRESSURE OF 200 PSF.

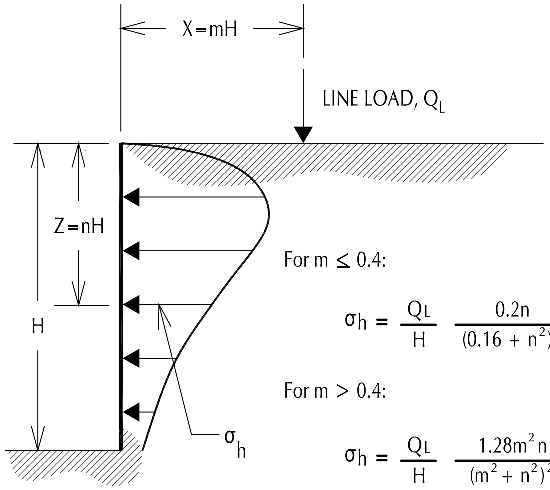
2) THE PRESSURES ACT OVER THE SURFACE AREA OF THE SHEETS.

GRI

WALLIS ENGINEERING
GARDEN CORNER CURVES

EARTH PRESSURES FOR BRACED SHORING

FEB. 2019 | JOB NO. 6173 | FIG. 4



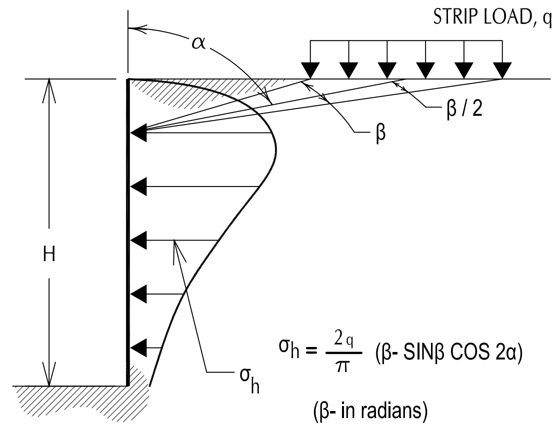
For $m \leq 0.4$:

$$\sigma_h = \frac{Q_L}{H} \frac{0.2n}{(0.16 + n^2)^2}$$

For $m > 0.4$:

$$\sigma_h = \frac{Q_L}{H} \frac{1.28m^2 n}{(m^2 + n^2)^2}$$

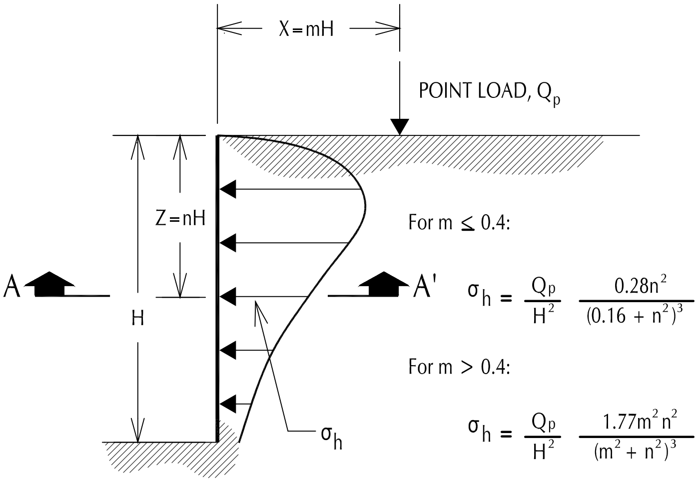
LINE LOAD PARALLEL TO WALL



$$\sigma_h = \frac{2q}{\pi} (\beta - \text{SIN}\beta \text{ COS } 2\alpha)$$

(β- in radians)

STRIP LOAD PARALLEL TO WALL

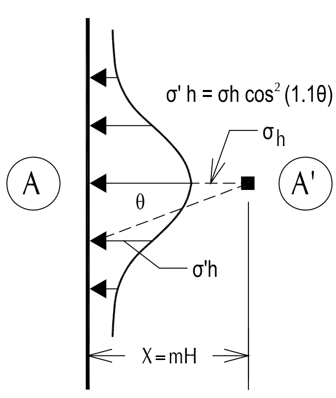


For $m \leq 0.4$:

$$\sigma_h = \frac{Q_p}{H^2} \frac{0.28n^2}{(0.16 + n^2)^3}$$

For $m > 0.4$:

$$\sigma_h = \frac{Q_p}{H^2} \frac{1.77m^2 n^2}{(m^2 + n^2)^3}$$



DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD

NOTES:

1) THESE GUIDELINES APPLY TO RIGID WALLS WITH POISSON'S RATIO ASSUMED TO BE 0.5 FOR BACKFILL MATERIALS.

2) LATERAL PRESSURES FROM ANY COMBINATION OF ABOVE LOADS MAY BE DETERMINED BY THE PRINCIPLE OF SUPERPOSITION.

GRI

WALLIS ENGINEERING
GARDEN CORNER CURVES

SURCHARGE-INDUCED LATERAL PRESSURE

FEB. 2019 | JOB NO. 6173 | FIG. 5

APPENDIX A

Field Explorations and Laboratory Testing

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATIONS

Borings

Subsurface materials and conditions at the site were investigated on January 3 through 17, 2019, with five borings, designated B-1 through B-5, and three hand-augered borings, designated I-1 through I-3, which were completed for infiltration testing. The approximate locations of the borings are shown on Figure 2. Hollow-stem auger drilling techniques were used to advance the borings through the overburden soils. The borings were completed with a Geoprobe 7720DT track-mounted drill rig provided and operated by Western States Soil Conservation, Inc., of Hubbard, Oregon. All drilling and sampling operations were observed by a representative of GRI, who maintained a log of the materials and conditions disclosed during the course of the work.

The borings were advanced to depths of about 3 to 31.5 ft below the ground surface. Disturbed samples were typically obtained at 2.5- to 5-ft intervals using a standard split-spoon sampler. At the time of sampling, the Standard Penetration Test (SPT) was conducted. This test consists of driving a standard split-spoon sampler into the soil a distance of 18 in. using a 140-lb hammer dropped 30 in. The number of blows required to drive the sampler the last 12 in. is known as the Standard Penetration Resistance, or SPT N-value. SPT N-values provide a measure of the relative density of granular soils, such as sand, and the relative consistency, or stiffness, of cohesive soils, such as silt. The split-spoon samples were carefully examined in the field and representative portions were saved in airtight jars. In addition, relatively undisturbed samples were collected by pushing a 3-in.-outside-diameter (O.D.) Shelby tube into the undisturbed soil a maximum distance of 24 in. using the hydraulic ram of the drill rig. The soil exposed in the end of the Shelby tube was examined and classified in the field. After classification, the tube was sealed with rubber caps. All samples were returned to our laboratory for further examination and physical testing.

Logs of the borings are provided on Figures 1A through 7A. Each log provides a descriptive summary of the various types of materials encountered in the borings and notes the depths where the materials and characteristics of the materials change. The terms used to describe the soil encountered in the explorations are defined in Table 1A and on the attached legend. To the left of the descriptive summaries, the depths and types of samples taken, driving resistances (SPT N-values), and moisture contents are indicated. To the right of the descriptive summary, a graphic log indicates the general soil types encountered in the borings. Where applicable, fines contents are also summarized. The ground surface elevations shown on the boring logs were estimated using the available topographic information provided on Figure 2.

Infiltration Testing

Encased falling head infiltration testing was completed in borings I-1 through I-3 on January 3 and 9, 2019, at a depth of about 3 ft in general conformance with the City of Portland 2016 Stormwater Management Manual (SMM). The testing consisted of advancing a 4-in.-O.D. hand auger to a depth of about 4 ft. An approximately 5-ft-long, 3-in.-inside-diameter (I.D.), 1/4-in.-thick walled, open-ended PVC standpipe was embedded approximately 3 to 6 in. into the bottom of the borehole. The auger was filled with water to about 12 in. above the soil level and allowed to soak for a minimum of 1 hour. After the soaking period, the water level inside the auger was measured at approximately 10- to 15-minute intervals for approximately 1 hour. After the infiltration testing was completed, disturbed samples of the material were collected and examined in the field and selected portions were saved in airtight jars for further examination and physical testing in our laboratory. Logs of the hand-augered borings are provided on Figures 6A through 7A, and a summary of the laboratory test results for the hand-augered borings is provided in Table 2A.

Pavement Cores

The pavement was cored at each pavement boring location to assist in evaluation of the type, thickness, and condition of the pavement encountered. The pavement was cored using an electric drill owned and operated by GRI. Logs of the pavement cores are provided on Figures 9A through 13A. Photographs of the core locations and the core samples are shown on Figures 14A through 18A.

LABORATORY TESTING

General

All samples obtained from the borings were returned to our laboratory, where the physical characteristics of the samples were noted and the field classifications modified where necessary. The laboratory testing program included determinations of natural moisture content and washed-sieve analyses. A summary of laboratory test results is provided in Table 2A. The following paragraphs describe the testing program in more detail.

Natural Moisture Content

Natural moisture content determinations were made in conformance with ASTM International (ASTM) D2216. The results are shown on Figures 1A through 7A and in Table 2A.

Washed Sieve Analyses

Washed-sieve analyses were performed for selected soil samples obtained from the borings to assist in their classification. The test is performed by taking a sample of known dry weight and washing it over a No. 200 sieve. The material retained on the sieve is oven-dried and reweighed and the percentage of material (by weight) that passed the No. 200 sieve is calculated. Test results shown on Figures 1A through 7A and in Table 2A.

Undisturbed Unit Weight

The unit weight, or density, of undisturbed soil samples was determined in the laboratory in substantial conformance with ASTM D2937. The results are summarized on Figure 5A and in Table 2A.

Atterberg Limits

Atterberg limits were determined for selected soil samples in conformance with ASTM D4318. The test results are summarized on Figures 2A, 3A, and 5A and in Table 2A.

Table 1A: GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

Relative Density	Standard Penetration Resistance (N-values), blows per ft
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	over 50

Description of Consistency for Fine-Grained (Cohesive) Soils

Consistency	Standard Penetration Resistance (N-values), blows per ft	Torvane or Undrained Shear Strength, tsf
Very Soft	0 - 2	less than 0.125
Soft	2 - 4	0.125 - 0.25
Medium Stiff	4 - 8	0.25 - 0.50
Stiff	8 - 15	0.50 - 1.0
Very Stiff	15 - 30	1.0 - 2.0
Hard	over 30	over 2.0

Grain-Size Classification

Boulders:
- >12 in.

Cobbles:
- 3 - 12 in.

Gravel:
- $^1/_4$ - $^3/_4$ in. (fine)
- $^3/_4$ - 3 in. (coarse)

Sand:
- No. 200 - No. 40 sieve (fine)
- No. 40 - No. 10 sieve (medium)
- No. 10 - No. 4 sieve (coarse)

Silt/Clay:
- pass No. 200 sieve

Modifier for Subclassification

Adjective	Primary Constituent SAND or GRAVEL	Primary Constituent SILT or CLAY
	Percentage of Other Material (by weight)	
trace:	5 - 15 (sand, gravel)	5 - 15 (sand, gravel)
some:	15 - 30 (sand, gravel)	15 - 30 (sand, gravel)
sandy, gravelly:	30 - 50 (sand, gravel)	30 - 50 (sand, gravel)
trace:	<5 (silt, clay)	*Relationship of clay and silt determined by plasticity index test*
some:	5 - 12 (silt, clay)	
silty, clayey:	12 - 50 (silt, clay)	

Table 2A

SUMMARY OF LABORATORY RESULTS

Sample Information						Atterberg Limits			
Location	Sample	Depth, ft	Elevation, ft	Moisture Content, %	Dry Unit Weight, pcf	Liquid Limit, %	Plasticity Index, %	Fines Content, %	Soil Type
B-1	S-2	1.5	244.5	20	--	--	--	--	Sandy SILT
	S-3	3.0	243.0	23	--	--	--	58	Sandy SILT
	S-4	5.0	241.0	21	--	--	--	--	SILT
	S-5	7.5	238.5	25	--	--	--	77	SILT
	S-6	10.0	236.0	22	--	--	--	--	SILT
B-2	S-2	1.0	242.0	25	--	--	--	--	SILT
	S-3	3.0	240.0	28	--	46	18	80	Clayey SILT
	S-4	5.0	238.0	31	--	--	--	--	Clayey SILT
	S-5	7.5	235.5	28	--	--	--	--	Clayey SILT
	S-6	10.0	233.0	25	--	--	--	--	Silty SAND
B-3	S-2	1.5	230.5	26	--	36	5	62	Sandy SILT
	S-3	3.0	229.0	25	--	--	--	--	Silty SAND
	S-4	5.0	227.0	27	--	--	--	--	Silty SAND
	S-5	7.5	224.5	26	--	--	--	--	Silty SAND
	S-6	10.0	222.0	30	--	28	3	72	SILT
B-4	S-2	1.5	252.5	21	--	--	--	--	SILT
	S-3	3.0	251.0	21	--	--	--	67	Sandy SILT
	S-4	5.0	249.0	32	--	--	--	--	SILT
	S-5	7.5	246.5	32	--	--	--	--	SILT
	S-6	10.0	244.0	33	--	--	--	--	SILT
B-5	S-2	5.0	198.0	37	--	35	11	64	Sandy CLAY
	S-3	7.5	195.5	78	--	--	--	--	Sandy SILT
	S-4	10.0	193.0	42	--	--	--	53	Sandy SILT
	S-4	11.0	192.0	40	--	--	--	--	Sandy SILT
	S-5	13.0	190.0	34	87	--	--	--	Silty SAND
	S-5	14.0	189.0	38	--	--	--	23	Silty SAND
	S-6	15.0	188.0	40	--	--	--	--	Silty SAND
	S-6	15.5	187.5	40	--	--	--	--	Silty SAND
	S-7	20.0	183.0	31	--	--	--	--	Silty SAND
	S-8	25.0	178.0	36	--	32	7	96	SILT
	S-8	26.0	177.0	34	--	--	--	--	CLAY
	S-9	26.5	176.5	32	--	--	--	95	CLAY
	S-9	27.0	176.0	31	95	--	--	--	CLAY
	S-9	28.0	175.0	35	--	--	--	89	CLAY
	S-10	30.0	173.0	34	--	--	--	--	CLAY
I-1	S-1	2.5	221.5	37	--	--	--	88	SILT
	S-2	3.5	220.5	38	--	--	--	82	SILT
I-2	S-1	2.5	229.5	36	--	--	--	81	Silty CLAY
I-3	S-1	2.5	249.5	30	--	--	--	64	Sandy SILT

BORING AND TEST PIT LOG LEGEND

SOIL SYMBOLS

Symbol	Typical Description
	LANDSCAPE MATERIALS
	FILL
	GRAVEL; clean to some silt, clay, and sand
	Sandy GRAVEL; clean to some silt and clay
	Silty GRAVEL; up to some clay and sand
	Clayey GRAVEL; up to some silt and sand
	SAND; clean to some silt, clay, and gravel
	Gravelly SAND; clean to some silt and clay
	Silty SAND; up to some clay and gravel
	Clayey SAND; up to some silt and gravel
	SILT; up to some clay, sand, and gravel
	Gravelly SILT; up to some clay and sand
	Sandy SILT; up to some clay and gravel
	Clayey SILT; up to some sand and gravel
	CLAY; up to some silt, sand, and gravel
	Gravelly CLAY; up to some silt and sand
	Sandy CLAY; up to some silt and gravel
	Silty CLAY; up to some sand and gravel
	PEAT

BEDROCK SYMBOLS

Symbol	Typical Description
	BASALT
	MUDSTONE
	SILTSTONE
	SANDSTONE

SURFACE MATERIAL SYMBOLS

Symbol	Typical Description
	Asphalt concrete PAVEMENT
	Portland cement concrete PAVEMENT
	Crushed rock BASE COURSE

SAMPLER SYMBOLS

Symbol	Sampler Description
	2.0-in. O.D. split-spoon sampler and Standard Penetration Test with recovery (ASTM D1586)
	Shelby tube sampler with recovery (ASTM D1587)
	3.0-in. O.D. split-spoon sampler with recovery (ASTM D3550)
	Grab Sample
	Rock core sample interval
	Sonic core sample interval
	Geoprobe sample interval

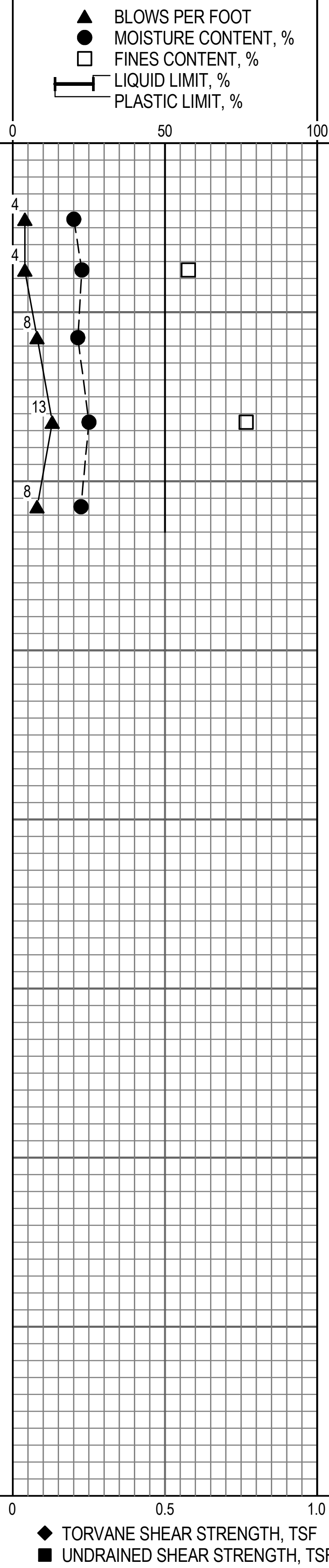
INSTALLATION SYMBOLS

Symbol	Symbol Description
	Flush-mount monument set in concrete
	Concrete, well casing shown where applicable
	Bentonite seal, well casing shown where applicable
	Filter pack, machine-slotted well casing shown where applicable
	Grout, vibrating-wire transducer cable shown where applicable
P	Vibrating-wire pressure transducer
	1-in.-diameter solid PVC
	1-in.-diameter hand-slotted PVC
	Grout, inclinometer casing shown where applicable

FIELD MEASUREMENTS

Symbol	Typical Description
	Groundwater level during drilling and date measured
	Groundwater level after drilling and date measured
	Rock core recovery (%)
	Rock quality designation (RQD, %)

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Surface Elevation: 246.0 ft [±] (NAVD 88)						
		Asphalt concrete PAVEMENT (7.25 in.) over crushed rock BASE COURSE (4.75 in.)	245.0 / 1.0		S-1			
		Sandy SILT, trace to some clay, gray to brown mottled rust, soft to medium stiff, fine-grained sand, contains organics and wood debris (Possible Fill)			S-2		3 2 2	
					S-3		2 2 2	
5		SILT, trace to some fine-grained sand, brown mottled rust, medium stiff to stiff	241.0 / 5.0		S-4		3 4 4	
					S-5		4 7 6	
10					S-6		3 4 4	
		(1/16/2019)	234.5 / 11.5					
		Groundwater not encountered						
15								
20								
25								
30								
35								
40								



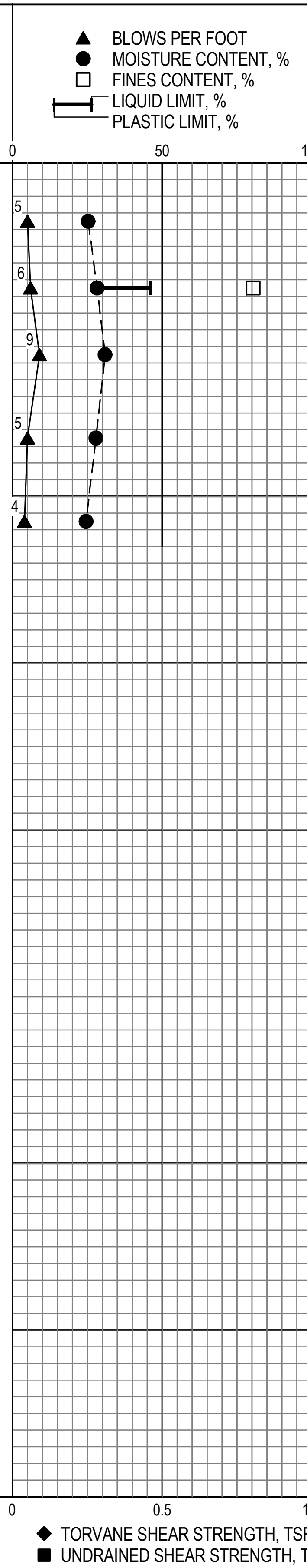
GRI BORING LOG (GPS) GRI DATA TEMPLATE.GDT 2/4/19

Logged By: G. Timm	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 1/16/19	**GPS Coordinates:** Not Available
Drilling Method: Hollow-Stem Auger **Equipment:** Geoprobe 7720 DT **Hole Diameter:** 6 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:**
Note: See Legend for Explanation of Symbols	

GRI

BORING B-1

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT
		Surface Elevation: 243.0 ft [±] (NAVD 88)					
		Asphalt concrete PAVEMENT (4.25 in.) over crushed rock BASE COURSE (6.75 in.)	242.1 / 0.9		S-1		
		SILT, some clay to clayey, trace fine- to coarse-grained sand, brown mottled rust, medium stiff			S-2		2 2 3
		---clayey, trace to some fine-grained sand, brown below 3 ft			S-3		2 3 3
5		---some sand to sandy below 5 ft			S-4		2 4 5
					S-5		2 2 3
10		Silty SAND, gray-brown, loose, fine grained	233.0 / 10.0		S-6		1 2 2
		(1/16/2019)	231.5 / 11.5				
		Groundwater not encountered					
15							
20							
25							
30							
35							
40							



COMMENTS AND ADDITIONAL TESTS

Logged By: G. Timm	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 1/16/19	**GPS Coordinates:** Not Available
Drilling Method: Hollow-Stem Auger **Equipment:** Geoprobe 7720 DT **Hole Diameter:** 6 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:**
Note: See Legend for Explanation of Symbols	

GRI

BORING B-2

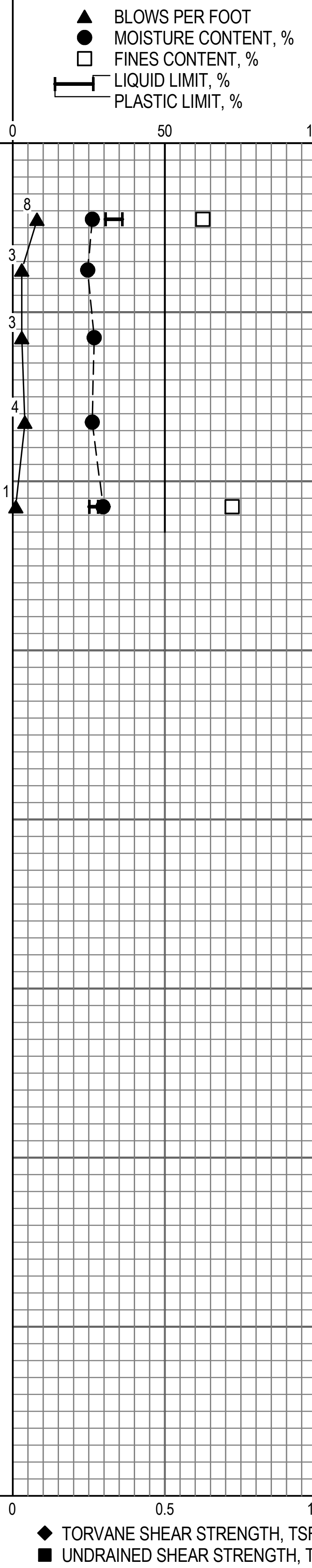
FEB. 2019 JOB NO. 6173 FIG. 2A

BORING B-3

Surface Elevation: 232.0 ft [±] (NAVD 88)

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT
0	Asphalt concrete PAVEMENT (6.5 in.) over crushed rock BASE COURSE (11.5 in.)	230.5 / 1.5
1.5	Sandy SILT, brown, medium stiff to stiff, fine-grained sand	229.0 / 3.0
3.0	Silty SAND to sandy SILT, brown, very loose to soft, fine-grained sand	222.0 / 10.0
10.0	SILT, some fine-grained sand, trace clay, brown, very soft	220.5 / 11.5
11.5	(1/16/2019) Groundwater not encountered	

SAMPLE NO.	BLOW COUNT	BLOWS PER FOOT
S-1		
S-2	2, 4, 4	8
S-3	2, 2, 1	3
S-4	1, 1, 2	3
S-5	1, 2, 2	4
S-6	1, 0, 1	1

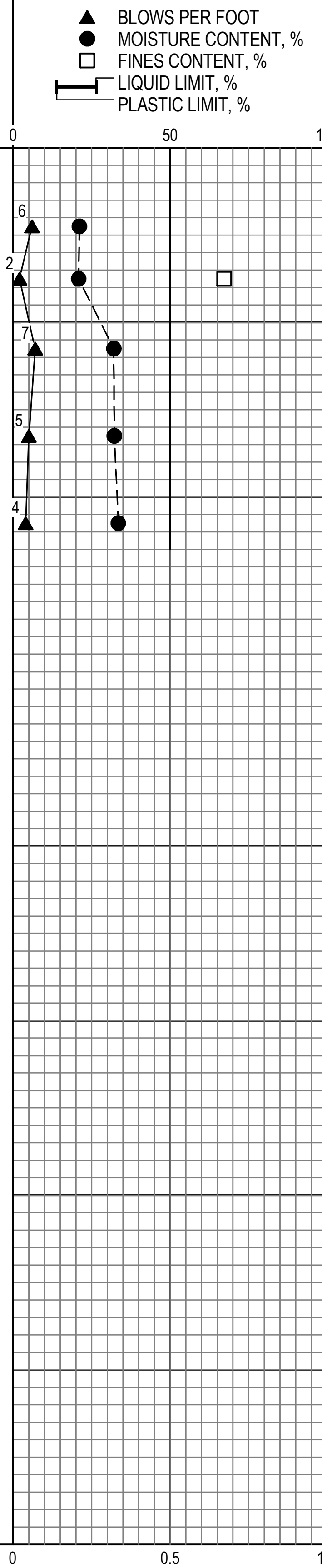


COMMENTS AND ADDITIONAL TESTS

Logged By: G. Timm	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 1/16/19	**GPS Coordinates:** Not Available
Drilling Method: Hollow-Stem Auger **Equipment:** Geoprobe 7720 DT **Hole Diameter:** 6 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:**
Note: See Legend for Explanation of Symbols	

GRI

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT
		Surface Elevation: 254.0 ft [±] (NAVD 88)					
		Asphalt concrete PAVEMENT (3.75 in.) over crushed rock BASE COURSE (11.25 in.)	252.8 / 1.3		S-1		
		SILT, trace to some clay, trace fine- to medium-grained sand, brown mottled rust, medium stiff, contains wood debris (Possible Fill)			S-2		8 4 2
		---sandy below 3 ft	249.0 / 5.0		S-3		1 1 1
5		SILT, some fine-grained sand to sandy, gray to brown mottled rust, medium stiff			S-4		1 3 4
					S-5		2 3 2
10		---fine- to medium-grained sand, soft to medium stiff below 10 ft	242.5 / 11.5		S-6		1 2 2
		(1/17/2019)					
		Groundwater not encountered					
15							
20							
25							
30							
35							
40							



COMMENTS AND ADDITIONAL TESTS

Logged By: G. Timm	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 1/17/19	**GPS Coordinates:** Not Available
Drilling Method: Hollow-Stem Auger **Equipment:** Geoprobe 7720 DT **Hole Diameter:** 6 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:**
Note: See Legend for Explanation of Symbols	

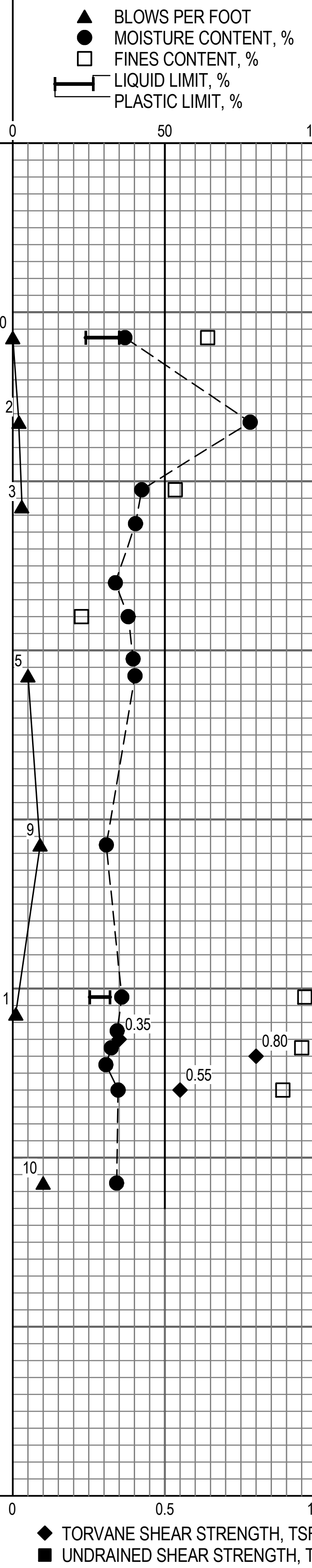
GRI

BORING B-4

FEB. 2019 JOB NO. 6173 FIG. 4A

BORING B-5

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	SAMPLE NO.	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	Surface Elevation: 203.0 ft [±] (NAVD 88)				
0	Asphalt concrete PAVEMENT (8.5 in.) over crushed rock BASE COURSE (6 in.)	201.8 / 1.2	S-1		
	Sandy CLAY, some silt, gray, soft, fine-grained sand, contains fine organics				Boring advanced by hydroexcavation to a depth of approximately 5 ft. SPT sample S-2 was likely disturbed by air knife; blow counts may not be representative
5			S-2	0 0 0	
	Sandy SILT to silty SAND, gray, soft to very loose, fine- to medium-grained sand, contains wood debris	195.5 / 7.5	S-3	0 1 1	
10	---light brown at 11 ft		S-4	1 2 1	▽11.5 ft (1/17/2019)
	---silty SAND, loose, moderately cemented below 12.5 ft		S-5		Dry Density = 87 pcf
15			S-6	1 2 3	
20			S-7	3 5 4	
25	SILT, trace fine-grained sand, gray, very soft	178.0 / 25.0	S-8	0 0 1	
	CLAY, trace silt and fine-grained sand, dark gray, soft ---some silt to silty, trace to some sand, dark gray mottled rust, stiff below 27 ft	177.0 / 26.0	S-9		Dry Density = 95 pcf
30	---red-brown mottled rust below 30 ft		S-10	3 4 6	
	(1/17/2019)	171.5 / 31.5			
35					
40					



Logged By: G. Timm	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 1/17/19	**GPS Coordinates:** Not Available
Drilling Method: Mud Rotary **Equipment:** Geoprobe 7720 DT **Hole Diameter:** 6 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:**
Note: See Legend for Explanation of Symbols	

GRI

BORING B-5

FEB. 2019 JOB NO. 6173 FIG. 5A

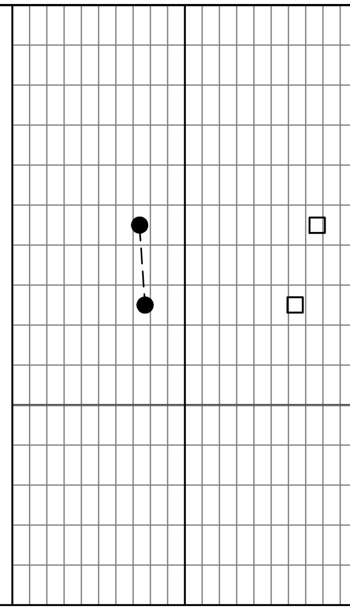
DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	SAMPLE NO.	SAMPLE TYPE	● MOISTURE CONTENT, % □ FINES CONTENT, % ├┤ LIQUID LIMIT, % PLASTIC LIMIT, % (0 50 100)	COMMENTS AND ADDITIONAL TESTS

I-1

Surface Elevation: 224.0 ft [±] (NAVD 88)

Depth, ft	Classification of Material	Elevation, ft / Depth, ft	Sample No.
	SILT, trace to some fine-grained sand, trace clay, brown mottled rust, 3-in.-thick heavily rooted zone at ground surface	220.0 / 4.0	S-1, S-2
5	(1/3/2018) Groundwater not encountered		

0 50 100



0 0.5 1.0

◆ TORVANE SHEAR STRENGTH, TSF

Logged By: C. Smerdon **Excavated by:** GRI **Equipment:** Hand Auger

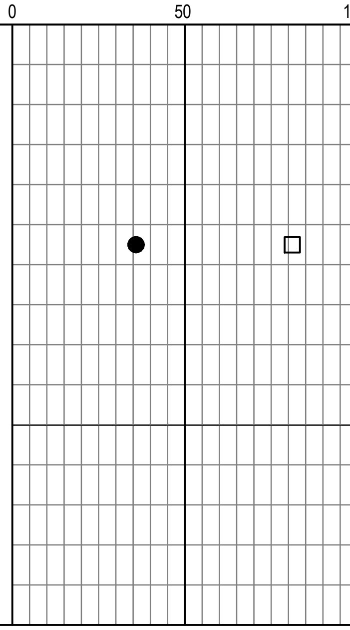
Date Started: 1/3/19 **Coordinates:** Not Available **Note:** See Legend for Explanation of Symbols

I-2

Surface Elevation: 232.0 ft [±] (NAVD 88)

Depth, ft	Classification of Material	Elevation, ft / Depth, ft	Sample No.
	Silty GRAVEL, angular, 2-in.-thick heavily rooted zone at ground surface (Possible Fill)	231.0 / 1.0	
	Silty CLAY, some fine-grained sand, brown, contains organics	229.0 / 3.0	S-1
5	(1/9/2018) Groundwater not encountered		

0 50 100



0 0.5 1.0

◆ TORVANE SHEAR STRENGTH, TSF

Logged By: G. Timm **Excavated by:** GRI **Equipment:** Hand Auger

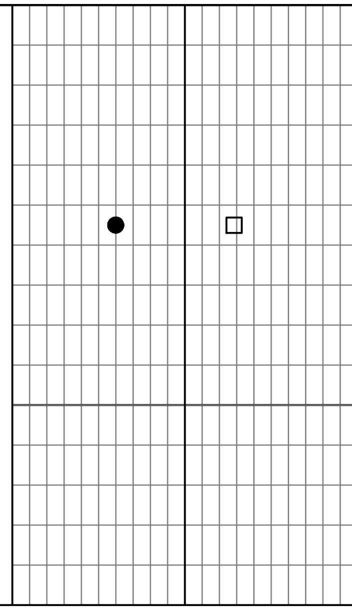
Date Started: 1/9/19 **Coordinates:** Not Available **Note:** See Legend for Explanation of Symbols

GRI HA LOG (LAT/LONG) - 2 PP GRI DATA TEMPLATE.GDT 2/4/19

BORINGS

FEB. 2019 JOB NO. 6173 FIG. 6A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	SAMPLE NO.	SAMPLE TYPE	● MOISTURE CONTENT, % □ FINES CONTENT, % LIQUID LIMIT, % PLASTIC LIMIT, % (0 50 100)	COMMENTS AND ADDITIONAL TESTS
I-3		**Surface Elevation:** 252.0 ft [±] (NAVD 88)				0 50 100	
		Sandy SILT, brown, fine-grained sand, contains organics, 3-in.-thick heavily rooted zone at ground surface	249.0 / 3.0	S-1			
5		(1/9/2019) Groundwater not encountered					



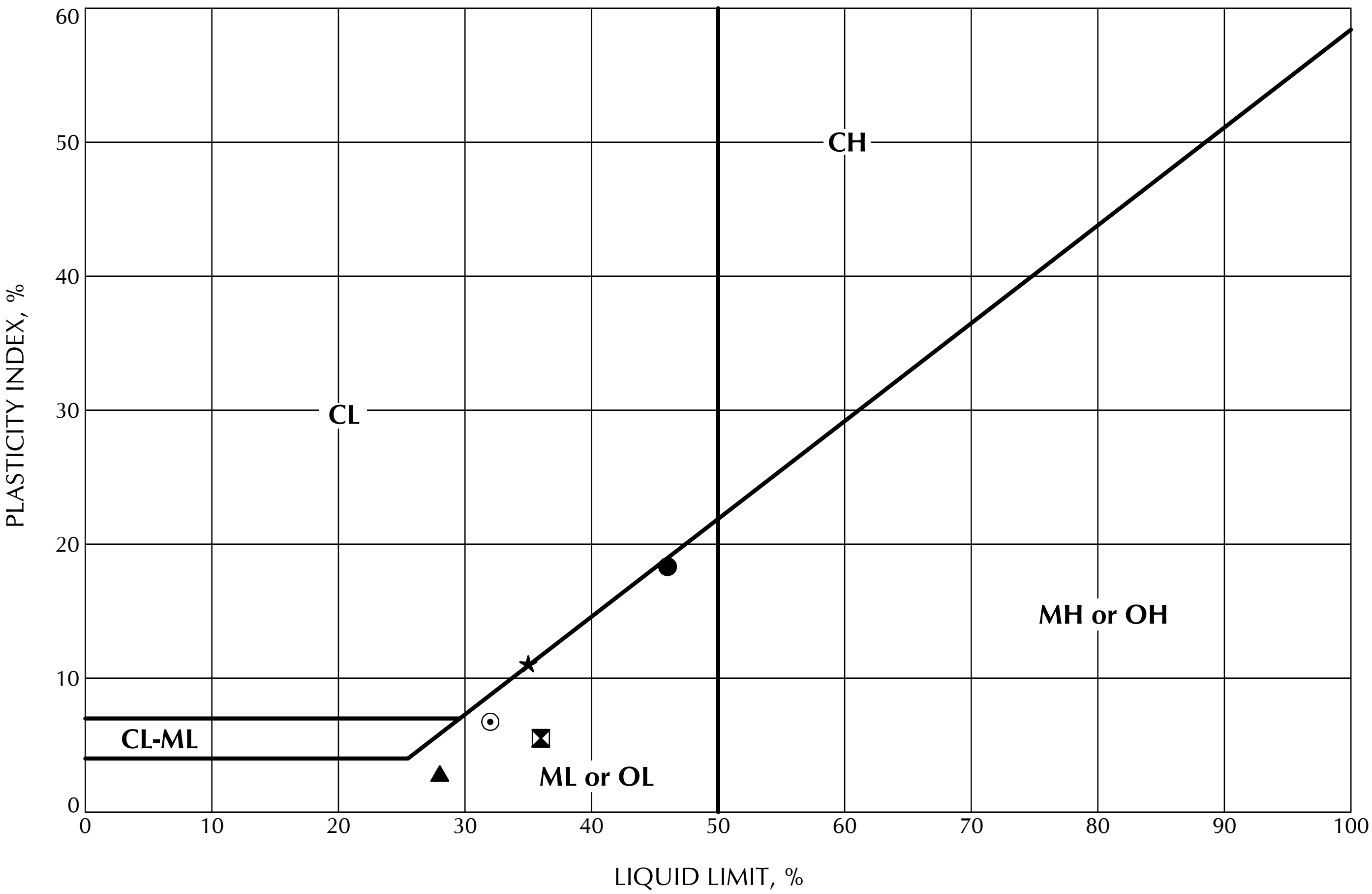
Logged By: G. Timm | **Excavated by:** GRI | **Equipment:** Hand Auger

Date Started: 1/9/19 | **Coordinates:** Not Available | **Note:** See Legend for Explanation of Symbols

GRI HA LOG (LAT/LONG) - 2 PP GRI DATA TEMPLATE.GDT 2/4/19

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
ML	INORGANIC CLAYEY SILTS TO VERY FINE SANDS OF SLIGHT PLASTICITY
CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
MH	INORGANIC SILTS AND CLAYEY SILT
CH	INORGANIC CLAYS OF HIGH PLASTICITY



	Location	Sample	Depth, ft	Classification	LL	PL	PI	MC, %
●	B-2	S-3	3.0	Clayey SILT, some fine-grained sand, brown	46	28	18	28
⧗	B-3	S-2	1.5	Sandy SILT, brown, fine-grained sand	36	31	5	26
▲	B-3	S-6	10.0	SILT, some fine-grained sand, trace clay, brown	28	25	3	30
★	B-5	S-2	5.0	Sandy CLAY, some silt, gray, fine-grained sand	35	24	11	37
⊙	B-5	S-8	25.0	SILT, trace fine-grained sand, gray	32	25	7	36

GRI

PLASTICITY CHART

FEB. 2019 JOB NO. 6173 FIG. 8A

GRI
9750 SW Nimbus Avenue
Beaverton, OR 97008-7172
p| 503-641-3478 f| 503-644-8034

PAVEMENT CORE LOG

Saved: Yes No

Project: Tualatin Garden Corner Curves

Project No.: 6173

Highway: SW Blake St

Date: 1-16-19

Key No.:

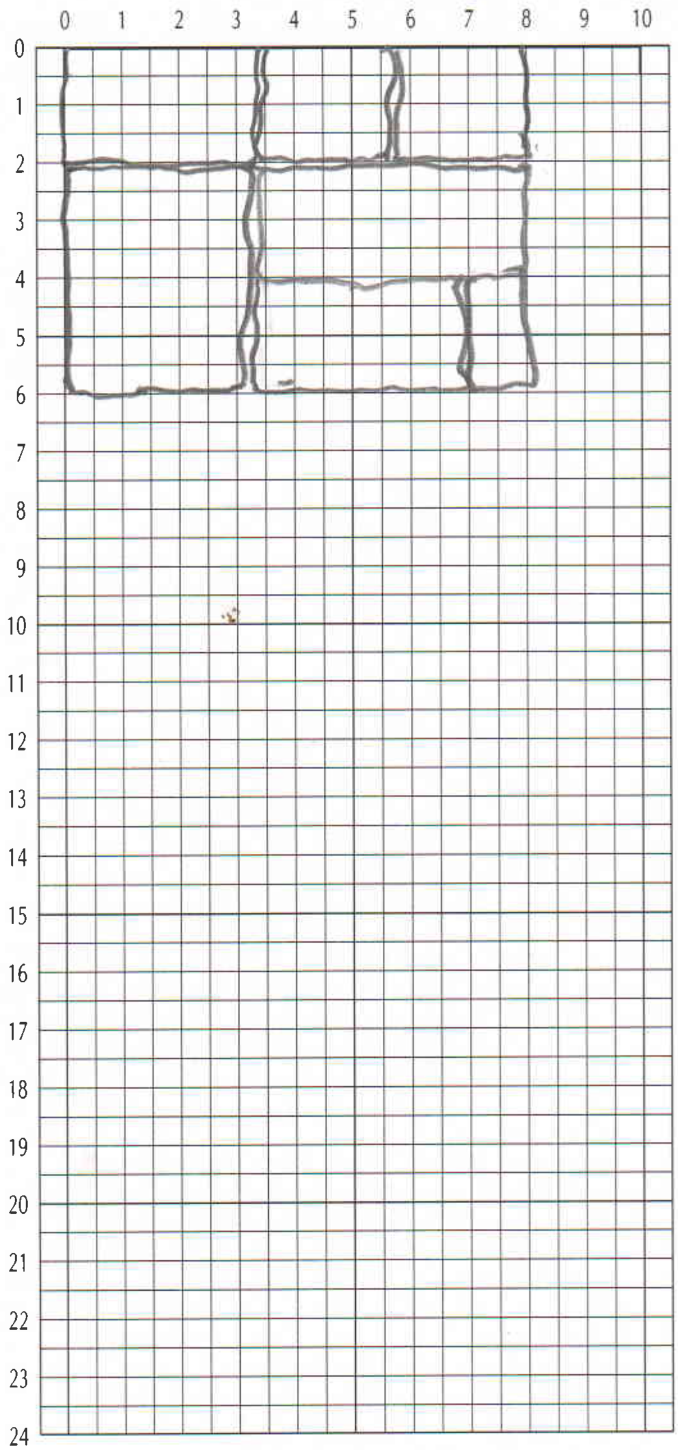
Milepost:

Core No.: B-1

Location: Southbound Lane

Logged By. GDT

Core Length: 6.0"



DRILLED THROUGH PATCH: NO YES

DRILLED ON CRACK: NO YES (Trans. Long. Fat. Top Down Other)

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

NOTE DISTANCE FROM EDGE OF PAVEMENT AND DIRECTION:

EP EP

ODOT PAVEMENT CORE LOG TEMPLATE

PAVEMENT BORING CORE LOGS

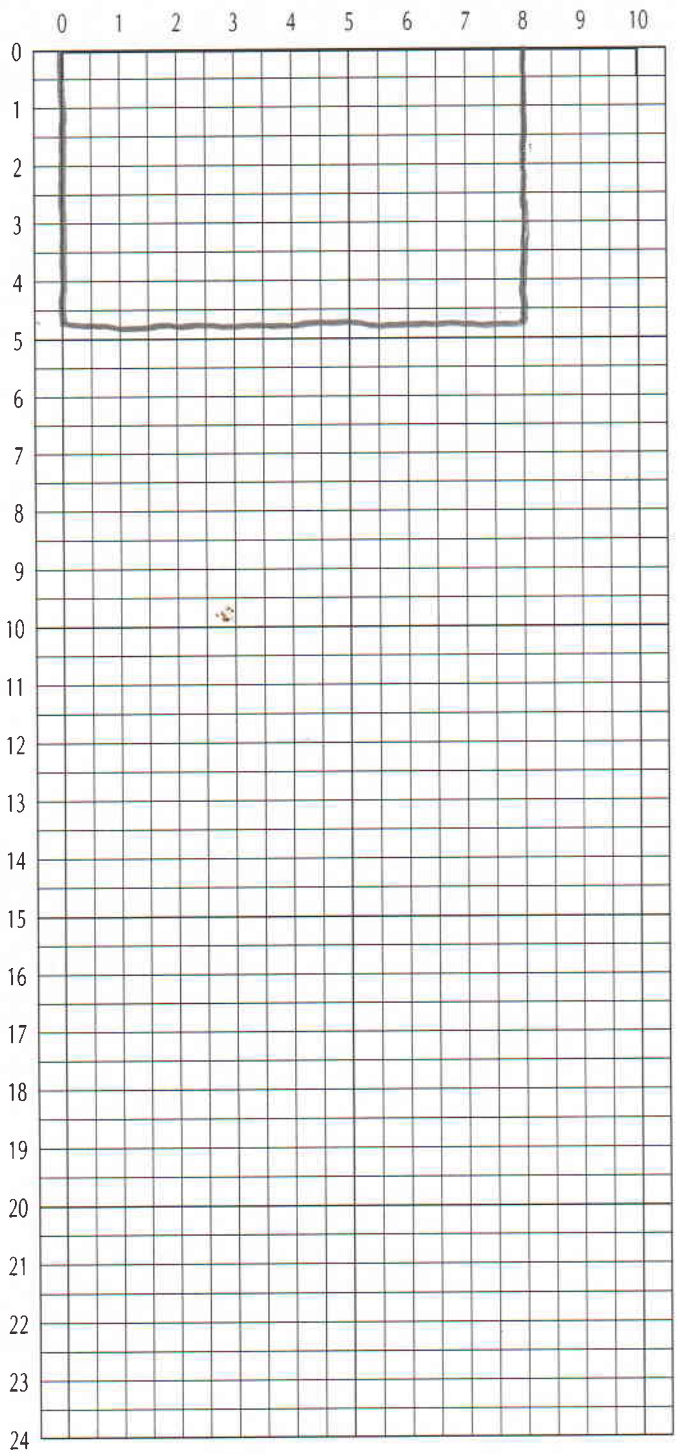
FEB. 2019 JOB NO. 6173 FIG. 9A

GRI
9750 SW Nimbus Avenue
Beaverton, OR 97008-7172
p| 503-641-3478 f| 503-644-8034

PAVEMENT CORE LOG

Saved: Yes No

Project:	Tualatin Garden Corner Curves
Highway:	SW Blake St
Key No.:	
Milepost:	
Location:	Northbound Lane
Core Length:	4.75"
Project No.:	6173
Date:	1-16-19
Core No.:	B-2
Logged By:	GDT



ODOT PAVEMENT CORE LOG TEMPLATE

DRILLED THROUGH PATCH: NO YES

DRILLED ON CRACK: NO YES (Trans. Long. Fat. Top Down Other)

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

NOTE DISTANCE FROM EDGE OF PAVEMENT AND DIRECTION:

EP EP

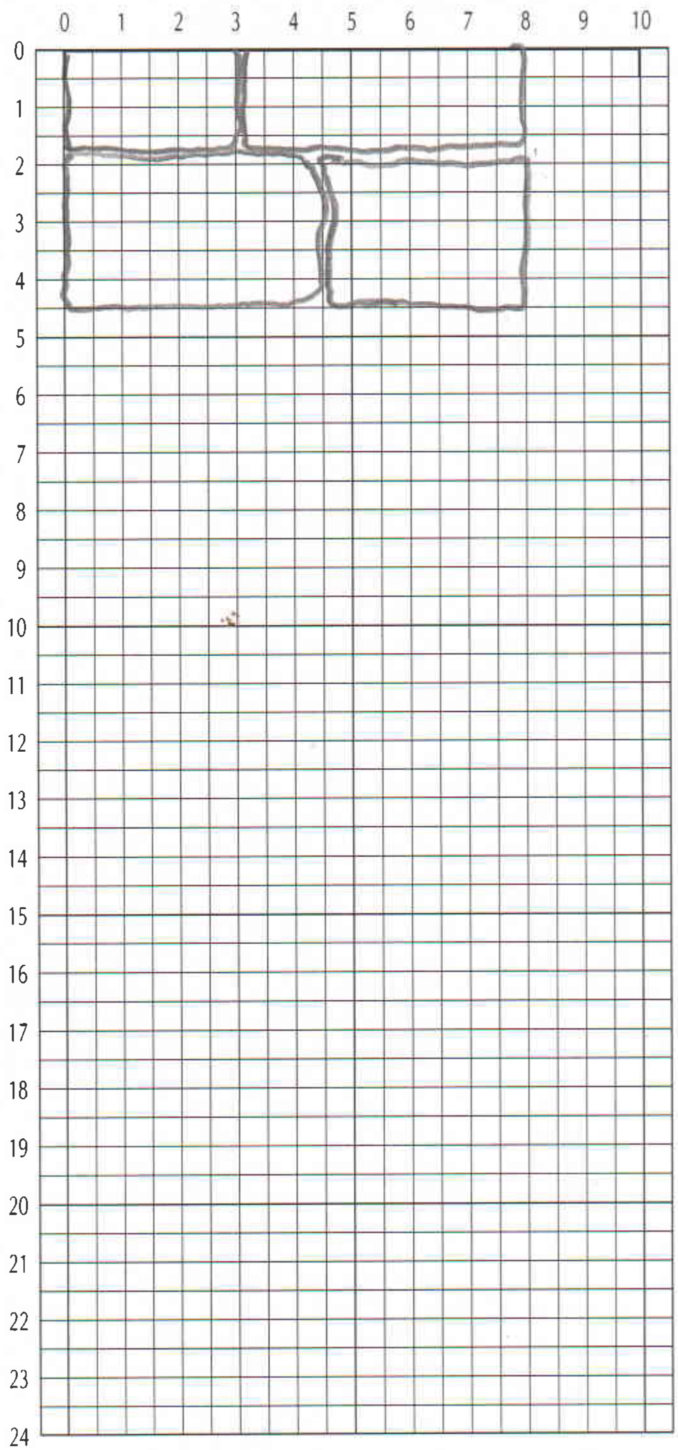
PAVEMENT BORING CORE LOGS

GRI
9750 SW Nimbus Avenue
Beaverton, OR 97008-7172
p| 503-641-3478 f| 503-644-8034

PAVEMENT CORE LOG

Saved: Yes No

Project: Tualatin Garden Corner Curves
Project No.: 6173
Highway: SW Blake St
Date: 1-16-19
Key No.:
Milepost:
Core No.: B-3
Location: Northbound Lane
Logged By. GDT
Core Length: 4.5"



ODOT PAVEMENT CORE LOG TEMPLATE

DRILLED THROUGH PATCH: NO YES

DRILLED ON CRACK: NO YES (Trans. Long. Fat. Top Down Other)

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

NOTE DISTANCE FROM EDGE OF PAVEMENT AND DIRECTION:

EP EP

GRI
9750 SW Nimbus Avenue
Beaverton, OR 97008-7172
p| 503-641-3478 f| 503-644-8034

PAVEMENT CORE LOG

Saved: Yes No

Project: Tualatin Garden Corner Curves

Project No.: 6173

Highway: SW 105th Ave

Date: 1-17-19

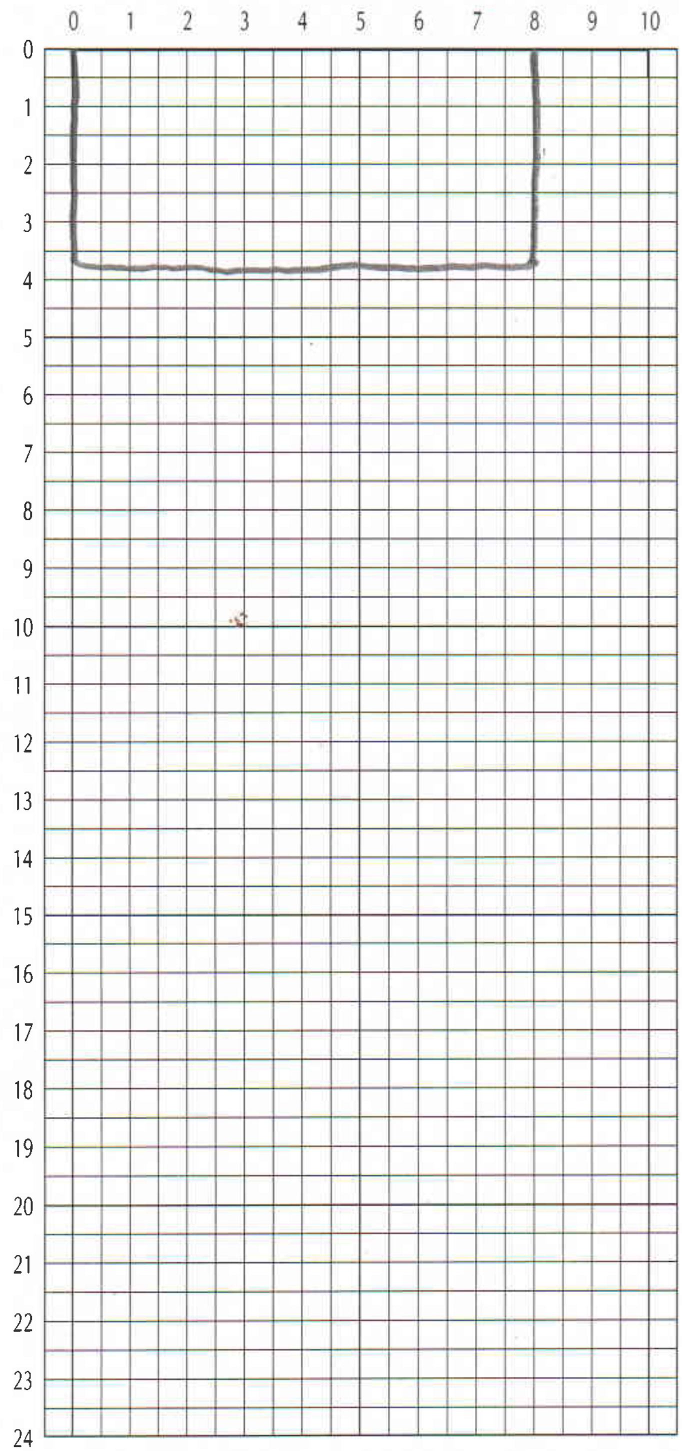
Key No.: ______ Milepost: ______

Core No.: B-4

Location: Southbound Lane

Logged By. GDT

Core Length: 3.75"



DRILLED THROUGH PATCH: NO YES

DRILLED ON CRACK: NO YES (Trans. Long. Fat. Top Down Other)

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

NOTE DISTANCE FROM EDGE OF PAVEMENT AND DIRECTION:

EP EP

ODOT PAVEMENT CORE LOG TEMPLATE

PAVEMENT BORING CORE LOGS

FEB. 2019 JOB NO. 6173 FIG. 12A

GRI
9750 SW Nimbus Avenue
Beaverton, OR 97008-7172
p| 503-641-3478 f| 503-644-8034

PAVEMENT CORE LOG

Saved: Yes No

Project: Tualatin Garden Corner Curves

Project No.: 6173

Highway: SW 108th Ave

Date: 1-17-19

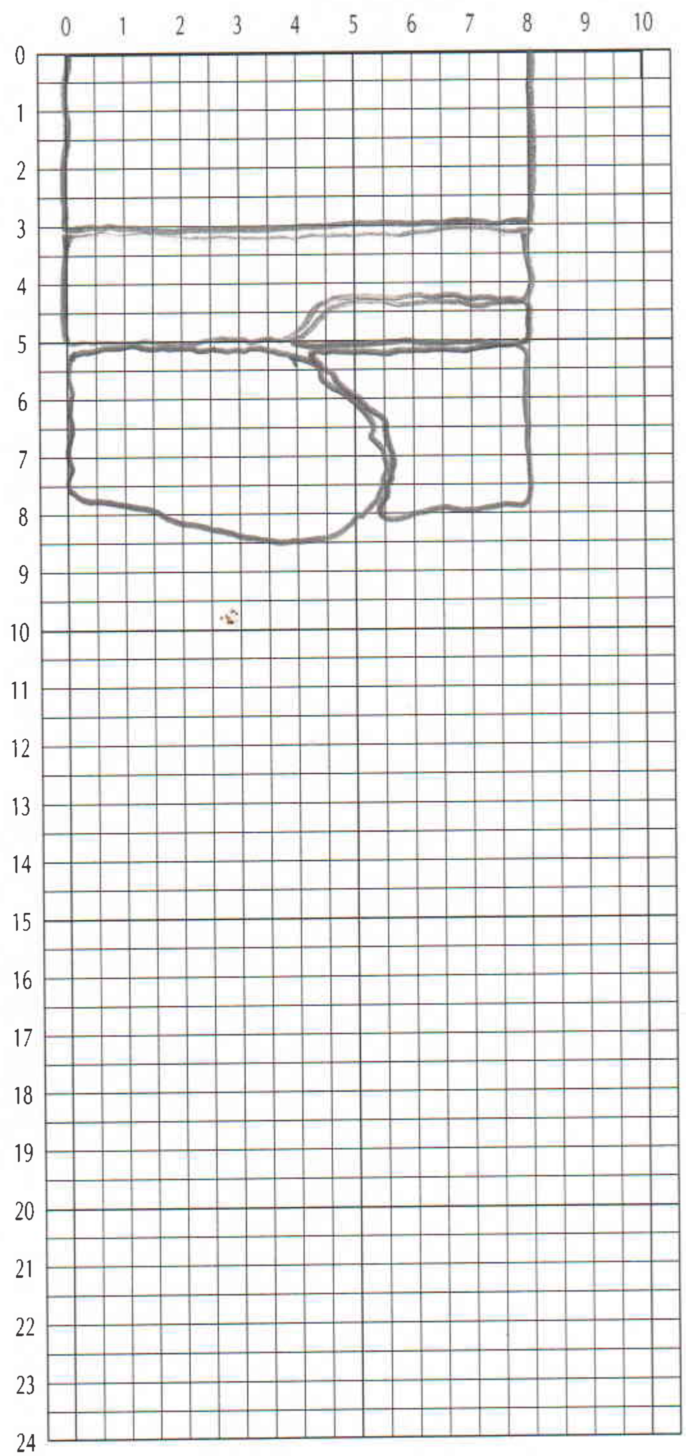
Key No.: ______ Milepost: ______

Core No.: B-5

Location: Northbound Lane

Logged By. GDT

Core Length: 8.5"



DRILLED THROUGH PATCH: NO YES

DRILLED ON CRACK: NO YES (Trans. Long. Fat. Top Down Other)

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good / Fair Poor

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

TYPE: Dense AC Open AC PCC CTB Oil Mat Other
CONDITION: Good Fair Poor

NOTE DISTANCE FROM EDGE OF PAVEMENT AND DIRECTION:

EP EP

ODOT PAVEMENT CORE LOG TEMPLATE

PAVEMENT BORING CORE LOGS

FEB. 2019 JOB NO. 6173 FIG. 13A

Core B-1 (SW Blake St., STA 15+87, FWD #65 SB)

B-1 (Pavement Core Sample, 6.00 in.)

GRI

PAVEMENT CORE PHOTOGRAPHS

FEB. 2019 JOB NO. 6173 FIG. 14A

Core B-2 (SW Blake St., STA 17+72, FWD #15 NB)

B-2 (Pavement Core Sample, 4.75 in.)

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PAVEMENT CORE PHOTOGRAPHS

FEB. 2019 JOB NO. 6173 FIG. 15A

Core B-3 (SW Blake St., STA 19 + 78, FWD #20 NB)

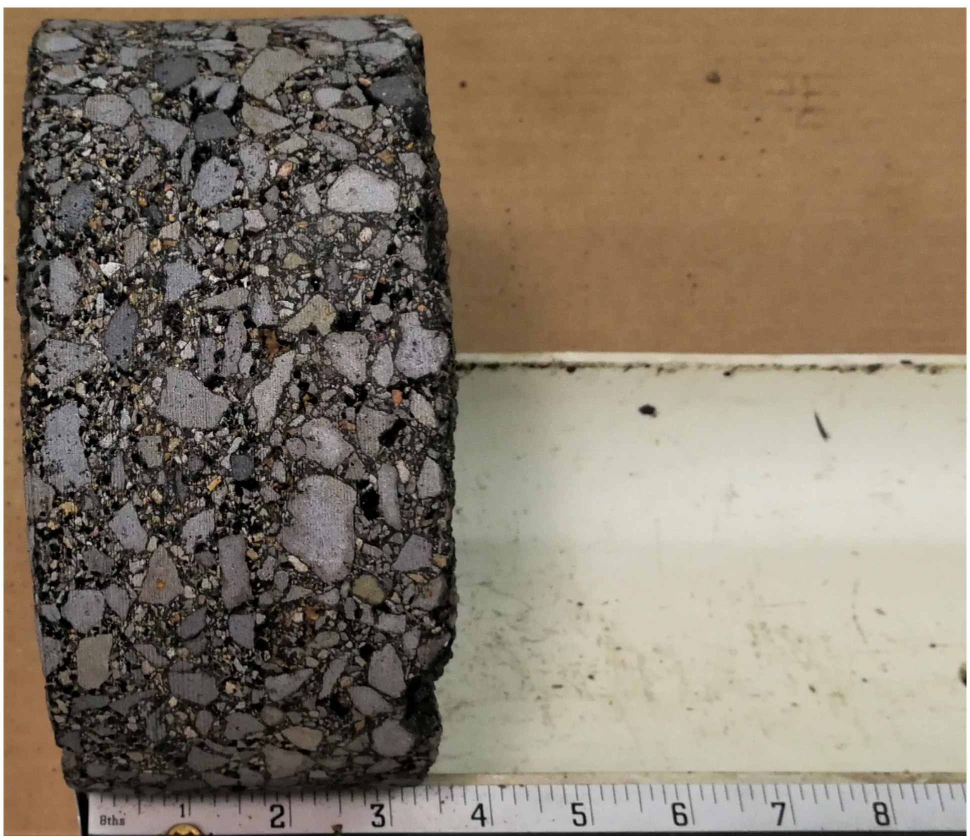
B-3 (Pavement Core Sample, 4.50 in.)

GRI

PAVEMENT CORE PHOTOGRAPHS

FEB. 2019 JOB NO. 6173 FIG. 16A

Core B-4 (SW 108th Ave., STA 12+89, FWD #5 NB)



B-4 (Pavement Core Sample, 3.75 in.)

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PAVEMENT CORE PHOTOGRAPHS

FEB. 2019 JOB NO. 6173 FIG. 17A

Core B-5 (SW 105th Ave., STA 23+86, FWD #29 NB)

B-5 (Pavement Core Sample, 8.50 in.)

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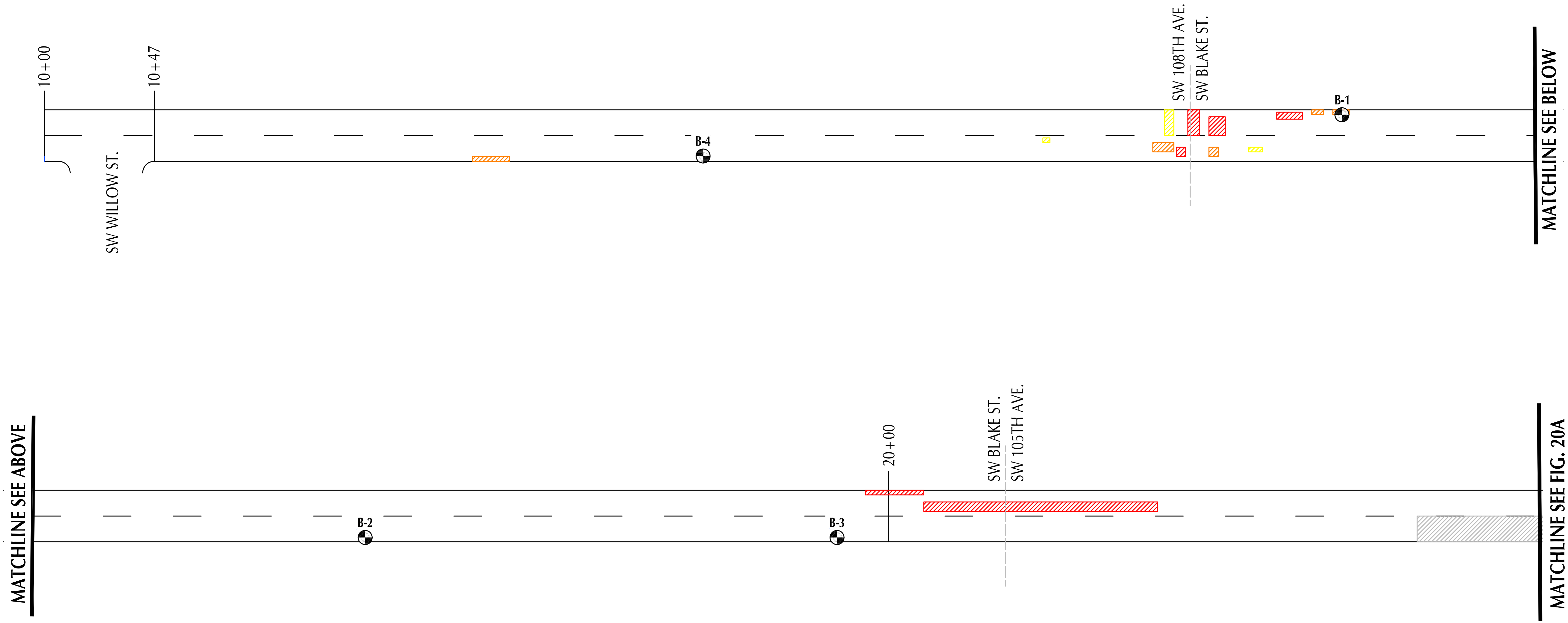
PAVEMENT CORE PHOTOGRAPHS

FEB. 2019 JOB NO. 6173 FIG. 18A



HIGH SEVERITY ALLIGATOR CRACKING

MEDIUM TO HIGH SEVERITY ALLIGATOR CRACKING

MEDIUM SEVERITY ALLIGATOR CRACKING

LOW SEVERITY SKIN PATCH

BORING COMPLETED BY GRI
(JANUARY 16-17, 2019)

North

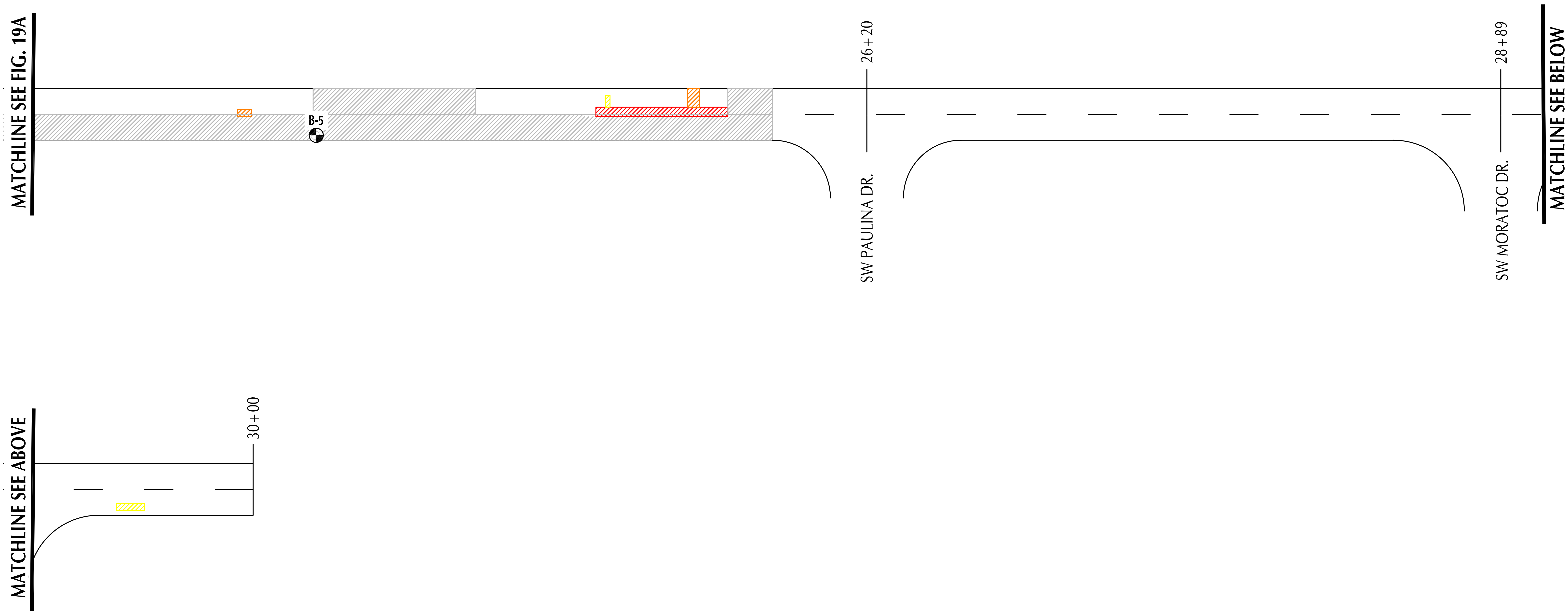
0 40 80 FT

GRI

CITY OF TUALATIN
GARDEN CORNER CURVES
TRANSPORTATION IMPROVEMENT PROJECT

PAVEMENT CONDITION MAP

FEB. 2019 | JOB NO. 6173 | FIG. 19A



HIGH SEVERITY ALLIGATOR CRACKING

MEDIUM TO HIGH SEVERITY ALLIGATOR CRACKING

MEDIUM SEVERITY ALLIGATOR CRACKING

LOW SEVERITY SKIN PATCH

BORING COMPLETED BY GRI
(JANUARY 16-17, 2019)

North

0 40 80 FT

GRI

CITY OF TUALATIN
GARDEN CORNER CURVES
TRANSPORTATION IMPROVEMENT PROJECT

PAVEMENT CONDITION MAP

FEB. 2019 | JOB NO. 6173 | FIG. 20A

APPENDIX B

Falling Weight Deflectometer Test Results

APPENDIX B

FALLING WEIGHT DEFLECTOMETER TEST RESULTS

GENERAL

Falling weight deflectometer (FWD) tests were conducted on January 18, 2019, using our KUAB 2m Model 150 FWD. FWD testing was conducted at approximately 100-ft intervals within the project limits in both the northbound and southbound travel lanes.

TEST PROCEDURES

The FWD test sequence consisted of an unrecorded seating impact load at nominally 6,000 lbs followed by two recorded impact loads at nominally 9,000 lbs. The FWD load is generated by a two-mass/two-buffer falling weight system that produces a nearly haversine-shaped load-pulse waveform. The buffer and weight combination used for these tests produces a load rise time of approximately 14 ms, with an equivalent haversine frequency of approximately 32 Hz. The load pulse was applied to the pavement surface through a 300-mm-diameter (5.91-in.-radius), four-part, segmented plate designed to apply uniform surface-pressure distribution despite irregularities in the pavement surface. Pavement deflections were measured by seismometers (absolute deflection sensors) positioned, with respect to the center of the load plate, at distances of 0, 12, 18, 24, 30, 36, 48, 60, and 72 in. The air temperature and pavement surface temperature (the latter measured by infrared thermometer) were recorded for each test.

TEST DATA

The average deflections from the two impact loads at nominally 9,000 lbs were linearly normalized to a 9-kip (9,000-lb) load basis and are tabulated in Table 1B and plotted on Figure 1B of this appendix. The measurement units for the test data are distance in feet, deflections in mil units (1 mil = 0.001 in.), load in pounds, sensor distance in inches, load plate radius in inches, and temperature in degrees Fahrenheit.

FWD CALIBRATION

The annual reference calibration for the FWD was accomplished on October 20, 2018, at the KUAB manufacturing facility in Savoy, Illinois.

Table 1B: FWD Normalized Deflection Test Data

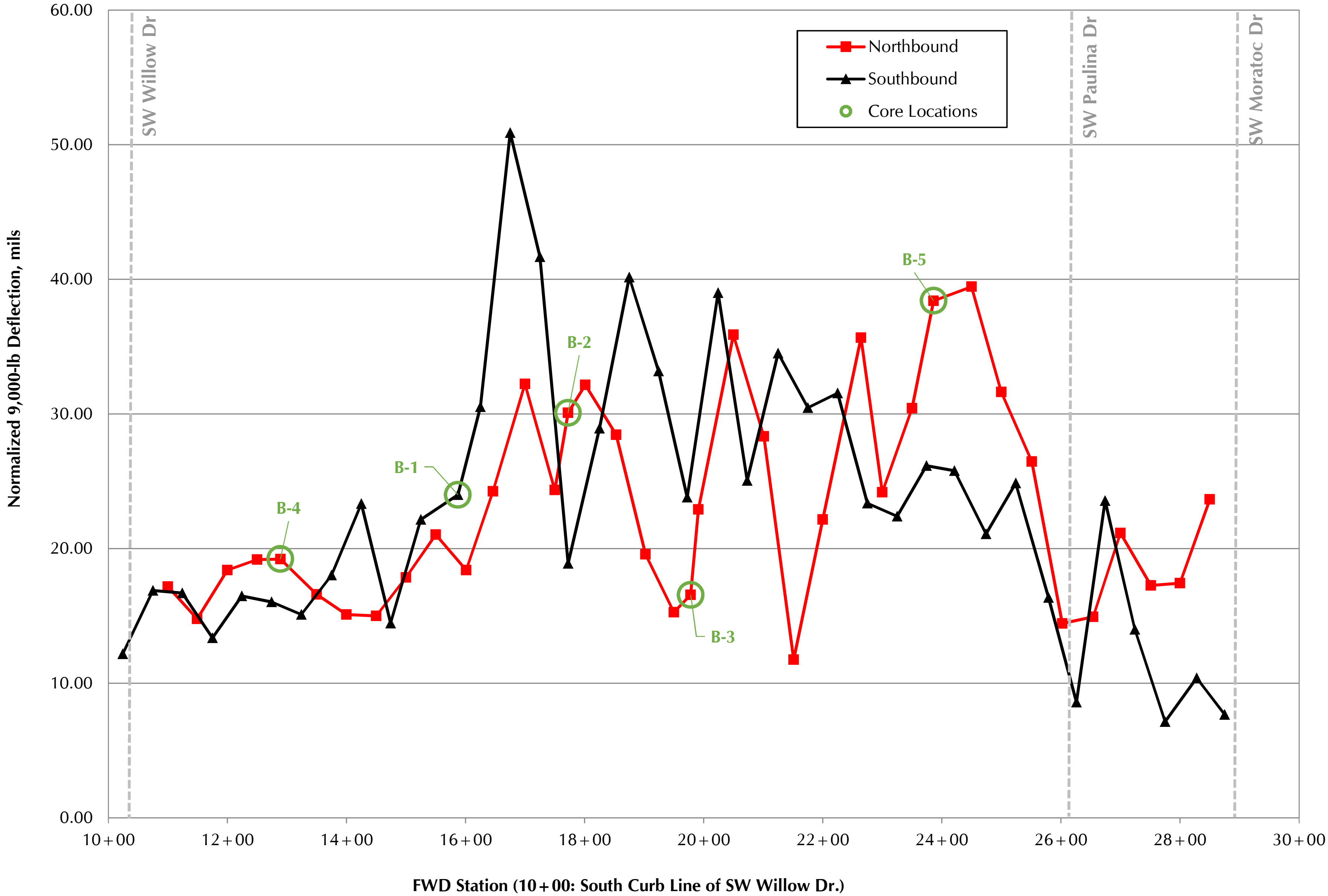
Test Section:	SW 108th-Blake-105th									
Start Point:	NC Willow = 10 + 00									
Test Date:	1/18/2019									
Test File:	6173-Tualatin GC.fwd									
Load Plate Radius, in:	5.91									
Sensor Distance, in:	0	8	12	18	24	36	48	60	72	

Deflections Normalized to 9000 lbf Basis

Test No.	Test Station	Test Line	Boring No.	D 0, mils	D 1, mils	D 2, mils	D 3, mils	D 4, mils	D 5, mils	D 6, mils	D 7, mils	D 8, mils	Surface Temp., °F	Time	Surface Modulus, Ksi	Comments
1	11 + 00	NB		17.17	14.35	12.37	9.42	7.40	4.46	2.97	2.18	1.79	42	9:19:29	42	nb
2	11 + 49	NB		14.77	12.47	10.89	8.31	6.60	3.91	2.52	1.83	1.54	42	9:20:22	49	
3	12 + 00	NB		18.39	15.90	14.10	11.07	8.80	5.18	3.15	2.10	1.64	42	9:21:17	40	
4	12 + 50	NB		19.18	15.87	13.62	10.41	8.04	4.68	2.99	2.08	1.65	43	9:22:05	38	
5	12 + 89	NB	B-4	19.22	16.71	14.89	11.81	9.74	6.25	4.20	2.92	2.25	43	9:23:02	38	B-4
6	13 + 50	NB		16.59	14.06	12.35	9.32	7.23	4.11	2.53	1.74	1.41	43	9:23:56	44	
7	14 + 00	NB		15.09	12.55	10.86	8.20	6.38	3.92	2.87	2.25	1.85	43	9:24:45	48	
8	14 + 50	NB		15.00	12.17	10.27	7.64	5.85	3.45	2.40	1.89	1.64	43	9:25:32	48	
9	15 + 00	NB		17.85	15.66	13.96	11.14	8.75	4.97	3.04	2.12	1.75	43	9:26:31	41	
10	15 + 50	NB		21.03	16.97	14.58	11.21	8.64	4.90	2.86	2.01	1.64	43	9:27:38	35	
11	16 + 01	NB		18.40	15.39	13.31	10.18	7.77	4.40	2.79	2.04	1.71	43	9:28:33	40	1559 = PC
12	16 + 46	NB		24.25	19.71	16.59	11.83	8.78	4.53	2.72	1.96	1.69	43	9:29:27	30	
13	17 + 00	NB		32.23	26.43	22.55	16.24	11.95	6.02	3.47	2.49	2.05	43	9:30:16	23	
14	17 + 50	NB		24.36	19.58	16.60	12.34	9.33	4.93	3.01	2.15	1.77	43	9:31:04	30	
15	17 + 72	NB	B-2	30.08	23.93	19.97	14.24	10.46	5.39	3.29	2.35	1.89	43	9:31:56	24	B-2
16	18 + 01	NB		32.16	26.65	22.37	16.34	12.18	6.32	3.75	2.60	2.10	43	9:32:47	23	
17	18 + 53	NB		28.45	22.75	19.00	13.32	9.63	4.95	3.06	2.20	1.82	43	9:33:35	26	
18	19 + 02	NB		19.57	16.56	14.46	11.17	8.89	5.38	3.52	2.43	1.96	43	9:34:27	37	
19	19 + 50	NB		15.27	13.61	12.37	10.47	8.88	6.02	4.20	2.95	2.28	43	9:35:49	48	
20	19 + 78	NB	B-3	16.56	14.42	35.38	13.77	8.79	3.69	4.36	3.36	2.59	43	9:36:48	44	At B-3 - Deflection is not decreasing; 1975 = pc
21	19 + 91	NB		22.90	20.22	18.19	15.11	12.55	8.23	5.51	3.73	2.80	43	9:38:28	32	Redo B-3
22	20 + 50	NB		35.88	27.91	22.77	15.88	11.55	6.08	3.95	2.93	2.37	43	9:39:57	20	
23	21 + 01	NB		28.33	23.96	20.18	14.90	11.29	6.15	3.91	2.86	2.38	43	9:40:49	26	
24	21 + 51	NB		11.75	8.94	7.16	4.92	3.54	2.02	1.42	1.10	0.96	43	9:41:39	62	
25	22 + 00	NB		22.15	15.95	11.77	7.41	5.16	2.93	2.00	1.53	1.27	43	9:42:30	33	
26	22 + 64	NB		35.67	29.76	25.59	19.29	14.62	8.50	5.71	4.36	3.61	43	9:44:45	20	
27	23 + 00	NB		24.19	20.15	17.48	13.51	10.59	6.39	4.44	3.41	2.87	43	9:45:33	30	
28	23 + 50	NB		30.42	25.95	22.37	16.89	13.04	7.50	5.02	3.67	3.04	43	9:46:35	24	
29	23 + 86	NB	B-5	38.41	30.81	25.79	19.21	14.52	8.32	5.61	4.18	3.33	44	9:47:52	19	B-3
30	24 + 50	NB		39.45	31.36	26.70	20.15	15.17	8.61	5.93	4.55	3.78	44	9:49:21	18	
31	25 + 00	NB		31.63	25.63	21.86	16.45	12.36	7.40	5.15	3.89	3.17	44	9:50:10	23	
32	25 + 51	NB		26.46	21.87	18.86	14.31	11.21	6.57	4.75	3.68	3.03	44	9:51:02	27	
33	26 + 02	NB		14.44	13.39	12.52	10.95	9.35	6.56	4.75	3.50	2.75	44	9:52:23	50	2576 = PC 2620 = CL PAULINA
34	26 + 54	NB		14.92	13.10	11.73	9.45	7.62	4.99	3.46	2.51	2.04	44	9:53:58	49	
35	27 + 00	NB		21.15	18.10	15.91	12.56	10.21	6.52	4.40	3.24	2.54	44	9:54:53	34	
36	27 + 51	NB		17.26	14.23	12.39	9.87	7.81	5.11	3.64	2.82	2.25	44	9:55:49	42	
37	28 + 00	NB		17.43	15.03	13.30	10.98	9.07	6.29	4.76	3.68	2.96	45	9:56:44	42	
38	28 + 50	NB		23.66	20.31	17.73	13.79	10.71	6.62	4.74	3.66	3.03	45	9:57:37	31	END NB; 2889 = CL MOROTOC
39	28 + 75	SB		7.66	6.85	6.38	5.42	4.75	3.49	2.63	2.07	1.69	45	10:02:24	95	SB
40	28 + 28	SB		10.39	9.09	8.25	6.67	5.59	3.86	2.83	2.25	1.91	45	10:03:17	70	
41	27 + 75	SB		7.14	6.88	6.12	5.47	4.72	3.61	2.80	2.15	1.71	45	10:04:05	102	
42	27 + 24	SB		14.00	12.37	11.15	9.33	7.78	5.15	3.59	2.51	1.98	45	10:05:03	52	
43	26 + 74	SB		23.55	19.25	16.67	12.32	9.40	5.53	3.66	2.77	2.27	45	10:05:54	31	
44	26 + 26	SB		8.58	7.52	6.95	5.89	5.12	3.74	2.77	2.06	1.63	45	10:06:50	85	

Table 1B: FWD Normalized Deflection Test Data

				Deflections Normalized to 9000 lbf Basis												
Test No.	Test Station	Test Line	Boring No.	D 0, mils	D 1, mils	D 2, mils	D 3, mils	D 4, mils	D 5, mils	D 6, mils	D 7, mils	D 8, mils	Surface Temp., °F	Time	Surface Modulus, Ksi	Comments
45	25+79	SB		16.36	12.26	10.15	7.33	5.98	3.92	2.95	2.37	2.05	45	10:07:43	44	
46	25+24	SB		24.87	18.81	14.88	10.27	7.58	5.22	4.59	3.80	3.27	45	10:08:53	29	
47	24+74	SB		21.08	16.95	14.27	10.84	8.65	5.89	4.85	4.03	3.48	45	10:09:44	34	
48	24+21	SB		25.78	21.79	19.19	15.30	11.96	7.29	4.87	3.73	3.20	45	10:10:37	28	TEST WAS ON A SKIN PATCH
49	23+74	SB		26.16	21.60	18.51	14.04	10.95	6.93	4.77	3.56	2.85	45	10:11:44	28	
50	23+25	SB		22.39	18.42	15.88	12.20	9.47	5.98	4.32	3.20	2.59	45	10:12:40	32	
51	22+75	SB		23.37	19.18	16.26	12.50	9.88	6.30	4.52	3.43	2.83	45	10:13:37	31	
52	22+25	SB		31.55	25.24	20.75	15.18	11.10	6.27	4.31	3.07	2.43	44	10:14:34	23	
53	21+75	SB		30.46	25.21	21.28	16.14	12.56	7.59	5.44	4.05	3.21	44	10:15:31	24	
54	21+25	SB		34.50	29.14	25.71	20.14	15.82	9.34	6.02	4.07	3.10	45	10:16:22	21	
55	20+73	SB		25.05	20.43	16.97	12.28	8.86	4.82	3.23	2.46	2.03	45	10:17:14	29	
56	20+24	SB		39.00	32.88	28.73	22.15	17.61	10.80	7.41	5.26	4.16	45	10:18:07	19	
57	19+72	SB		23.81	19.63	16.51	12.12	8.99	5.10	3.50	2.74	2.32	45	10:19:05	31	
58	19+24	SB		33.18	26.68	22.39	15.88	11.33	5.59	3.31	2.36	1.98	45	10:20:01	22	
59	18+75	SB		40.15	31.53	25.36	16.81	11.37	5.41	3.37	2.43	2.03	46	10:20:49	18	
60	18+25	SB		28.92	24.42	21.16	16.09	12.32	6.85	4.07	2.74	2.17	46	10:21:43	25	
61	17+72	SB		18.89	15.66	13.29	9.88	7.37	4.09	2.59	1.89	1.59	45	10:22:31	38	
62	17+25	SB		41.67	30.51	24.29	16.68	11.47	5.39	3.33	2.47	2.10	45	10:23:19	17	
63	16+75	SB		50.88	37.69	29.72	19.81	13.12	5.41	3.02	2.21	1.88	45	10:24:06	14	
64	16+25	SB		30.53	24.78	20.71	14.94	10.84	5.46	3.14	2.20	1.84	45	10:24:59	24	
65	15+87	SB	B-1	23.99	20.02	17.01	13.04	10.12	5.86	3.57	2.40	1.89	45	10:26:01	30	B-1, 1589=PC
66	15+25	SB		22.15	16.40	13.29	9.42	7.00	3.90	2.53	1.94	1.65	47	10:27:10	33	
67	14+74	SB		14.46	12.71	11.32	9.18	7.40	4.67	3.06	2.15	1.74	47	10:28:16	50	
68	14+25	SB		23.33	19.18	16.12	11.67	8.54	4.36	2.51	1.96	1.66	48	10:29:25	31	
69	13+75	SB		18.03	15.39	13.40	10.57	8.20	3.98	2.92	2.19	1.77	48	10:30:18	40	
70	13+24	SB		15.09	12.97	11.31	8.93	7.04	4.09	2.48	1.71	1.39	47	10:31:07	48	
71	12+74	SB		16.04	14.15	12.16	9.44	7.50	4.76	3.26	2.39	1.90	47	10:31:57	45	
72	12+24	SB		16.47	12.69	10.31	7.40	5.55	3.26	2.28	1.74	1.44	47	10:32:48	44	
73	11+75	SB		13.36	10.53	8.79	6.68	5.10	3.05	1.95	1.51	1.26	47	10:33:55	54	
74	11+24	SB		16.69	11.91	9.59	6.75	4.80	2.63	2.15	1.74	1.46	47	10:34:53	44	
75	10+75	SB		16.89	13.75	11.84	9.26	7.23	4.29	2.77	1.98	1.57	48	10:35:51	43	
76	10+24	SB		12.18	10.35	8.94	6.93	5.46	3.40	2.27	1.70	1.39	48	10:36:45	60	1047=NC WILLOW END SB



GRI

9-KIP NORMALIZED DEFLECTIONS

FEB. 2019 JOB NO. 6173 FIG. 1B

APPENDIX C

Pavement Design Calculations

APPENDIX C

PAVEMENT DESIGN CALCULATIONS

BACKCALCULATION ANALYSIS

Overview of Backcalculation Analysis Procedure

A backcalculation analysis was accomplished using the PAVBACK backcalculation analysis program, which is an iterative, elastic layered analysis procedure. Deflections are calculated using the Boussinesq-Odemark equivalent thickness procedure (Ullidtz, 1998). PAVBACK solutions were validated by comparing calculated and measured values of asphalt tensile strain and subgrade compressive strain/stress using data from tests on instrumented pavement (test data published in Ullidtz, ASTM STP 1375, 2000), where the calculated values were based on the moduli backcalculated by PAVBACK from the deflections measured on the instrumented pavement. The calculated strains and stress were found to agree nearly exactly with the measured values (within $\pm$10% of the measured values).

The FWD deflection data were analyzed using the asphalt concrete (AC) and aggregate base (AB) thicknesses measured at the core explorations in each direction to backcalculate the equivalent elastic moduli of the AC, AB, and subgrade soils at the FWD test locations. The backcalculation analysis results are shown along with the overlay/inlay analysis results in Table 1C, which includes the equivalent elastic moduli of the pavement layers and subgrade soil and the effective structural number (*SNeff*) of the pavement. The AC moduli reported in Table 1C were normalized to 68-°F temperature and 10-Hz loading rate conditions.

Backcalculation of Subgrade Modulus

PAVBACK analyzes the change in deflection with distance from the load plate in the outer portion of the deflection basin to check for the effect of an apparent rigid layer at shallow depth. If it appears that a rigid layer may be present, the depth of the apparent layer below the subgrade is calculated and the subgrade modeled as a linear-elastic layer with a depth equal to the depth to the apparent rigid layer. The apparent rigid layer may represent the effect of bedrock (if actually present within the upper subgrade), the singular or combined effects of a shallow water table, layering in the subgrade, or the finite time duration of the FWD load pulse. If an apparent rigid layer is not detected, the subgrade is modeled as an infinitely deep, non-linear, stress-softening, elastic material using the following constitutive relationship:

$$M_r = k_1 \sigma_1^{k_2} \quad ; k_2 \leq 0 \qquad (1)$$

in which:

M_r	=	Subgrade resilient modulus
σ_1	=	Principal (deviator) stress on subgrade surface
k_1	=	Constitutive model parameter
k_2	=	Stress exponent (less than or equal to 0).

This backcalculation procedure is superficially similar to the backcalculation procedure recommended in the 1993 American Association of State Highway and Transportation Officials (AASHTO) Guide for Design of Pavement Structures (AASHTO Guide) but with an important difference. The resilient moduli computed from outer sensor deflections using the AASHTO Guide backcalculation procedure are surface moduli based on the assumption that the subgrade is linear elastic and infinitely deep. If either of these assumptions is incorrect, the modulus calculated by the AASHTO Guide surface-modulus formula will be significantly higher than the M_r of the subgrade soil at a deviator stress of 6 psi. However, the effect of non-linearity or finite subgrade depth is explicitly accounted for in the PAVBACK backcalculation program, allowing direct calculation of subgrade moduli corresponding to 6-psi deviator stress. This eliminates the need to apply an arbitrary correction factor to the backcalculated moduli.

Backcalculation of Layer Elastic Moduli

The pavement structure was modeled as a multilayered elastic system to backcalculate the equivalent elastic moduli (as applicable) of the AC, crushed-rock base (CRB), and subgrade soils.

The pavement layer thicknesses used in the backcalculation analysis were based on our core explorations within each analysis unit.

The multilayered backcalculation analysis uses mathematical optimization techniques to calculate the equivalent elastic modulus values of the pavement layers and subgrade soil to minimize the difference between deflections calculated according to the analysis model and the field-measured deflections. This analysis is conducted by an iterative approach beginning with an assumed set of layer moduli. Pavement-surface deflections are calculated according to elastic-layer theory using these initial layer moduli. The computed deflections are compared with the measured deflections and the initial layer moduli are adjusted to reduce the differences between the measured and calculated deflections. The adjusted moduli are then used to start the next analysis iteration. The iteration process continues until the computed and measured deflections match within a specified tolerance or until the adjustment to the solution values is less than a specified tolerance. The "goodness of fit" between the measured and computed deflections is measured by the root mean squared relative error (RMSE), which is calculated using the percent difference between the measured and calculated deflections relative to the measured deflection and roughly a measure of the relative percent error per deflection sensor.

Backcalculation Analysis Results

The multilayer backcalculation analysis results are shown in Table 1C of this appendix. The AC moduli have been normalized to 68-°F temperature and 10-Hz loading rate conditions using the Asphalt Institute's predictive equation. The backcalculated moduli of the AC and AB layers were used to calculate their effective layer coefficients and the corresponding effective structural number of the pavement section, *SNeff*, using Equation PP.17 from the 1986 AASHTO Guide for Design of Pavement Structures – Volume 2 after adjusting the moduli of the AC to 68-°F temperature and 10-Hz loading rate conditions. The backcalculated subgrade M_r values were normalized if necessary (i.e., in cases where the backcalculated value for k_2 is less than 0) to a principal stress of 6 psi (based on the backcalculated values for k_1 and k_2) to correspond to the AASHTO design procedure for determination of subgrade resilient modulus.

Rehabilitation Analysis

The overlay thicknesses and depths of inlay required for strengthening of the pavement were calculated at each of the FWD test locations based on the maximum overlay thickness or inlay depth for the design traffic loading with respect to the backcalculated moduli of the existing AB and the subgrade soil. The inlay analysis considered the structural effect of removing existing pavement materials within the inlay depth by reducing the effective structural number of the pavement to account for the removed materials based on their backcalculated layer coefficients and thicknesses. A discussion of the overlay analysis is provided below. The overlay and inlay analysis results for the test locations on the Garden Curves travel lanes for a 20-year design period are shown in Table 1C. This table also includes the structural characteristics of the existing pavement as determined from the backcalculation analysis.

Pavement Overlay and Inlay Analysis. Our overlay analysis software, PAVCALC, was used to calculate the overlay thickness or inlay depth required at each FWD test point based on the structural deficiency of the existing pavement structure with respect to the subgrade and base layer in order to determine the controlling overlay thickness or inlay depth. The overlay thicknesses and inlay depths were calculated using the following AASHTO design equation (1993 AASHTO Guide):

$$D_{ol} = \frac{\Delta SN}{a_{ol}}$$

where:

D_{ol}	=	Overlay thickness or inlay depth, in.
ΔSN	=	Structural deficiency
	=	SN_f - $SNeff$
SN_f	=	Structural number required for the design traffic loading
$SNeff$	=	Backcalculated effective structural number of the existing pavement, which for Inlay is adjusted for the effect of material removed by inlay
a_{ol}	=	coefficient for the AC overlay (0.42)

The design parameters presented in Table 3 were used to compute the structural number required for the design traffic loading, SN_f, using the AASHTO design equation (Guide Part III, Section 3.1.1) to determine the structural deficiencies above the subgrade and base layer. For analysis of structural deficiency above the subgrade, SN_f is calculated based on the backcalculated subgrade modulus and $SNeff$ is the backcalculated effective structural number of the entire pavement structure above the subgrade. For analysis of structural deficiency above the base layer, SN_f is calculated based on the backcalculated modulus of the base layer and $SNeff$ is the backcalculated effective structural number of the AC above the base layer. The controlling overlay thickness or inlay depth is set equal to the maximum, as required by these two procedures for calculating structural deficiency.

For inlay analysis, the program adjusts $SNeff$ to take into account the thickness of removed pavement materials by subtracting the effective structural number of the removed materials. The effective structural number of the removed materials within the inlay depth is computed based on the thicknesses and backcalculated layer coefficients of the materials within the inlay depth.

Pavement Design Analysis for Reconstruction. The pavement design worksheets for new pavement construction within the Garden Corner Curves project limits are shown in Tables 2C and 3C.

References

Ullidtz, P., 1998, Modeling flexible pavement response and performance, Polyteknisk Forlag.

Ullidtz, P., 2000, Will nonlinear backcalculation help?, Nondestructive testing of pavements and backcalculation of moduli: ASTM STP 1375, Tayabji, D., and Lukanen, E. O., Eds., American Society for Testing and Materials, v. 3, pp. 14-22.

Table 1C: BACKCALCULATION AND OVERLAY ANALYSIS RESULTS

Tualatin Garden Corner Curves
Based on FWD Testing Conducted: 1/18/19
Distance Reference: 10+47 = SW Willow St.

Overlay Analysis Parameters:

Traffic Loading, ESAL's	20-yr Estimate:	50,000
Design Reliability, %		80
Standard Deviation		0.50
Initial Serviceability		4.2
Terminal Serviceability		2.5

															Overlay/Inlay Analysis						
						Structural Characteristics					Calculated Structural Number				Overlay Thickness above Existing Pavement			Overlay Thickness above 2-inch Milled Surface			
FWD Test #	Test Station	Dir	Street	Analysis Unit	D0, mils	AC Thickness, inches	AB Thickness, inches	AC Modulus @ 68 ^{o}F & 10 Hz, psi	AB Modulus, psi	Subgrade M_r at 6 psi deviator stress, psi	SNeff of Existing Pavement Above SG	Required SN above SG	SNeff of Existing Pavement Above AB	Required SN above AB	Based on Analysis above the Subgrade, inches	Based on Analysis above the Base, inches	Controlling Overlay (maximum of requirements above the SG and AB), inches	Based on Analysis above the Subgrade, inches	Based on Analysis above the Base, inches	Controlling Inlay Depth (maximum of requirements above the SG and AB), inches	Cores
1	11+00	NB	SW 108th Ave	1	17.17	5.50	8.00	181,462	23,099	7,595	2.20	1.96	1.31	1.48	zero	0.42	0.42	NC	1.55	1.55	
2	11+49	NB	SW 108th Ave	1	14.77	5.50	8.00	239,248	26,651	7,414	2.26	1.97	1.36	1.48	zero	0.27	0.27	NC	1.45	1.45	
3	12+00	NB	SW 108th Ave	Digout	18.39	5.50	8.00	246,795	24,422	3,614	2.25	2.63	1.36	1.48	0.92	0.31	0.92	2.09	1.48	2.09	
4	12+50	NB	SW 108th Ave	1	19.18	5.50	8.00	147,377	30,350	5,136	2.22	2.31	1.24	1.29	0.20	0.14	0.20	1.27	1.21	1.27	
5	12+89	NB	SW 108th Ave	1	19.22	3.75	11.25	553,817	27,071	3,662	2.90	2.63	1.41	1.18	zero	zero	zero	NC	NC	NC	B-4
6	13+50	NB	SW 108th Ave	1	16.59	5.50	8.00	242,203	21,155	5,579	2.29	2.21	1.40	1.49	zero	0.21	0.21	NC	1.42	1.42	
7	14+00	NB	SW 108th Ave	1	15.09	4.50	8.00	359,281	18,698	12,254	2.13	1.61	1.24	1.48	zero	0.58	0.58	NC	1.89	1.89	
8	14+50	NB	SW 108th Ave	1	15.00	5.00	8.00	232,160	21,852	11,715	2.16	1.64	1.23	1.41	zero	0.43	0.43	NC	1.61	1.61	
9	15+00	NB	SW Blake St	Digout	17.85	5.50	8.00	346,295	11,176	4,489	2.26	2.38	1.51	1.85	0.29	0.79	0.79	1.60	2.10	2.10	
10	15+50	NB	SW Blake St	Digout	21.03	5.50	8.00	115,899	40,289	3,603	2.24	2.63	1.13	1.10	0.93	zero	0.93	1.91	NC	1.91	
11	16+01	NB	SW Blake St	1	18.40	5.00	8.00	252,400	19,335	5,955	2.15	2.16	1.24	1.45	0.02	0.50	0.50	1.20	1.68	1.68	
12	16+46	NB	SW Blake St	2	24.25	4.50	8.00	197,107	17,639	4,422	1.89	2.42	1.04	1.57	1.26	1.27	1.27	2.36	2.37	2.37	
13	17+00	NB	SW Blake St	2	32.23	4.50	8.00	174,571	12,504	2,738	1.85	2.92	1.04	1.66	2.53	1.49	2.53	3.63	2.59	3.63	
14	17+50	NB	SW Blake St	2	24.36	4.50	8.00	182,886	21,030	4,375	1.93	2.43	0.99	1.38	1.19	0.92	1.19	2.24	1.97	2.24	
15	17+72	NB	SW Blake St	2	30.08	4.75	6.75	132,927	13,045	4,498	1.93	2.43	1.17	1.46	1.20	0.70	1.20	2.37	1.87	2.37	B-2
16	18+01	NB	SW Blake St	2	32.16	4.50	8.00	165,775	12,992	2,987	1.84	2.81	1.06	1.78	2.33	1.70	2.33	3.45	2.82	3.45	
17	18+53	NB	SW Blake St	2	28.45	4.00	8.00	230,015	11,416	4,911	1.69	2.33	0.91	1.74	1.51	1.97	1.97	2.60	3.05	3.05	
18	19+02	NB	SW Blake St	2	19.57	5.50	8.00	157,454	34,822	4,318	2.20	2.48	1.24	1.35	0.67	0.27	0.67	1.74	1.34	1.74	
19	19+50	NB	SW Blake St	2	15.27	6.50	8.00	247,620	38,187	4,022	2.65	2.53	1.70	1.37	zero	zero	zero	NC	NC	NC	
21	19+91	NB	SW Blake St	2	22.90	4.50	11.50	367,677	20,957	2,289	2.39	3.13	1.11	1.49	1.78	0.90	1.78	2.96	2.08	2.96	B-3
22	20+50	NB	SW Blake St	2	35.88	3.50	8.00	193,304	12,160	4,308	1.53	2.45	0.75	1.74	2.19	2.36	2.36	3.21	3.38	3.38	
23	21+01	NB	SW 105th Ave	2	28.33	4.00	8.00	300,981	9,124	4,897	1.69	2.30	1.03	2.14	1.44	2.62	2.62	2.67	3.85	3.85	
24	21+51	NB	SW 105th Ave	2	11.75	5.00	8.00	199,823	30,791	17,996	2.20	1.34	1.22	1.31	zero	0.22	0.22	NC	1.38	1.38	
25	22+00	NB	SW 105th Ave	2	22.15	3.50	8.00	62,851	37,679	11,537	1.71	1.61	0.87	1.61	zero	1.76	1.76	NC	2.95	2.95	
26	22+64	NB	SW 105th Ave	2	35.67	3.50	8.00	339,252	7,444	4,509	1.56	2.42	0.86	1.96	2.03	2.64	2.64	3.20	3.80	3.80	
27	23+00	NB	SW 105th Ave	2	24.19	4.00	8.00	332,535	12,430	6,909	1.86	2.06	0.99	1.55	0.50	1.31	1.31	1.68	2.50	2.50	
28	23+50	NB	SW 105th Ave	2	30.42	4.00	8.00	308,774	7,673	4,943	1.79	2.33	1.10	2.02	1.28	2.19	2.19	2.59	3.50	3.50	
29	23+86	NB	SW 105th Ave	Unreasonable	38.41	8.50	6.00	--	--	--	--	--	--	--	--	--	--	--	--	--	B-5
30	24+50	NB	SW 105th Ave	2	39.45	3.50	8.00	221,820	10,444	3,980	1.55	2.55	0.72	1.62	2.38	2.14	2.38	3.36	3.13	3.36	
31	25+00	NB	SW 105th Ave	2	31.63	3.50	8.00	307,423	10,212	5,800	1.61	2.18	0.83	1.73	1.36	2.16	2.16	2.49	3.28	3.28	
32	25+51	NB	SW 105th Ave	3	26.46	4.00	8.00	288,133	12,419	5,960	1.81	2.19	0.96	1.57	0.90	1.45	1.45	2.04	2.59	2.59	
33	26+02	NB	SW 105th Ave	3	14.44	7.00	8.00	356,934	13,646	5,250	2.74	2.18	2.18	2.55	zero	0.89	0.89	NC	2.37	2.37	
34	26+54	NB	SW 105th Ave	3	14.92	6.00	8.00	241,480	29,074	6,451	2.43	2.11	1.57	1.56	zero	zero	zero	NC	NC	NC	
35	27+00	NB	SW 105th Ave	3	21.15	5.00	8.00	216,785	21,858	5,622	2.02	2.21	1.17	1.57	0.44	0.94	0.94	1.56	2.06	2.06	
36	27+51	NB	SW 105th Ave	3	17.26	5.00	8.00	207,597	26,688	9,572	2.11	1.79	1.13	1.32	zero	0.44	0.44	NC	1.52	1.52	
37	28+00	NB	SW 105th Ave	3	17.43	6.50	8.00	134,524	36,074	7,146	2.48	2.04	1.50	1.31	zero	zero	zero	NC	NC	NC	
38	28+50	NB	SW 105th Ave	3	23.66	4.00	8.00	420,147	9,280	7,482	1.82	1.96	1.11	1.94	0.33	1.98	1.98	1.65	3.30	3.30	
39	28+75	SB	SW 105th Ave	3	7.66	8.50	8.00	310,475	32,999	16,231	3.26	1.37	2.58	2.05	zero	zero	zero	NC	NC	NC	
40	28+28	SB	SW 105th Ave	3	10.39	6.50	8.00	298,893	27,153	15,097	2.76	1.46	1.87	1.48	zero	zero	zero	NC	NC	NC	
41	27+75	SB	SW 105th Ave	3	7.14	9.50	8.00	334,847	37,862	11,071	3.46	1.56	2.91	2.61	zero	zero	zero	NC	NC	NC	
42	27+24	SB	SW 105th Ave	3	14.00	7.00	8.00	218,238	38,903	4,568	2.73	2.41	1.82	1.45	zero	zero	zero	NC	NC	NC	
43	26+74	SB	SW 105th Ave	3	23.55	4.50	8.00	209,737	17,017	5,977	1.91	2.16	1.04	1.52	0.59	1.14	1.14	1.69	2.24	2.24	
44	26+26	SB	SW 105th Ave	3	8.58	8.50	8.00	244,059	58,703	9,743	3.24	1.77	2.34	1.46	zero	zero	zero	NC	NC	NC	
45	25+79	SB	SW 105th Ave	3	16.36	4.50	8.00	143,823	32,492	16,064	2.02	1.45	0.97	1.19	zero	0.53	0.53	NC	1.55	1.55	
46	25+24	SB	SW 105th Ave	Digout	24.87	3.00	8.00	339,692	12,692	12,768	1.62	1.54	0.86	1.81	zero	2.26	2.26	NC	3.63	3.63	
47	24+74	SB	SW 105th Ave	2	21.08	6.00	8.00	70,983	30,152	10,528	2.11	1.77	1.15	1.35	zero	0.47	0.47	NC	1.38	1.38	
48	24+21	SB	SW 105th Ave	2	25.78	4.50	8.00	266,678	13,934	4,708	1.94	2.37	1.08	1.55	1.03	1.12	1.12	2.17	2.26	2.26	
49	23+74	SB	SW 105th Ave	2	26.16	4.50	8.00	169,618	18,283	5,884	1.86	2.18	1.01	1.57	0.76	1.34	1.34	1.82	2.41	2.41	
50	23+25	SB	SW 105th Ave	2	22.39	4.00	8.00	314,374	14,130	8,936	1.85	1.84	1.00	1.57	zero	1.35	1.35	NC	2.55	2.55	
51	22+75	SB	SW 105th Ave	2	23.37	5.00	8.00	128,047	23,216	6,955	1.98	2.05	1.07	1.45	0.18	0.90	0.90	1.20	1.92	1.92	
52	22+25	SB	SW 105th Ave	2	31.55	3.50	8.00	263,870	11,458	5,420	1.58	2.23	0.83	1.81	1.53	2.35	2.35	2.66	3.48	3.48	
53	21+75	SB	SW 105th Ave	2	30.46	4.00	8.00	220,744	11,174	5,841	1.77	2.18	1.00	1.79	0.97	1.88	1.88	2.17	3.07	3.07	
54	21+25	SB	SW 105th Ave	Digout	34.50	4.50	8.00	149,698	23,989	2,109	1.92	3.27	0.96	1.35	3.24	0.93	3.24	4.25	1.95	4.25	
55	20+73	SB	SW 105th Ave	Digout	25.05	4.00	8.00	283,400	11,533	6,232	1.74	2.09	1.00	1.87	0.83	2.08	2.08	2.02	3.27	3.27	
56	20+24	SB	SW Blake St	Digout	39.00	4.50	8.00	122,390	19,269	2,468	1.78	3.09	0.91	1.53	3.10	1.47	3.10	4.07	2.43	4.07	
57	19+72	SB	SW Blake St	2	23.81	4.00	8.00	309,949	12,003	6,846	1.78	2.02	1.03	1.83	0.58	1.91	1.91	1.80	3.13	3.13	
58	19+24	SB	SW Blake St	2	33.18	4.00	8.00	217,057	10,944	3,344	1.66	2.69	0.89	1.77	2.46	2.09	2.46	3.51	3.15	3.51	
59	18+75	SB	SW Blake St	2	40.15	3.50	8.00	216,968	8,339	3,441	1.44	2.61	0.79	2.15	2.79	3.26	3.26	3.86	4.33	4.33	
60	18+25	SB	SW Blake St	2	28.92	4.50	8.00	228,740	15,753	2,790	1.90	2.90	1.08	1.63	2.36	1.32	2.36	3.50	2.46	3.50	

Table 1C: BACKCALCULATION AND OVERLAY ANALYSIS RESULTS

						Structural Characteristics					Calculated Structural Number				Overlay/Inlay Analysis: Overlay Thickness above Existing Pavement			Overlay/Inlay Analysis: Overlay Thickness above 2-inch Milled Surface			
FWD Test #	Test Station	Dir	Street	Analysis Unit	D0, mils	AC Thickness, inches	AB Thickness, inches	AC Modulus @ 68 °F & 10 Hz, psi	AB Modulus, psi	Subgrade M_r at 6 psi deviator stress, psi	SNeff of Existing Pavement Above SG	Required SN above SG	SNeff of Existing Pavement Above AB	Required SN above AB	Based on Analysis above the Subgrade, inches	Based on Analysis above the Base, inches	Controlling Overlay (maximum of requirements above the SG and AB), inches	Based on Analysis above the Subgrade, inches	Based on Analysis above the Base, inches	Controlling Inlay Depth (maximum of requirements above the SG and AB), inches	Cores
61	17+72	SB	SW Blake St	2	18.89	5.00	8.00	221,624	18,773	6,423	2.08	2.09	1.23	1.57	0.01	0.80	0.80	1.18	1.97	1.97	
62	17+25	SB	SW Blake St	2	41.67	3.50	8.00	116,912	15,436	3,236	1.50	2.74	0.63	1.53	2.96	2.15	2.96	3.82	3.01	3.82	
63	16+75	SB	SW Blake St	2	50.88	3.50	8.00	124,438	10,925	2,068	1.50	3.29	0.68	1.65	4.26	2.30	4.26	5.18	3.23	5.18	
64	16+25	SB	SW Blake St	2	30.53	4.50	8.00	162,219	15,611	2,984	1.79	2.82	0.98	1.66	2.44	1.64	2.44	3.48	2.67	3.48	
65	15+87	SB	SW Blake St	1	23.99	6.00	4.75	118,270	52,087	3,381	1.85	2.72	1.24	1.22	2.08	zero	2.08	3.06	NC	3.06	B-1
66	15+25	SB	SW Blake St	Digout	22.15	4.50	8.00	118,754	25,013	8,386	1.87	1.88	0.90	1.32	0.04	1.02	1.02	0.99	1.96	1.96	
67	14+74	SB	SW 108th Ave	Digout	14.46	6.00	8.00	291,816	26,993	5,612	2.48	2.21	1.64	1.58	zero	zero	zero	NC	NC	NC	
68	14+25	SB	SW 108th Ave	1	23.33	5.00	8.00	185,993	18,373	3,791	2.00	2.56	1.17	1.61	1.33	1.05	1.33	2.44	2.16	2.44	
69	13+75	SB	SW 108th Ave	1	18.03	5.00	8.00	491,830	6,300	5,608	2.33	2.48	1.35	1.30	0.36	zero	0.36	1.64	NC	1.64	
70	13+24	SB	SW 108th Ave	1	15.09	6.00	8.00	259,318	22,997	5,264	2.47	2.24	1.60	1.52	zero	zero	zero	NC	NC	NC	
71	12+74	SB	SW 108th Ave	1	16.04	5.50	8.00	238,548	25,233	7,390	2.21	1.97	1.51	2.00	zero	1.16	1.16	NC	2.47	2.47	
72	12+24	SB	SW 108th Ave	1	16.47	4.50	8.00	197,305	26,103	12,294	2.02	1.60	1.07	1.36	zero	0.70	0.70	NC	1.82	1.82	
73	11+75	SB	SW 108th Ave	1	13.36	6.00	8.00	134,449	51,472	9,115	2.42	1.83	1.32	1.12	zero	zero	zero	NC	NC	NC	
74	11+24	SB	SW 108th Ave	1	16.69	4.50	8.00	137,385	34,497	11,665	2.03	1.65	0.93	1.12	zero	0.45	0.45	NC	1.44	1.44	
75	10+75	SB	SW 108th Ave	1	16.89	5.50	8.00	174,400	33,689	6,632	2.33	2.07	1.26	1.16	zero	zero	zero	NC	NC	NC	
76	10+24	SB	SW 108th Ave	1	12.18	6.00	8.00	245,198	31,764	10,181	2.45	1.72	1.59	1.56	zero	zero	zero	NC	NC	NC	

Statistical Summary

															Summary of Overlay Analysis Results			Summary of Inlay Analysis Results			
Structural Unit#	From Sta	To Sta	Direction		D0, mils	Average AC Thickness, in.	Average AB Thickness, in.	Average AC Modulus, psi	Average AB Modulus, psi	Average SG Modulus, psi	Average SNeff of Existing Pavement Above SG	Average Required SN above SG	Average SNeff of Existing Pavement Above AB	Average Required SN above AB	Average Based on Analysis above the Subgrade, inches	Average Based on Analysis above the Base, inches	Average Controlling Overlay (maximum of requirements above the SG and AB), inches	Average Overlay above the 2-inch Milled Surface Based on Analysis above the Subgrade, inches	Average Overlay above the 2-inch Milled Surface Based on Analysis above the Base, inches	Average Controlling Overlay above the 2-inch Milled Surface (maximum of requirements above the SG and AB), inches	Average Grade Increase, in
1	10+24	16+01	NB & SB		17.08	5.24	8.00	243,925	27,263	7,480	2.25	2.07	1.30	1.40	0.80	0.54	0.80	1.92	1.70	1.92	0.00
2	16+25	25+00	NB & SB		28.41	4.30	8.07	216,819	16,687	5,419	1.84	2.38	1.00	1.65	1.64	1.61	1.64	2.75	2.71	2.75	0.75
3	25+51	28+75	NB & SB		15.93	6.18	8.00	258,977	28,155	9,017	2.49	1.90	1.65	1.68	0.56	1.05	1.05	1.73	2.23	2.23	0.23

Design Subgrade Resilient Modulus for Digout or Reconstruction

Structural Unit #	From	To	Direction	Average Subgrade Modulus, psi	2.3 Percentile Subgrade Modulus/0.70, psi	Subgrade Modulus for Design, psi
1	10+24	16+01	NB & SB	7,480	4,987	3,100
2	16+25	25+00	NB & SB	5,419	3,187	
3	25+51	28+75	NB & SB	9,017	6,817	
Digout	12+00	25+24	NB & SB	5,476	3,107	

Table 2C: PAVEMENT DESIGN WORKSHEET FOR CONSTRUCTION WITH COMPACTED SUBGRADE

Project Segment: Garden Corner Curves - SW 108th Ave, SW Blake Street, SW 105th Ave
Design Alternative 1: Areas with Traffic Loading - Compacted Subgrade

AASHTO Design Parameters & Input Values:		Notes	
Design Period, Yrs:	20	per 2011 ODOT Pavement Design Guide Section 8.2	**Denotes user data field**
Cumulative ESAL Repetitions:	50,000	per 1993 AASHTO Design Guide	
Design Reliability:	80	per 2019 ODOT Pavement Design Guide Table 11	
Overall Standard Deviation, S_o:	0.49	per 2019 ODOT Pavement Design Guide Table 15	
Initial Serviceability, P_o:	4.2	per 2019 ODOT Pavement Design Guide Table 13	
Terminal Serviceability, P_t:	2.5	per 2019 ODOT Pavement Design Guide Table 14	
Effective Subgrade M_r, psi:	3,100		
Aggregate Base Backfill Modulus, psi:	6,776	per Dorman and Metcalf Procedure	**Denotes calculated fiel**
Aggregate Base Modulus, psi:	20,000	per 2019 ODOT Pavement Design Guide	
Asphalt Concrete (AC) Layer Coefficient:	0.42	per 2019 ODOT Pavement Design Guide Table 16	
Aggregate Base Backfill Layer Coefficient:	0.06	per 1993 AASHTO Design Guide (minimum 0.06)	
Aggregate Base (AB) Layer Coefficient:	0.10	per 2019 ODOT Pavement Design Guide Table 16	
AB Drainage Coefficient:	1.0	per 2019 ODOT Pavement Design Guide Section 5.5	
SN required above subgrade:	2.79		
SN required above ASB:	2.06		
SN required above AB:	1.32		

Pavement Section

Layer Description	Thickness, in.	Layer Coeff.	SN	SN Subtotals		Notes
Level 2, 1/2-inch ACP	3.00	0.42	1.26			PG 64-22
Level 2, 1/2-inch ACP	2.00	0.42	0.84	2.10	>1.32 required above AB - OK	PG 64-22
1" or 3/4"-0 Aggregate Base	4.00	0.13	0.52	2.62	>2.06 required above ASB - OK	
1" or 3/4"-0 Aggregate Base Backfil	4.00	0.06	0.24	2.86	>2.79 required above subgrade - OK	
Compacted Subgrade	12.00					
Geotextile	NA					
Total Depth	13.00					

GRI

Table 3C: PAVEMENT DESIGN WORKSHEET FOR CONSTRUCTION WITH AGGREGATE STABILIZATION WITH GEOTEXTILE

Project Segment: Garden Corner Curves - SW 108th Ave, SW Blake Street, SW 105th Ave
Design Alternative 2: Areas with Traffic Loading - Geotextile Reinforced Working Platform

AASHTO Design Parameters & Input Values:		Notes	
Design Period, Yrs:	20	per 2011 ODOT Pavement Design Guide Section 8.2	**Denotes user data field**
Cumulative ESAL Repetitions:	50,000	per 1993 AASHTO Design Guide	
Design Reliability:	80	per 2019 ODOT Pavement Design Guide Table 11	
Overall Standard Deviation, S_o:	0.49	per 2019 ODOT Pavement Design Guide Table 15	
Initial Serviceability, P_o:	4.2	per 2019 ODOT Pavement Design Guide Table 13	
Terminal Serviceability, P_t:	2.5	per 2019 ODOT Pavement Design Guide Table 14	
Effective Subgrade M_r, psi:	3,100	per PAVBACK Backcalculation Analysis	
Aggregate Base Backfill Modulus, psi:	8,716	per Dorman and Metcalf Procedure	**Denotes calculated fiel**
Aggregate Base Modulus, psi:	20,000	per 2019 ODOT Pavement Design Guide	
Asphalt Concrete (AC) Layer Coefficient:	0.42	per 2019 ODOT Pavement Design Guide Table 16	
Aggregate Base Backfill Layer Coefficient:	0.06	per 1993 AASHTO Design Guide (minimum 0.06)	
Aggregate Base (AB) Layer Coefficient:	0.10	per 2019 ODOT Pavement Design Guide Table 16	
AB Drainage Coefficient:	1.0	per 2019 ODOT Pavement Design Guide Section 5.5	
Minimum AB thickness on geotextile for support of construction, in.:	14.0	per Giroud & Han procedure	
SN required above subgrade:	2.79		
SN required above ASB:	1.86		
SN required above AB:	1.32		

Pavement Section

Layer Description	Thickness, in.	Layer Coeff.	SN	SN Subtotals		Notes
Level 2, 1/2-inch ACP	3.00	0.42	1.26			PG 64-22
Level 2, 1/2-inch ACP	2.00	0.42	0.84	2.10	> 1.32 required above AB - OK	PG 64-22
1" or 3/4"-0 Aggregate Base	4.00	0.13	0.52	2.62	> 1.86 required above ASB - OK	
1" or 3/4"-0 Aggregate Base Backfil	10.00	0.06	0.60	3.22	> 2.79 required above subgrade - OK	
Geotextile	NA					
Total Depth	19.00					

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