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August 16, 2024

City of Manzanita
Dan Weitzel, Public Works Director
1090 Oak Street
Manzanita, Oregon 97130

Report of
Limited Geotechnical Investigation
Classic Street Improvements
Classic Street
Manzanita, Oregon

CGT Project Number G2406158

Dear Dan Weitzel:

Carlson Geotechnical (CGT), a division of Carlson Testing, Inc. (CTI), is pleased to submit this report summarizing the results of our limited geotechnical investigation for the proposed improvements to Classic Street. The subject roadway is located between Dorcas Lane and Necarney City Road in Manzanita, Oregon. We performed our work in general accordance with CGT Proposal GP24-125, dated May 9, 2024. Written authorization for our services was received on June 10, 2024.

We appreciate the opportunity to work with you on this project. Please contact us at (503) 601-8250 if you have any questions regarding this report.

Respectfully Submitted,
CARLSON GEOTECHNICAL

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ATTACHMENTS

1.0 INTRODUCTION

Carlson Geotechnical (CGT), a division of Carlson Testing, Inc. (CTI), is pleased to submit this report summarizing the results of our limited geotechnical investigation for the proposed improvements to Classic Street. The subject roadway is located between Dorcas Lane and Necarney City Road in Manzanita, Oregon, as shown on the attached Site Location, Figure 1.

1.1 Project Information

CGT developed an understanding of the proposed project based on our correspondence with the City of Manzanita and project documents provided to us. The documents provided included an aerial image showing the proposed boring locations, and a site schematic plan, dated March 24, 2024. Based on our review, we understand the project will include improvements to the existing Classic Street. The improvements will take place over an approximate 2,220-foot long stretch of the roadway, effectively spanning between Dorcas Lane and Necarney City Road. The improvements are anticipated to include, but not limited, to widening of the roadway, installation of underground utilities, installation of sidewalks, installation of site retaining wall(s), and other features. Design of the roadway improvements will rest with others.

Although no grading plans have been provided, we anticipate permanent grade changes at the site will be minimal, with maximum cuts and fills on the order of 2 feet in depth.

Although no stormwater plans have been provided, we anticipate stormwater collected from new impervious areas of the site will be collected and routed to the nearest storm drain or other suitable discharge point(s) approved by Tillamook County.

1.2 Scope of Services

Our scope of work included the following:

- Contact the Oregon Utilities Notification Center to mark the locations of public utilities within a 20-foot radius of our explorations at the site.
- Explore subsurface conditions within the roadway (Classic Street) by advancing six drilled borings and six dynamic cone penetrometer (DCP) tests to depths of up to about 11½ feet below pavement surface (bps). Details of the subsurface investigation are presented in Appendix A. Results of the DCP tests are presented in Appendix B.
- Classify the soils encountered in the explorations in general accordance with ASTM D2488 (Visual-Manual Procedure).
- Provide a technical narrative describing surface and subsurface deposits, and local geology of the site, based on the results of our explorations and published geologic mapping.
- Provide geotechnical recommendations for site preparation and earthwork.
- Perform a structural capacity evaluation of the existing pavement structure within the referenced roadway in general accordance with Sections 5.3 and 5.4 of the 1993 AASHTO Pavement Design Manual.
- Provide geotechnical engineering recommendations for use in design and construction of site retaining walls and pavements.
- Provide this written limited geotechnical report summarizing the results of the field investigation and recommendations for the project.

This report is considered "limited" as this assignment did not include an evaluation of seismic/geologic hazards at the site.

2.0 SITE DESCRIPTION

2.1 Site Geology

Based on available geologic mapping of the area[1], the site is underlain by Holocene age, beach and dune deposits (Qb). This unit consists primarily of unconsolidated, moderately well sorted, fine- to medium-grained beach sand. The area is also composed of cross-bedded, fine-grained sand deposited through active and inactive dune ridges. The beach and dune deposits are occasionally interbedded with fluvial and lacustrine mud and sand deposits found inland from the dune ridges, as well as locally found basalt gravel and boulder debris deposited from erosion of rocky headlands.

2.2 Site Surface Conditions

The subject portion of Classic Street is a two-lane, asphalt-paved roadway that generally runs north to south. Classic Street spans approximately 2,220 feet and connects Dorcas Lane and Necarney City Road. The road is located within a relatively level to gently sloping area and provides vehicular access to both established residential properties and unestablished residential properties (i.e., portions of subdivisions yet to be fully built out). Residential streets that intersect with Classic Street include Ridge Drive, Highlands Drive, and Jackson Way.

In terms of topography adjacent to the street, the northern 950 feet (approximate) of the street was flanked by a descending vegetated slope exhibiting gradients of about 2H:1V (horizontal:vertical) to 1½H:1V. The central portion of the street (between the south end of Jackson Way and spanning about 450 feet) was flanked by a vegetated/forested ascending slope exhibiting gradients of up to about 1½H:1V. The remaining street areas were generally flanked by level to gentle side slopes.

Site layout and surface conditions at the time of our field investigation are shown on the attached Site Plan (Figure 2) and Site Photographs (Figure 3).

2.3 Subsurface Conditions

2.3.1 <u>Subsurface Investigation & Laboratory Testing</u>

Our subsurface investigation consisted of six drilled borings (B-1 through B-6) completed on July 8, 2014. The approximate exploration locations are shown on the Site Plan, attached as Figure 2. In summary, the borings were advanced to depths of about 11½ feet bps. Details regarding the subsurface investigation, logs of the explorations, and results of laboratory testing are presented in Appendix A. Subsurface conditions encountered during our investigation are summarized below.

2.3.2 <u>Subsurface Materials</u>

Logs of the explorations are presented in Appendix A. The following describes each of the subsurface materials encountered at the site.

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[1] **Wells, R.E., Snavely, P.D., MacLeod, N.S., Kelly, M.M., and Parker, M.J., 1994,** Geologic map of the Tillamook Highlands, northwest Oregon Coast Range (Tillamook, Nehalem, Enright, Timber, Fairdale, and Blaine 15 minute quadrangles)**: U.S. Geological Survey, Open-File Report OF-94-21, scale 1:62,500**.

Asphalt Concrete Pavement

Asphalt concrete (AC) pavement was encountered at the surface of each boring and was about 2 to 3 inches thick.

Undocumented Poorly Graded Gravel Fill (GP Fill)

Undocumented poorly graded gravel fill (aggregate base rock) was encountered below the AC pavement in each boring. Undocumented fill refers to materials placed without (available) records of subgrade conditions or evaluation of compaction. The poorly graded gravel fill was typically brown, dry, angular, up to about ¾-inch in diameter, and contained no to trace low plasticity fines. The gravel fill extended to depths of about 1⅓ to 3 feet bps.

Elastic Silt (MH)

Underlying the gravel fill in boring B-6 was native elastic silt. This soil was typically stiff, brown, moist, exhibited medium plasticity, and contained trace fine-grained sand. This soil extended to a depth of about 5½ feet bgs in that boring.

Silty Sand (SM)

Underlying the gravel fill in borings B-2, B-3, and B-5, was native, silty sand. This soil was typically loose to medium dense, tan, moist, fine- to medium-grained, and contained varying amounts of low to medium plasticity silt. This soil extended to depths of about 5 feet bps in those borings.

Poorly Graded Sand (SP)

Underlying the gravel fill in borings B-1 and B-4, the silty sand in borings B-2, B-3 and B-5, and the elastic silt in boring B-6, was native, poorly graded sand. This soil was typically loose to medium dense, tan, moist to wet, fine- to medium-grained, and contained no to trace low plasticity silt. This soil extended to the full depths explored in the borings, about 11½ feet bps.

2.3.3 Groundwater

Groundwater was encountered at a depth of about 10 feet bgs in boring B-1 advanced on July 8, 2024. Groundwater was not encountered within the remaining borings, B-2 through B-6, advanced on that day. To determine approximate regional groundwater levels in the area, we researched well logs available on the Oregon Water Resources Department (OWRD)[2] website for wells located within Section 29, Township 03 North, Range 10 West, Willamette Meridian. Our review indicated that groundwater levels in the area generally ranged from about 50 to 59 feet bgs. It should be noted groundwater levels vary with local topography. In addition, the groundwater levels reported on the OWRD logs often reflect the purpose of the well, so water well logs may only report deeper, confined groundwater, while geotechnical or environmental borings will often report any groundwater encountered, including shallow, unconfined groundwater. Therefore, the levels reported on the OWRD well logs referenced above are considered generally indicative of local water levels and may not reflect actual groundwater levels at the project site. We anticipate that groundwater levels will fluctuate due to seasonal and annual variations in precipitation, changes in site utilization, or other factors. Additionally, the native elastic silt (MH) is conducive to formation of perched groundwater.

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[2] Oregon Water Resources Department, 2024. Well Log Records, *accessed June 2024,* from OWRD web site: http://apps.wrd.state.or.us/apps/gw/well_log/.

3.0 PAVEMENT STRUCTURAL CAPACITY EVALUATION

CGT performed a pavement structural capacity evaluation within the subject portion of Classic Street to determine whether structural enhancement (e.g. an overlay) was appropriate to help meet design vehicular traffic loading over a design period of 20 years and maintain a minimum standard level of serviceability. The results of the evaluation are presented in the attached Appendix C.

4.0 CONCLUSIONS

4.1 Overview

As indicated in the attached Appendix C, our analyses indicate the existing pavement structure within the subject portion of Classic Street exhibited a modest structural deficiency for the modeled vehicular traffic over a 20-year design period. As evidenced during our visual condition survey, we observed localized areas exhibiting fatigue cracking, longitudinal cracking, transverse cracking, and other distress within the existing AC pavement. Three pavement areas within the north portion of the street exhibited localized subsidence (slumping); additional discussion of those areas is presented in Section 4.2 above.

We conclude the existing AC pavement is approaching the end of its intended service life and improvements are warranted to maintain desired minimum level of serviceability over the indicated design period (20 years). Three options may be considered for improving Classic Street, as follows:

- **Option 1 – Repair Surface Deficiencies & Install Overlay:** This option would include repairing/treating surface deficiencies (e.g. fatigue cracking, longitudinal cracks, etc.) within the existing pavement structure and installing an overlay. Based on our analyses and factoring in best practices for placing AC pavement, we recommend the overlay be a minimum of 1½ inches thick. If overlaying is considered, we recommend the project civil engineer be consulted to review impacts to stormwater management, as well as review grade changes with respect to existing nearby features (public streets, sidewalks, curbs, etc.). Geotechnical recommendations for placement of a pavement overlay within the subject roadway, if considered, are presented in Section 6.1 of this report.
- **Option 2 – Full Removal & Replacement (R&R):** This option would include removing the existing AC pavement and installation of a new AC pavement section. Recommendations for this approach are presented in Section 6.2 of this report.
- **Option 3 – Full Depth Reclamation (FDR):** This option would include pulverizing the existing AC, blending it with the underlying aggregate base in-situ, compacting the materials to serve as aggregate base, and placing a new AC section. If this is considered, we recommend the project civil engineer be consulted to review impacts to stormwater management, as well as review inherent grade changes with respect to existing nearby features (public streets, sidewalks, curbs, etc.). Recommendations for this approach are presented in Section 6.3 of this report.

Other options typically pursued in pavement rehabilitation, including "grind and inlay" and surface treatments (e.g. slurry seals, chip seals, etc.), are not recommended for Classic Street. The grind and inlay technique is not recommended due to the relatively thin (predominantly 2 inches thick) existing pavement section. Surface treatments are not recommended due to the structural deficiency identified in our analyses.

4.2 Pavement Areas Exhibiting Subsidence

As indicated above and shown on the attached Site Plan, Figure 2, we observed three areas exhibiting subsidence (slumping) within the north portion of the street alignment. Each area was located along the west margin of the road and relatively close to a relatively steep, descending slope. The cause(s) of the subsidence was not unequivocally determined, but may be due to one, or a combination of, the following factors: (1) long-term (gradual) downslope movement (creep) of the near surface slope materials and (2) long-term consolidation (settlement) from transient (vehicular) loads of the subgrade materials directly below the pavement materials. Mitigation of these areas is recommended to provide assurance of long-term performance of the pavement structure. The following options are presented for consideration:

- **Installation of Retaining Wall(s):** This option would include installation of engineered retaining wall(s) at the top, or at some point within, the descending slope directly west of those slumping areas. Recognizing the relatively steep slopes, we recommend consideration be made to utilize pile-supported walls (e.g., sheet pile walls, soldier pile walls, etc.). Once the retaining wall(s) have been installed, the affected pavements should be removed and soft/loose subgrade soils (if present) should be over-excavated and replaced with structural fill. Geotechnical (soil) parameters for use in design of pile-supported walls are presented in Section 7.2 of this report.
- **Buttressing Slopes:** This option would include buttressing the descending slope (west of street) by adding new fill in a controlled (engineered) manner and achieve a maximum gradient of 2H:1V. This would invariably include removal of existing trees and vegetation on the slope and near its toe, and extending the slope outward (beyond its current footprint) to achieve that gradient. Keying and benching of the existing slope is recommended prior to placement of new structural fill. If considered, we recommend this approach be reviewed by the project civil engineer to review whether special considerations[3] are applicable for this construction.
- **Realignment of Street Segment:** This option would include realigning this segment of the street towards the east to achieve a greater setback from the descending slope. If considered, we recommend this approach be reviewed by the project civil engineer to review whether special considerations[4] are applicable to allow for this construction.

5.0 RECOMMENDATIONS: SITE WORK

The recommendations presented in this report are based on the information provided to us, results of our field investigation and analyses, laboratory data, and professional judgment. CGT has observed only a small portion of the pertinent subsurface conditions. The recommendations are based on the assumptions that the subsurface conditions do not deviate appreciably from those found during the field investigation. CGT should be consulted for further recommendations if the design of the proposed development changes and/or variations or undesirable geotechnical conditions are encountered during site development.

5.1 Site Preparation

The following recommendations are presented in the event the existing pavement structure is removed in its entirety (R&R) and/or the project includes widening the existing roadway beyond its current footprint.

3 Review of the extent of the public right of way and impacts to neighboring properties (to the west) would need to be evaluated.
4 Review of the extent of the public right of way and impacts to neighboring properties (to the east) would need to be evaluated.

5.1.1 Stripping

Stripping activities associated with site preparation should be minimal at this site. Where slated for removal, existing asphalt concrete (AC) pavement, surface vegetation, and rooted soils should be removed from within, and for a minimum 3-foot margin around (where feasible), planned new pavements and retaining walls. Stripped AC should be transported off site for disposal, or stockpiled for later use as structural fill on the project site as described in Section 5.4.1 of this report. Stripped rooted soils should be transported off site for disposal, or stockpiled for later use as landscaping fill on the site.

5.1.2 Existing Utilities & Below-Grade Structures

All existing utilities at the site should be identified prior to excavation. Abandoned utility lines beneath the new pavements, retaining walls, and hardscaping features should be completely removed or grouted full. Soft, loose, or otherwise unsuitable soils encountered in utility trench excavations should be removed and replaced with structural fill in conformance with Section 5.4 this report. Buried structures (i.e. footings, foundation walls, retaining walls, slabs-on-grade, tanks, etc.), if encountered during site development, should be completely removed and replaced with structural fill in conformance with Section 5.4 of this report.

5.1.3 Roadway Subgrade Preparation

5.1.3.1 Dry Weather Construction

After site preparation as recommended above, but prior to placement of structural fill and/or aggregate base, the geotechnical engineer or his representative should observe a proof roll test of the exposed subgrade soils in order to identify areas of excessive yielding. Proof rolling of subgrade soils is typically conducted during dry weather conditions using a fully-loaded, 10- to 12-cubic-yard, tandem-axle, tire-mounted, dump truck or equivalent weighted water truck. Areas that appear too soft and wet to support proof rolling equipment should be prepared in general accordance with the recommendations for wet weather construction presented in Section 5.3 of this report. If areas of soft soil or excessive yielding are identified, the affected material should be over-excavated to firm, stable subgrade, and replaced with imported granular structural fill in conformance with Section 5.4.2 of this report.

5.1.3.2 Wet Weather Construction

Preparation of pavement subgrade soils during wet weather should be in conformance with Section 5.3 of this report. As indicated therein, a granular sub-base and geotextile separation fabric may be required in wet conditions in order to support construction traffic and protect the subgrade. Cement amendment may also be considered to help stabilize subgrade soils during wet weather.

5.1.4 Erosion Control

Erosion and sedimentation control measures should be employed in accordance with applicable City, County, and State regulations.

5.2 Temporary Excavations

5.2.1 Overview

Conventional earthmoving equipment in proper working condition should be capable of making necessary excavations for the anticipated site cuts as described earlier in this report. All excavations should be in accordance with applicable OSHA and state regulations. It is the contractor's responsibility to select the excavation methods, to monitor site excavations for safety, and to provide any shoring required to protect personnel and adjacent improvements. A “competent person,” as defined by OR-OSHA, should be on-site during construction in accordance with regulations presented by OR-OSHA. CGT’s current role on the project does not include review or oversight of excavation safety.

5.2.2 OSHA Soil Type

For use in the planning and construction of temporary excavations up to 10 feet in depth, an OSHA soil type “C” should be used for the granular soils (GP Fill, SM, SP) encountered in the borings. Similarly, an OSHA soil type “A” may be used for the native elastic silt (MH) encountered in boring B-6.

5.2.3 Utility Trenches

Caving is anticipated in excavations extending more than a few feet below the ground surface, particularly in areas underlain by relatively clean loose sand (SP). If seepage undermines the stability of the trench, or if caving of the sidewalls is observed during excavation, the sidewalls should be flattened or shored. Depending on the time of year trench excavations occur, trench dewatering may be required in order to maintain dry working conditions. If groundwater is encountered, we recommend placing trench stabilization material at the base of the excavations. Trench stabilization material should be in conformance with Section 5.4.3 of this report.

5.2.4 Excavations Near Foundations

Excavations near footings should not extend within a 1 horizontal to 1 vertical (1H:1V) plane projected out and down from the outside, bottom edge of the footings. In the event excavation needs to extend below the referenced plane, temporary shoring of the excavation and/or underpinning of the subject footing may be required. The geotechnical engineer should be consulted to review proposed excavation plans for this design case to provide specific recommendations.

5.3 Wet Weather Considerations

For planning purposes, the wet season should be considered to extend from late September to late June. It is our experience that dry weather working conditions should prevail between early July and mid-September. Notwithstanding the above, soil conditions should be evaluated in the field by the geotechnical engineer or their representative at the initial stage of site preparation to determine whether the recommendations within this section should be incorporated into construction.

5.3.1 Overview

Due to their fines content, the on-site near-surface silty soils (SM, MH) are susceptible to disturbance during wet weather. Trafficability of these soils may be difficult, and significant damage to subgrade soils could occur, if earthwork is undertaken without proper precautions at times when the exposed soils are more than a few percentage points above optimum moisture content. For wet weather construction, site preparation activities may need to be accomplished using track-mounted equipment, loading removed material onto trucks supported on granular haul roads, or other methods to limit soil disturbance. The geotechnical engineer’s representative should evaluate the subgrade during excavation by probing rather than proof rolling. Soils that have been disturbed during site preparation activities, or soft or loose areas identified during probing, should be over-excavated to firm, unyielding subgrade, and replaced with imported granular structural fill in conformance with Section 5.4.2.

5.3.2 Geotextile Separation Fabric

We recommend a geotextile separation fabric be placed to serve as a barrier between the prepared subgrade and granular fill/base rock in areas of repeated or heavy construction traffic. The geotextile fabric should meet the requirements presented in the current Oregon Department of Transportation (ODOT) Standard Specification for Construction (ODOT SSC), Section 02320.

5.3.3 Granular Working Surfaces (Haul Roads & Staging Areas)

Haul roads subjected to repeated heavy, tire-mounted, construction traffic (e.g. dump trucks, concrete trucks, etc.) will require a minimum of 18 inches of imported granular material. For light staging areas, 12 inches of imported granular material is typically sufficient. Additional granular material, cement amendment, or geo-grid reinforcement may be recommended based on site conditions and/or loading at the time of construction. The imported granular material should be in conformance with Section 5.4.2 and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. The prepared subgrade should be covered with geotextile fabric (Section 5.3.2) prior to placement of the imported granular material. The imported granular material should be placed in a single lift (up to 24 inches deep) and compacted using a smooth-drum, non-vibratory roller until well-keyed.

5.3.4 Cement Amendment

It is sometimes less costly to amend near-surface, moisture-sensitive, fine-grained soils with Portland cement than to remove and replace those soils with imported granular material. Successful use of soil cement amendment depends on use of correct techniques and equipment, soil moisture content, and the amount of cement added to the subgrade (mix design). We anticipate the on-site native silty and sandy soils (SM, SP, MH) are conducive for cement amendment due to their generally low plasticity and experience with similar soils.

The recommended percentage of cement is based on soil moisture contents at the time the work is performed. Based on our experience, 3 percent cement by weight of dry soil can generally be used when the soil moisture content does not exceed approximately 20 percent. If the soil moisture content is in the range of 25 to 35 percent, 4 to 6 percent by weight of dry soil is recommended. Similarly, if the soil moisture content is in the range of 35 to 45 percent, 7 to 8 percent by weight of dry soil is recommended. It is difficult to accurately predict field performance due to the variability in soil response to cement amendment. The amount of cement added to the soil may need to be adjusted based on field observations and performance.

If cement amendment is considered, we recommend additional sampling, laboratory testing, and a mix design be performed to determine the level of improvement in engineering properties (strength, stiffness) of the on-site soils when blended with Portland cement. We recommend project scheduling allow for a minimum of 4 weeks for this testing and design to be completed, prior to initiating cement amendment.

5.3.5 Footing Subgrade Protection

A minimum of 3 inches of imported granular material (crushed rock) is recommended to protect fine-grained (silty) footing subgrades from foot traffic during inclement weather. The imported granular material should be in conformance with Section 5.4.2. The maximum particle size should be limited to 1 inch. The imported granular material should be placed in one lift over the prepared, undisturbed subgrade, and compacted using non-vibratory equipment until well keyed.

Surface water should not be allowed to collect in footing excavations. The excavations should be draped and/or provided with sumps to preclude water accumulation during inclement weather.

5.4 Structural Fill

The geotechnical engineer should be provided the opportunity to review all materials considered for use as structural fill (prior to placement). Samples of the proposed fill materials should be submitted to the geotechnical engineer a minimum of 5 business days prior their use on site[5]. The geotechnical engineer or their representative should be contacted to evaluate compaction of structural fill as the material is being placed. Evaluation of compaction may take the form of in-place density tests and/or proof roll tests with suitable equipment. Structural fill should be evaluated at intervals not exceeding every 2 vertical feet as the fill is being placed. The following table presents recommended guidelines for frequency of density testing (where practical) of various fill designations.

Table 1 Guidelines for Frequency of Density Testing of Structural Fill Materials

Fill Designation	Recommended Frequency of Density Tests[1]	
	Maximum Depth Interval	Area-Wide
General Structural Fill (Mass Grading)	Test every 1 vertical foot	At least one density test per every 100 feet of roadway
Utility Trench Backfill	Test every 2 vertical feet	At least one density test per 100 feet of trench line
Pavement Base Rock	Test at surface of section	At least one density test per every 100 feet of roadway

[1] Or as specified by the City of Manzanita, where applicable.

5.4.1 <u>On-Site Soils – General Use</u>

5.4.1.1 Asphalt Concrete Debris

Debris resulting from the demolition of existing pavements can be re-used as structural fill if processed/crushed into material that is fairly well graded between coarse and fine. The processed/crushed concrete should contain no organic matter, debris, or particles larger than 4 inches in diameter. Moisture conditioning (wetting) should be expected in order to achieve adequate compaction. When used as structural fill, this material should be placed and compacted in general accordance with Section 5.4.2.

5.4.1.2 Poorly Graded Gravel Fill (GP Fill), Poorly Graded Sand (SP)

Re-use of the on-site, relatively clean, poorly graded gravel fill and relatively clean sand as structural fill is feasible, provided these materials are kept clean of organics, debris, and particles larger than 4 inches in diameter. If reused as structural fill, these materials should be prepared in general accordance with Section 5.4.2.

5.4.1.3 Elastic Silt (MH), Silty Sand (SM)

Re-use of these soils as structural fill may be difficult because <u>they are sensitive to small changes in moisture content and are difficult, if not impossible, to adequately compact during wet weather</u>. We anticipate the moisture content of these soils will be higher than the optimum moisture content for satisfactory compaction. Therefore, <u>moisture conditioning (drying) should be expected in order to achieve adequate compaction</u>. If used as structural fill, these soils should be free of organic matter, debris, and particles larger than 4 inches. When used as structural fill, these soils should be placed in lifts with a maximum pre-compaction thickness of about 8 inches at moisture contents within –1 and +3 percent of optimum, and

5 Laboratory testing for moisture density relationship (Proctor) is required. Tests for gradation may be required.

compacted to not less than 92 percent of the material's maximum dry density, as determined in general accordance with AASHTO T180 (Modified Proctor).

If the on-site materials cannot be properly moisture-conditioned and/or processed, we recommend using imported granular material for structural fill.

5.4.2 Imported Granular Structural Fill – General Use

Imported granular structural fill should consist of angular pit or quarry run rock, crushed rock, or crushed gravel that is fairly well graded between coarse and fine particle sizes. The granular fill should contain no organic matter, debris, or particles larger than 4 inches, and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. For fine-grading purposes, the maximum particle size should be limited to 1½ inches. The percentage of fines can be increased to 12 percent of the material passing the U.S. Standard No. 200 Sieve if placed during dry weather, and provided the fill material is moisture-conditioned, as necessary, for proper compaction. Imported granular fill material should be placed in lifts with a maximum thickness of about 12 inches, and compacted to not less than 95 percent of the material's maximum dry density as determined in general accordance with AASHTO T180 (Modified Proctor). Proper moisture conditioning and the use of vibratory equipment will facilitate compaction of these materials.

Granular fill materials with high percentages of particle sizes in excess of 1½ inches are considered non-moisture-density testable materials. As an alternative to conventional density testing, compaction of these materials should be evaluated by proof roll test observation (deflection tests), where accepted by the geotechnical engineer.

5.4.3 Trench Base Stabilization Material

If groundwater is present at the base of utility excavations, trench base stabilization material should be placed. Trench base stabilization material should consist of a minimum of 1 foot of well-graded granular material with a maximum particle size of 4 inches and less than 5 percent material passing the U.S. Standard No. 4 Sieve. The material should be free of organic matter and other deleterious material, placed in one lift, and compacted until well-keyed.

5.4.4 Trench Backfill Material

Trench backfill for the utility pipe base and pipe zone should consist of granular material as recommended by the utility pipe manufacturer. Trench backfill above the pipe zone should consist of well-graded granular material containing no organic matter or debris, have a maximum particle size of ¾ inch, and have less than 8 percent material passing the U.S. Standard No. 200 Sieve. As a guideline, trench backfill should be placed in maximum 12-inch-thick lifts. The earthwork contractor may elect to use alternative lift thicknesses based on their experience with specific equipment and fill material conditions during construction in order to achieve the required compaction. The following table presents recommended relative compaction percentages for utility trench backfill.

Table 2 Utility Trench Backfill Compaction Recommendations

Backfill Zone	Recommended Minimum Relative Compaction	
	Structural Areas[1,2]	Landscaping Areas
Pipe Base and Within Pipe Zone	90% AASHTO T180 or pipe manufacturer's recommendation	85% AASHTO T180 or pipe manufacturer's recommendation
Above Pipe Zone	92% AASHTO T180	88% AASHTO T180
Within 3 Feet of Design Subgrade	95% AASHTO T180	90% AASHTO T180

1 Includes proposed pavements, structural fill areas, hardscaping, etc.
2 Or as specified by the local jurisdiction where located in the public right of way.

5.4.5 Controlled Low-Strength Material (CLSM)

CLSM is a self-compacting, cementitious material that is typically considered when backfilling localized areas. CLSM is sometimes referred to as "controlled density fill" or CDF. Due to its flowable characteristics, CLSM typically can be placed in restricted-access excavations where placing and compacting fill is difficult. If chosen for use at this site, we recommend the CLSM be in conformance with Section 00442 of the most recent, ODOT SSC. The geotechnical engineer's representative should observe placement of the CLSM and obtain samples for compression testing in accordance with ASTM D4832. As a guideline, for each day's placement, two compressive strength specimens from the same CLSM sample should be tested. The results of the two individual compressive strength tests should be averaged to obtain the reported 28-day compressive strength. If CLSM is considered for use on this site, please contact the geotechnical engineer for site-specific and application-specific recommendations.

5.5 Permanent Slopes

5.5.1 Overview

Permanent cut or fill slopes constructed at the site, if any, should be graded at 2H:1V or flatter. Constructed slopes should be overbuilt by a few feet depending on their size and gradient so that they can be properly compacted prior to being cut to final grade. The surface of all slopes should be protected from erosion by seeding, sodding, or other acceptable means. Adjacent on-site and off-site structures should be located at least 5 feet from the top of slopes.

5.5.2 Placement of Fill on Slopes

New fill should be placed and compacted against horizontal surfaces. Where slopes exceed 5H:1V, the slopes should be keyed and benched prior to structural fill placement in general accordance with the attached Fill Slope Detail, Figure 4. If subdrains are needed on benches, subject to the review of the geotechnical representative, they should be placed as shown on the attached Fill Slope Detail. In order to achieve well-compacted slope faces, slopes should be overbuilt by a few feet and then trimmed back to proposed final grades. The geotechnical engineer or their representative should observe the benches, keyways, and associated subdrains, if needed, prior to placement of structural fill.

5.6 Additional Considerations

5.6.1 Drainage

Subsurface drains should be connected to the nearest storm drain or other suitable discharge point. Surface water from paved surfaces and open spaces should be collected and routed to a suitable discharge point. Surface water should not be directed into retaining wall drains or onto site slopes.

5.6.2 Expansive Potential

The near surface native soils consist of moderate plasticity elastic silt (MH) and sandy soils (SM, SP). Based on our experience with similar soils in the vicinity of the site, these soils are not considered to be susceptible to appreciable movements from changes in moisture content. Accordingly, no special considerations are required to mitigate expansive potential of the near surface soils at the site.

6.0 RECOMMENDATIONS: NEW PAVEMENTS

6.1 Option 1 – Pavement Overlay

6.1.1 Treatment of Surface Deficiencies

6.1.1.1 Overview

The long-term performance of repairs to surface deficiencies in asphalt pavement is highly dependent on the quality of workmanship. Accordingly, we recommend an experienced, qualified asphalt contractor be retained to repair deficiencies. The contractor is encouraged to follow repair guidelines and procedures presented in the most recent, ODOT Standard Specifications for Construction (ODOT SSC) and the most recent, "Asphalt in Pavement Maintenance" manual developed by the Asphalt Institute (AI). Other resources may be utilized for review of repair procedures. Subject to review of the pavement engineer, the contractor retained for the repair work may present alternative methods than those indicated below.

6.1.1.2 Fatigue Cracking

We recommend areas exhibiting moderate to severe fatigue (alligator) cracking be repaired as a "deep patch". Sawcutting and removal of existing pavement should extend at least 1-foot into good pavement outside the cracked area. We recommend this form of pavement repair be in conformance with Section 00748 of the most recent, ODOT SSC. If encountered, soft, loose, or otherwise unsuitable subgrade materials should be removed to expose suitably firm subgrade, and brought back to grade with imported granular structural fill in conformance with Section 5.4.2 of this report. For planning purposes, we recommend a minimum 6 inches of subgrade over-excavation be performed at each deep patch location. We recommend geotextile separation fabric be placed between the prepared subgrade and granular backfill. The fabric should be in conformance with Section 02320 of the most recent, ODOT SSC.

6.1.1.3 Linear Cracking

For areas exhibiting linear (longitudinal and transverse) cracking, we recommend that all cracks exceeding ¼-inch in width be cleaned and sealed with rubber or other elastomeric modified asphalt in conformance with Section 00746 of the most recent, ODOT SSC.

6.1.2 Overlay

The following is recommended for overlay surface preparation and construction:

- The subject portion of Classic Street that exhibits surface deficiencies should be repaired in conformance with the recommendations presented in Section 6.1.1 above.
- Once repair of surface deficiencies is complete, the surface that is to be overlaid should be thoroughly cleaned. Compressed air should be used for cleaning to remove all loose matter.
- A tack coat should be applied to the cleaned pavement surface in conformance with Section 00730 of ODOT SSC.
- The recommended minimum 1½-inch thick overlay section should be placed on the tack coated surface in conformance with the project civil plans. The AC pavement should consist of Level 2, ½-inch, dense-graded AC in conformance with the most recent ODOT SSC, or as specified by the City of Manzanita (City). Minimum lift thickness of AC pavement should be 1½ inches, or as specified by City. Maximum lift thickness of AC pavement should be in conformance with Section 00748 of the most recent ODOT SSC, or as specified by City. AC pavement should be compacted to at least 91 percent of the material's theoretical maximum density as determined in general accordance with ASTM D2041 (Rice Specific Gravity), or as specified by the City.

6.2 Option 2 – Full Removal & Replacement

6.2.1 Subgrade Preparation

Pavement subgrade preparation should be in conformance with Section 5.1.3 of this report. Pavement subgrade surfaces should be crowned (or sloped) for proper drainage in accordance with specifications provided by the project civil engineer.

6.2.2 Input Parameters

Design of the asphalt concrete (AC) pavement section presented below were based on the parameters presented in the following table, the American Association of State Highway and Transportation Officials (AASHTO) 1993 "Design of Pavement Structures" manual, and pavement design manuals presented by APAO and ODOT[6]. If any of the items listed need revision, please contact us and we will reassess the provided design sections.

6 Oregon Department of Transportation (ODOT) Pavement Design Guide, January 2019.

Table 3 Input Parameters Used in AC Pavement Design

Parameter	Value	Parameter		Value
Pavement Design Life (years)[1]	20	Resilient Modulus	Aggregate Base (ksi)[2]	20
Growth Rate (%)	0		Subgrade (ksi)[3]	8.2
Initial Serviceability[2]	4.2	Structural Coefficient	Asphalt[2]	0.42
Terminal Serviceability[2]	2.5		Aggregate Base[2]	0.10
Standard Deviation[2]	0.49	Vehicle Traffic[4]	APAO Level III (Moderate) (high end of this traffic level)	100,000 ESAL
Reliability[2] (%)	85			
Drainage Coefficient – Asphalt, Base, Subgrade[2]	1.0			

[1] Value based on AASHTO and APAO guidelines for most pavements of this type.
[2] Value based on guidelines presented by the referenced ODOT design manual for asphalt concrete pavements.
[3] Values based on DCP testing (summarized in Appendix B) and consideration for seasonal variations.
[4] ESAL = Total 18-Kip equivalent single axle load. Refer to Appendix C for additional discussion of value used for design.

6.2.3 Recommended Minimum Sections

The following table presents the minimum AC pavement section for the ESAL value indicated in the preceding table, based on the referenced AASHTO procedures.

Table 4 Minimum AC Pavement Section – Full Removal & Replacement

Material	APAO Traffic Loading Level III
Asphalt Pavement (inches)	4
Crushed Aggregate Base (inches)[1]	8
Subgrade Soils	Prepared in conformance with Section 5.1.3 of this report.

[1] Where present, the existing gravel fill may be suitable for use as crushed aggregate base below new pavements at the site, provided it is kept clean of fines and other deleterious materials during construction and exhibits proper gradation and other characteristics preferred for pavement aggregate base. Geotechnical observation, sampling, and laboratory testing of the gravel fill may be recommended following stripping of the existing AC pavement to confirm the existing material(s) exhibit those desirable characteristics.

6.2.4 AC Pavement Materials

We recommend pavement aggregate base consist of dense-graded aggregate in conformance with Section 02630.10 of the most recent ODOT SSC, with the following additional considerations. We recommend the material consist of crushed rock or gravel, have a maximum particle size of 1½ inches, and have less than 10 percent material passing the U.S. Standard No. 200 Sieve[7]. Aggregate base should be compacted to not less than 95 percent of the material's maximum dry density as determined in general accordance with AASHTO T180 (Modified Proctor), or as specified by City of Manzanita.

We recommend asphalt pavement consist of Level 2, ½-inch, dense-graded AC in conformance with the most recent ODOT SSC. Asphalt pavement should be compacted to at least 91 percent of the material's theoretical maximum density as determined in general accordance with ASTM D2041 (Rice Specific Gravity), or as specified by City of Manzanita.

[7] The recommendation to limit fines (e.g. silt or clay) within the base rock is intended to assist with moisture-conditioning and facilitating compaction of the layer, particularly if site work takes place during the traditional wet season in this region.

6.3 Option 3 – Full Depth Reclamation

6.3.1 Overview

Full depth reclamation (FDR) consists of reclaiming the pavement and aggregate base by mechanically breaking up the existing AC section and mixing that material with the underlying aggregate base. The reclaimed material is pulverized in-place to a specified gradation and compaction to serve as granular base for the new pavement. This new base course shall be mixed, proportioned, placed, and compacted in accordance with Section 6.3.4.1 of this report, or as specified by City of Manzanita

6.3.2 Input Parameters

Design of the AC pavement sections presented below were based on the parameters presented in the following table, the AASHTO 1993 "Design of Pavement Structures" manual, and pavement design manuals presented by APAO and ODOT. If any of the items listed need revision, please contact us and we will reassess the provided design sections.

Table 5 Input Parameters Used in AC Pavement Design

Parameter	Value	Parameter		Value
Pavement Design Life (years)[1]	20	Resilient Modulus	Reclaimed Agg. Base (ksi)[4]	15
Growth Rate (%)	0		Subgrade (ksi)[3]	8.2
Initial Serviceability[2]	4.2	Structural Coefficient	Asphalt[2]	0.42
Terminal Serviceability[2]	2.5		Reclaimed Agg. Base (ksi)[4]	0.08
Standard Deviation[2]	0.49	Vehicle Traffic[5]	APAO Level III (Moderate) (high end of this traffic level)	100,000 ESAL
Reliability[2] (%)	85			
Drainage Coefficient – Asphalt, Base, Subgrade[2]	1.0			

[1] Value based on AASHTO and APAO guidelines for most pavements of this type.
[2] Value based on guidelines presented by the referenced ODOT design manual for asphalt concrete pavements.
[3] Values based on DCP testing (summarized in Appendix B) and consideration for seasonal variations.
[4] Value based on examination of the existing aggregate base at boring locations.
[5] ESAL = Total 18-Kip equivalent single axle load. Refer to Appendix C for additional discussion of value used for design.

6.3.3 Recommended Minimum Section

The following table presents the minimum AC pavement section for the ESAL value indicated in the preceding table, based on the referenced AASHTO procedures.

Table 6 Minimum AC Pavement Sections – FDR

Material	APAO Traffic Loading Level III
Asphalt Pavement (inches)	4½
Reclaimed Base Material (inches)[1]	7

[1] Pulverized AC blended with underlying aggregate base. Prepared in general accordance with Section 6.3.4.1 below.

6.3.4 Pavement Materials

6.3.4.1 Reclaimed Base Material

The following is recommended for preparation of reclaimed pavement material:

- *Gradation:* Reclaimed material shall be pulverized to a maximum particle size of 3 inches in diameter, and have 100 percent and 95 to 100 percent of the material passing the U.S. Standard 3-inch and 1½-inch sieves, respectively. The processed reclaimed base material should contain no organic matter or debris, and have less than 10 percent material passing the U.S. Standard No. 200 Sieve.
- *Mix Design:* The mixed design is an approximation of existing site conditions and may be adjusted at the direction of the Project Engineer. The mixed design shall be as follows:
  - Minimum depth: 12 inches
  - Materials: Existing 2 inches of AC pavement and 10 inches of granular base
  - Density: Maximum dry density and optimum moisture content to be determined in accordance with AASHTO T180 (Modified Proctor).
- *Compaction:* The reclaimed material shall be moisture conditioned at or near optimum moisture content and compacted in accordance with Section 5.4.2 of this report (at least 95% AASHTO T180), or visual equivalent based on deflection (proof roll) testing per ODOT test method TM 158.

6.3.4.2 AC Pavement

We recommend asphalt pavement consist of Level 2, ½-inch, dense-graded AC in conformance with the most recent ODOT SSC. Asphalt pavement should be compacted to at least 91 percent of the material's theoretical maximum density as determined in general accordance with ASTM D2041 (Rice Specific Gravity), or as specified by City of Manzanita.

7.0 RECOMMENDATIONS: NEW RETAINING WALLS

As indicated above, we understand that site improvements will likely include construction of new retaining walls at the site. The location(s), type(s), and height(s) of the retaining walls are not known at this time. The following recommendations are presented for *preliminary* planning and design of new retaining walls at the site, including conventional cast-in-place (CIP) cantilevered retaining walls and pile-supported retaining walls (e.g. sheet pile walls, soldier pile walls, etc.). The geotechnical engineer or his representative should be contacted to provide supplemental recommendations for use in design and construction once the location(s), type(s), and height(s) of site retaining walls are known.

7.1 Option 1 – Conventional CIP Cantilevered Retaining Walls

7.1.1 Footings

7.1.1.1 Subgrade Preparation

Satisfactory subgrade support for retaining wall footings can be obtained from:

- The native sandy soils (SM, SP) provided the material is compacted using suitable equipment (e.g. vibratory hoe-pack compactor, vibrating plate compactor, etc.) until achieving a well-keyed (dense) condition. The geotechnical engineer or his representative should witness application of compaction effort to confirm suitable conditions.
- The native, medium stiff to better elastic silt (MH), or new structural fill that is properly placed and compacted on this material during construction.

The geotechnical engineer's representative should be contacted to observe subgrade conditions prior to placement of forms, reinforcement steel, or granular backfill (if required). If soft, loose, or otherwise unsuitable soils are encountered, they should be over-excavated as recommended by the geotechnical representative at the time of construction. The resulting over-excavation should be brought back to grade with imported granular structural fill in conformance with Section 5.4.2. The maximum particle size of over-excavation backfill should be limited to 1½ inches. All granular pads for footings should be constructed a minimum of 6 inches wider on each side of the footing for every vertical foot of over-excavation.

7.1.1.2 Minimum Footing Width & Embedment

We recommend continuous wall footings have a minimum width of 18 inches. All footings should be founded at least 18 inches below the lowest, permanent adjacent grade to develop lateral capacity and for frost protection.

7.1.1.3 Horizontal Setback from Descending Slopes

Foundations constructed within or near descending slopes exhibiting gradients up to 2H:1V (horizontal:vertical) should be setback a minimum of 5 feet from the slope surface. Foundations constructed within or near descending slopes exhibiting gradients between 2H:1V and 1½H:1V should be setback a minimum of 8 feet from the slope surface. These distances should be measured between the face of the slope and the bottom, outside edge of the respective foundation. Organic topsoil and loose surface soils (if present) should not be included when determining this distance. The geotechnical engineer or his representative should be contacted to observe foundation subgrade conditions and confirm this recommended minimum setback is achieved.

7.1.1.4 Bearing Pressure & Settlement

Footings founded as recommended above should be proportioned for a maximum allowable soil bearing pressure of 2,000 pounds per square foot (psf). This bearing pressure is a net bearing pressure, applies to the total of dead and long-term live loads, and may be increased by one-third when considering seismic or wind loads. For foundations founded as recommended above, total settlement of foundations is anticipated to be less than 1 inch. If an increased allowable soil bearing pressure is desired, the geotechnical engineer should be consulted.

7.1.1.5 Lateral Capacity

A maximum passive (equivalent fluid) earth pressure of 150 pounds per cubic foot (pcf) is recommended for design of footings cast neat into excavations in suitable native soil or confined by granular structural fill that is properly placed and compacted during construction. The recommended earth pressure was computed using a factor of safety of 1½, which is appropriate due to the amount of movement required to develop full passive resistance. In order to develop the above capacity, the following should be understood:

1. Concrete must be poured neat in excavations or the foundations must be backfilled with imported granular structural fill,
2. The adjacent grade must be level,
3. The static ground water level must remain below the base of the footings throughout the year.
4. Adjacent floor slabs, pavements, or the upper 12-inch-depth of adjacent, unpaved areas should not be considered when calculating passive resistance.

An ultimate coefficient of friction equal to 0.35 may be used when calculating resistance to sliding for footings founded on the native soils described above. An ultimate coefficient of friction equal to 0.45 may be used when calculating resistance to sliding for footings founded on a minimum of 6 inches of imported granular structural fill (crushed rock) that is properly placed and compacted during construction.

7.1.2 Wall Drains

We recommend placing retaining wall drains at the base elevation of the heel of retaining wall footings. Retaining wall drains should consist of a minimum 4-inch-diameter, perforated, HDPE (High Density Polyethylene) drainpipe wrapped with a non-woven geotextile filter fabric. The drains should be backfilled with a minimum of 2 cubic feet of open graded drain rock per lineal foot of pipe. The drain rock should be encased in a geotextile fabric in order to provide separation from the surrounding soils. Retaining wall drains should be positively sloped and should outlet to a suitable discharge point. The geotechnical engineer's representative should be contacted to observe the drains prior to backfilling. Roof or area drains should not be tied into retaining wall drains.

7.1.3 Wall Backfill

Retaining walls should be backfilled with imported granular structural fill in conformance with Section 5.4.2 and contain less than 5 percent passing the U.S. Standard No. 200 Sieve. The backfill should be compacted to a minimum of 90 percent of the material's maximum dry density as determined in general accordance with AASHTO T180 (Modified Proctor). When placing fill behind walls, care must be taken to minimize undue lateral loads on the walls. Heavy compaction equipment should be kept at least "H" feet from the back of the walls, where "H" is the height of the wall. Light mechanical or hand tamping equipment should be used for compaction of backfill materials within "H" feet of the back of the walls.

7.1.4 Design Parameters & Limitations

For rigid retaining walls founded, backfilled, and drained as recommended above, the following table presents parameters recommended for design.

Table 7 Design Parameters for Rigid Retaining Walls

Retaining Wall Condition	Modeled Backfill Condition	Static Equivalent Fluid Pressure (S_A)[1]	Seismic Equivalent Fluid Pressure (S_{AE})[1,2]	Surcharge from Uniform Load, q, Acting on Backfill Behind Retaining Wall
Not Restrained from Rotation	Level (i=0)	28 pcf	42 pcf	0.22*q
Restrained from Rotation	Level (i=0)	50 pcf	63 pcf	0.38*q

[1] Refer to the attached Figure 5 for a graphical representation of static and seismic loading conditions. Seismic resultant force acts at 0.6H above the base of the wall.

[2] Seismic (dynamic) lateral loads were computed using the Mononobe-Okabe Equation as presented in the 1997 Federal Highway Administration (FHWA) design manual. Static and seismic equivalent fluid pressures are not additive.

The above design recommendations are based on the assumptions that:

- The walls consist of concrete cantilevered retaining walls (β = 0 and δ = 24 degrees, see Figure 5).
- The walls are 10 feet or less in height.
- The backfill is drained and consists of imported granular structural fill (ϕ = 38 degrees).
- No point, line, or strip load surcharges are imposed behind the walls.

- The grade behind the wall is level, or sloping down and away from the wall, for a distance of 10 feet or more from the wall.
- The grade in front of the walls is level or ascending for a distance of at least 5 feet from the wall.

Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project vary from these assumptions.

7.1.5 Surcharge Loads

Where present, surcharges from adjacent site features (i.e. buildings, slabs, pavements, etc.) should be evaluated in design of retaining walls at the site. Methods for calculating lateral pressures on rigid retaining walls from strip, line, and vertical point loads are presented on the attached Figure 6.

7.2 Option 2 - Pile-Supported Retaining Walls

The following recommendations are presented for use in *preliminary* design of pile-supported retaining walls, including, but not limited to, sheet pile walls and soldier pile walls. Site subsurface conditions are conducive for installation of driven pile-supported walls, or placing steel piles in pre-drilled holes, if warranted[8]. The geotechnical engineer should be contacted to review the selected wall system(s) once plans have been prepared to capture the proposed location(s), height(s), and backfill considerations for those walls.

7.2.1 LPILE Parameters

We anticipate retaining wall design will be performed (by others) using commercially available, industry-standard software (such as LPILE™). We have provided recommended values for soil parameters for use in design using this method of analysis in the following table.

[8] Placing piles in pre-drilled holes may be advisable in the event the piles are to be installed on a relatively steep slope (due to vibration effects associated with pile driving) and/or in relative close proximity to existing residential structures (due to vibration effects and noise typically associated with pile driving).

Table 8 Recommended LPile™ Design Parameters

Layer	Depth (feet)	Description	IGM	LPile Soil Type	γ' (pcf)	Soil Properties						
						ϕ' (deg.)	c' (psf)	$S_{u(ave)}$ (psf)	K_p	k (pci)	ε_{50}	E_s (ksf)
1	0 to 2	Existing Fill Materials (neglect)	1	Sand (Reese)	130	0.01	0	0	0.01	0.01	--	0.1
2	2 to 15+	Loose to Med. Dense Sandy Soils (SM, SP)	1	Sand (Reese)	120	34	0	0	3.5	50	---	70

Notes: Variable Descriptions and Source Information	
Depth	The depths listed in this table are with respect to the existing ground surface at the project site and based on subsurface conditions encountered in borings B-1 and B-2. *Please refer to approved building plans (by others) for the location of the dredge line in front of each shoring wall.*
IGM	Idealized geomaterial. Layers were defined as idealized geomaterials in accordance with FHWA –NHI-10-016 (FHWA, 2010). A numbering system was used to represent the IGM in the table as follows: 1= Cohesionless Soil. 2= Cohesive Soil. 3= Rock. 4= Cohesive IGM.
LPile	LPILE soil model assigned consistent with idealized soil models in LPile 2016.9.09.
γ'	Effective unit weight. Values presented based on previous laboratory testing and local experience with similar soil types.
ϕ'	Internal angle of friction. Values presented are based Equation 3-8 (FHWA, 2010) and experience with similar soils in this region.
c'	Effective cohesion. All soils are modeled as cohesionless.
$S_{u(ave)}$	Averaged undrained shear strength of cohesive layer. All soils are modeled as cohesionless.
K_p	Passive lateral earth pressure coefficient, based on Equation 13-10 (FHWA, 2010).
k	P-y modulus. Values presented based on "Soil Modulus Parameter k Value" tables (for sands) in the Help Menu of LPILE 2016.9.09.
ε_{50}	Strain Factor for cohesive soils. All soils are modeled as cohesionless.
E_s	Young's modulus for soil (E_s). Value presented based on Table 3-6 (FHWA, 2010) – SPT correlations (for cohesionless soils) and the average value within the soil profile.

We recommend a geotechnical plans review of the drilled pier design be performed to confirm the recommendations presented within this section are implemented as intended.

7.2.2 Retained Soils

The following table presents soil strength parameters recommended for modeling the retained soils behind the pile-supported retaining walls (i.e., above the dredge line). The parameters presented therein were based on the results of the laboratory testing performed on selected samples, published correlations with SPT N-values, and experience with similar soils.

Table 9 Soil Parameters Recommended for Retained Soils (Above Dredge Line)

Parameter[1]	Subsurface Material[2]	
	Existing Fill Materials (GP Fill)	Loose to Med. Dense Native Sandy Soils (SM, SP)
Effective Unit Weight, γ'	130 pcf	120 pcf
Internal Angle of Friction, ϕ'	38°	34°
Effective Cohesion, c'	0 psf	0 psf
Ultimate Coefficient of Active Pressure, K_a	0.24	0.28
Ultimate Coefficient of At Rest Pressure, K_o	0.38	0.44

[1] If additional soil parameters are required for design, the geotechnical engineer should be consulted.

[2] Refer to the attached boring logs (Appendix C) for layer thicknesses across the site.

7.2.3 Surcharges (if present)

Where present, surcharges from adjacent site features (i.e. buildings, slabs, pavements, etc.) should be evaluated in design of retaining walls at the site. Where uniform (area-wide) load(s) are present behind the walls (i.e., at the ground surface), we recommend the lateral pressure(s) be modeled as a rectangular distribution behind the wall and assigned equal to q * 0.30, where q is equal to the surcharge load in units of psf. This assumes the soldier piles are allowed to rotate some at the top, allowing for development of active pressures. Methods for calculating lateral pressures retaining walls from strip, line, and vertical point loads are presented on the attached Figure 6. Surcharge pressures, if present, should be added to those associated with lateral earth pressures calculated from the earthen soils behind the walls using the principle of superposition.

8.0 RECOMMENDED ADDITIONAL SERVICES

8.1 Design Review

Geotechnical design review is of paramount importance. We recommend the geotechnical design review take place prior to releasing bid packets to contractors.

8.2 Observation of Construction

Satisfactory earthwork, foundation, retaining wall, and pavement performance depends to a large degree on the quality of construction. Sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Subsurface conditions observed during construction should be compared with those encountered during subsurface explorations, and recognition of changed conditions often requires experience. We recommend that qualified personnel visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those observed to date and anticipated in this report. We recommend the geotechnical engineer or their representative attend a pre-construction meeting coordinated by the contractor and/or developer. The project geotechnical engineer or their representative should provide observations and/or testing of at least the following earthwork elements during construction:

- Site stripping and demolition
- Subgrade preparation for structural fills, retaining walls, and pavements
- Compaction of structural fill and utility trench backfill
- Compaction of base rock for pavements
- Compaction of asphalt concrete for pavements

It is imperative that the owner and/or contractor request earthwork observations and testing at a frequency sufficient to allow the geotechnical engineer to provide a final letter of compliance for the earthwork activities.

9.0 LIMITATIONS

At our client's request, the scope of our evaluation was limited to the scope of services described in this report. Other geotechnical considerations described in the 2022 Oregon Structural Specialty Code (OSSC) have not been addressed. Accordingly, this evaluation must be considered "limited." A more comprehensive evaluation may be completed if requested by our client, for an additional fee. Such evaluation would include, but not be limited to assessment of seismic/geologic hazards at the site, recommendations for seismic design criteria, and other geotechnical considerations. The responsibility for determining the sufficiency of our evaluation to meet the project needs rests solely with the owner and not with CGT. Please contact us if additional evaluation is desired.

We have prepared this report for use by the City of Manzanita and other members of the design and construction team for the proposed development. The opinions and recommendations contained within this report are forwarded to assist in the planning and design process and are not intended to be, nor should they be construed as, a warranty of subsurface conditions.

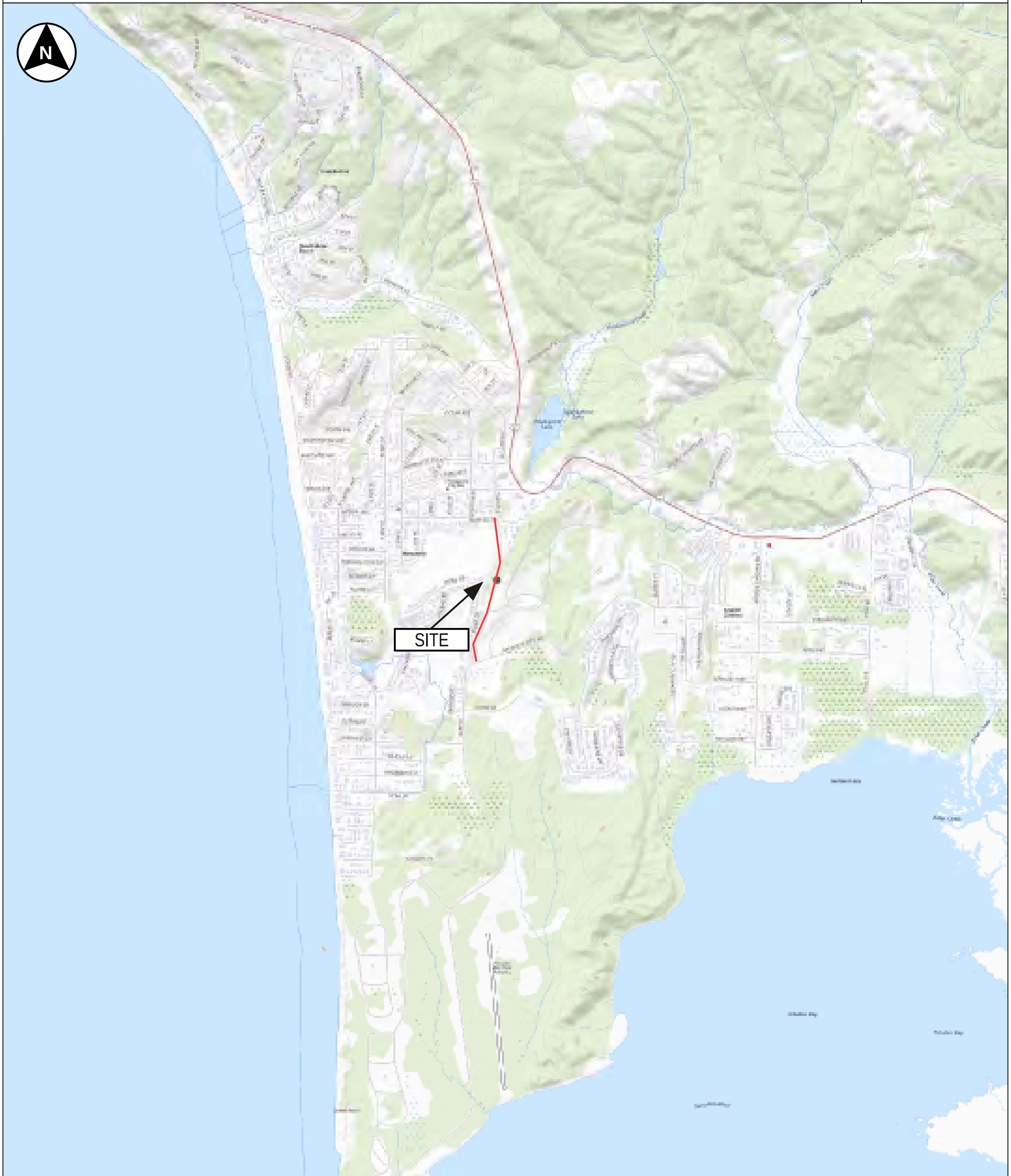
We have made observations based on our explorations that indicate the soil conditions at only those specific locations and only to the depths penetrated. These observations do not necessarily reflect soil types, strata thickness, or water level variations that may exist between or away from our explorations. If subsurface conditions vary from those encountered in our site explorations, CGT should be alerted to the change in conditions so that we may provide additional geotechnical recommendations, if necessary. Observation by experienced geotechnical personnel should be considered an integral part of the construction process.

The owner/developer is responsible for ensuring that the project designers and contractors implement our recommendations. When the design has been finalized, prior to releasing bid packets to contractors, we recommend that the design drawings and specifications be reviewed by our firm to see that our recommendations have been interpreted and implemented as intended. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written modification or verification. Design review and construction phase testing and observation services are beyond the scope of our current assignment, but will be provided for an additional fee.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design.

Geotechnical engineering and the geologic sciences are characterized by a degree of uncertainty. Professional judgments presented in this report are based on our understanding of the proposed construction, familiarity with similar projects in the area, and on general experience. Within the limitations of scope, schedule, and budget, our services have been executed in accordance with the generally accepted practices in this area at the time this report was prepared; no warranty, expressed or implied, is made. This report is subject to review and should not be relied upon after a period of three years.

CLASSIC STREET IMPROVEMENTS - MANZANITA, OREGON *Project Number G2406158*	**FIGURE 1** **Site Location**



CARLSON CGT GEOTECHNICAL 503-601-8250 Drafted by: MDI	USGS Topographic base map created with The National Map, 2024, at https://viewer.nationalmap.gov/advanced-viewer/ Township 3 North, Range 10 West, Section 29, Willamette Meridian	Latitude: 45.71563° North Longitude: 123.929562° West 1 Inch = 2,000 feet 0 2000 4000

CLASSIC STREET IMPROVEMENTS - MANZANITA, OREGON Project Number G2406158	FIGURE 2 Site Plan

LEGEND

B-1/ DCP-1 Drilled boring & dynamic cone penetrometer test.

1 Orientation of site photographs shown on Figure 3.

Approximate pavement area exhibiting uneven conditions along outer (west) portion. See Appendix C for additional discussion.

CARLSON CGT GEOTECHNICAL 503-601-8250

Drafted by: EEH/bmw

NOTES: Drawing based on observations made while on site. 2023 aerial image from ArcGIS (www.arcgis.com). All locations are approximate.

1 Inch = 250 Feet

0 250 500

Photograph 1

Photograph 2

Photograph 3

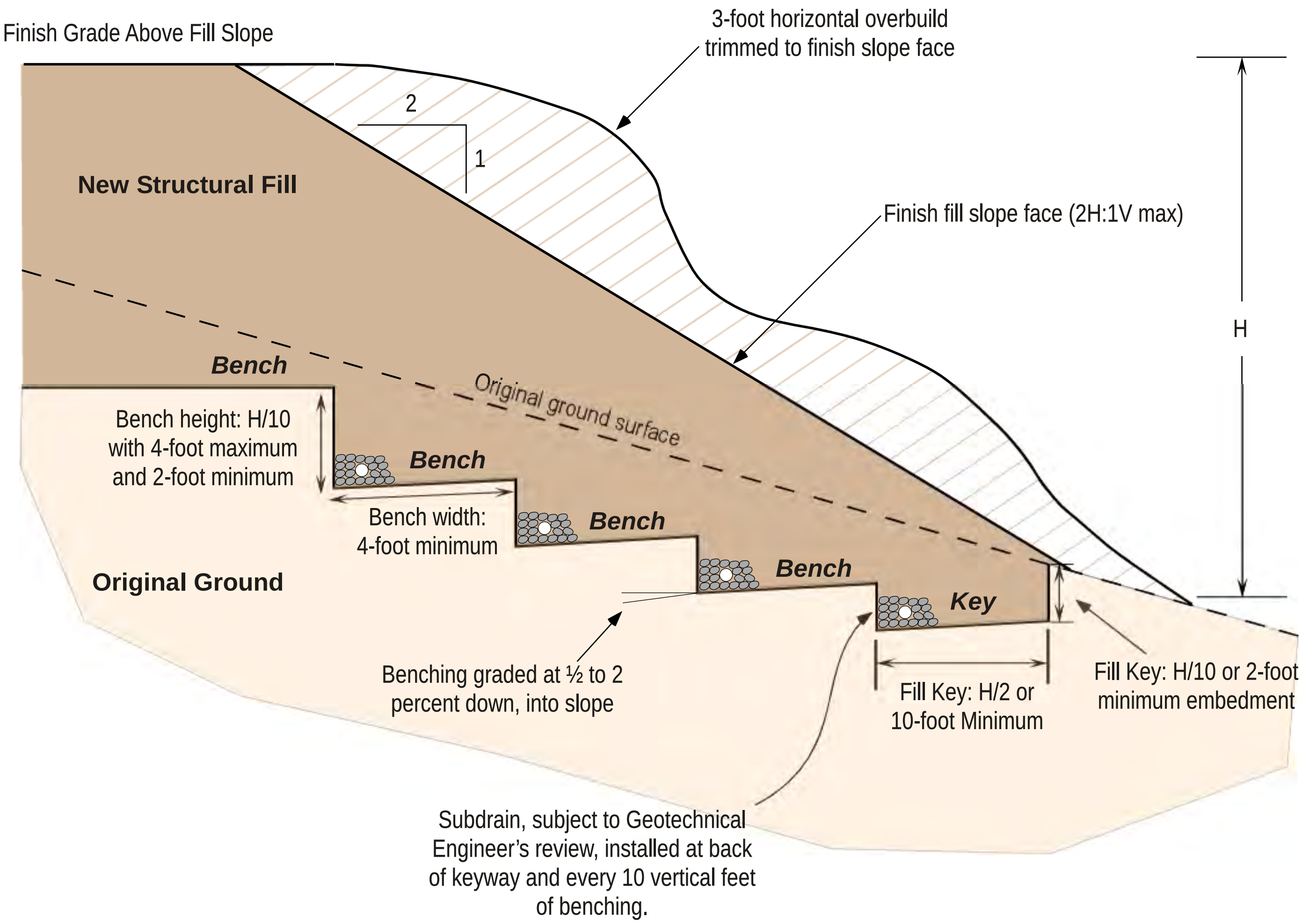
Photograph 4

See Figure 2 for approximate photograph locations and directions. Photographs were taken at the time of our fieldwork.

CLASSIC STREET IMPROVEMENTS - MANZANITA, OREGON
Project Number G2406158

FIGURE 4

Fill Slope Detail



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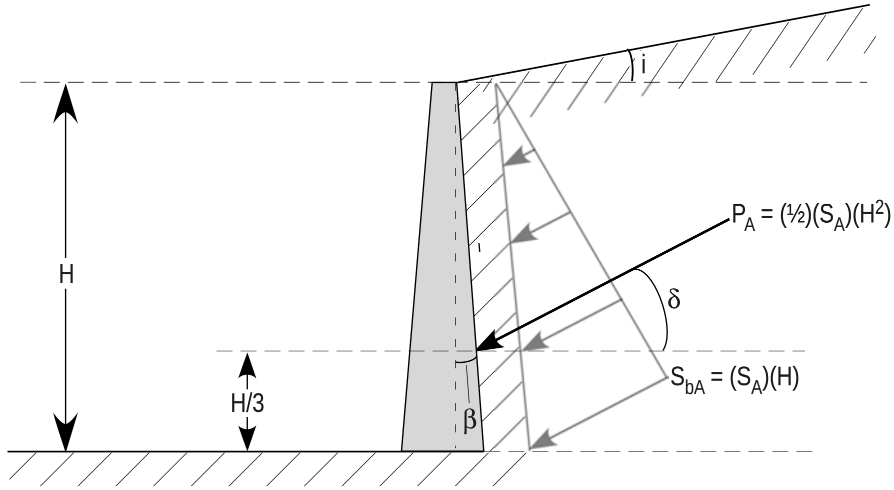
NOTE: Surfaces to receive fill with slopes steeper than 5H:1V (horizontal:vertical) should be benched and keyed as shown.

CLASSIC STREET IMPROVEMENTS - MANZANITA, OREGON
Project Number G2406158

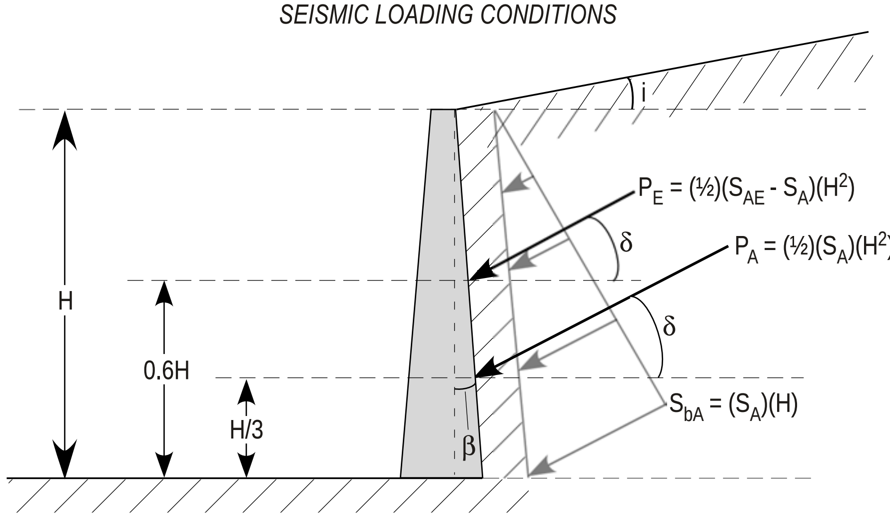
FIGURE 5
Retaining Walls

ACTIVE LATERAL PRESSURE DISTRIBUTION

STATIC LOADING CONDITIONS



SEISMIC LOADING CONDITIONS



LEGEND

S_A = Active lateral equivalent fluid pressure (lb/ft^3)*

S_{bA} = Active lateral earth pressure (static) at the bottom of wall (lb/ft^3)

S_{AE} = Active total (static + seismic) equivalent fluid pressure (lb/ft^3)*

i = Slope of backfill, relative to horizontal (degrees)**

β = Slope of back of wall, relative to vertical (degrees)**

P_A = Static active thrust force acting at H/3 from bottom of retaining wall (lb/ft)

P_E = Dynamic active thrust force acting at 0.6H from bottom of retaining wall (lb/ft)

δ = Angle from normal of back of wall (degrees). Based on friction developing between wall and backfill**

*Refer to report text for calculated values **Refer to report text for modeled/assumed values

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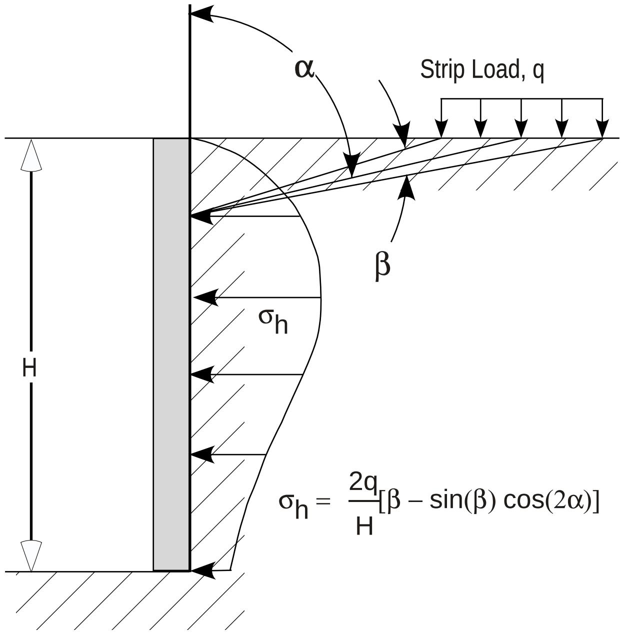
Notes

1. Uniform pressure distribution of seismic loading is based on empirical evaluations [Sherif et al, 1982 and Whitman, 1990].
2. Placement of seismic resultant force at 0.6H is based on wall behavior and model test results [Whitman, 1990].

FIGURE 6

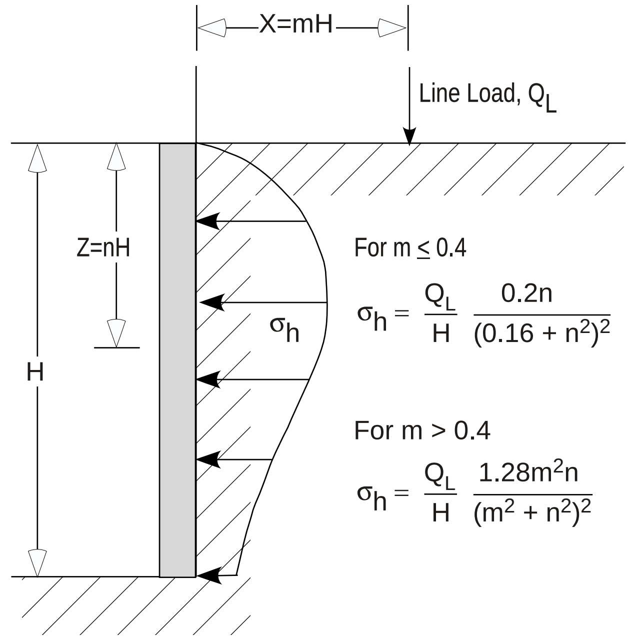
Retaining Wall Surcharge

STRIP LOAD PARALLEL TO WALL[1]



$$\sigma_h = \frac{2q}{H}[\beta - \sin(\beta)\cos(2\alpha)]$$

LINE LOAD PARALLEL TO WALL[2]



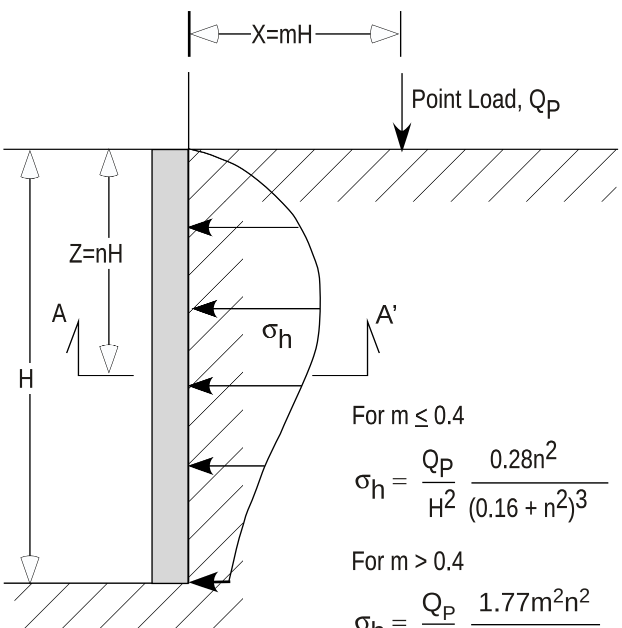
For $m \leq 0.4$

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

For $m > 0.4$

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

VERTICAL POINT LOAD[2]



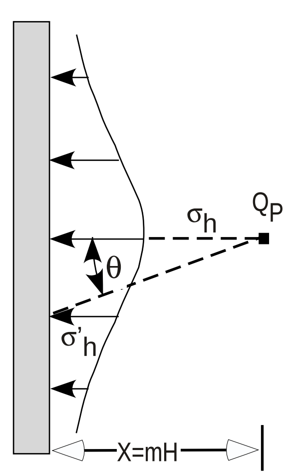
For $m \leq 0.4$

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

For $m > 0.4$

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

Section A - A'



$$\sigma'_h = \sigma_h \cos^2(1.1\,\theta)$$

Notes: 1. Das, Principles of Geotechnical Engineering, 1990 Edition.
2. NAVFAC Design Manual 7.06.

Refer to the referenced design manuals for additional guidance. Contact CGT if there are any questions with modeling surcharge loads.

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Tigard Office	**(503) 684-3460**

Appendix A: Subsurface Investigation and Laboratory Testing

Classic Street Improvements
Classic Street
Manzanita, Oregon

CGT Project Number G2406158

August 16, 2024

Prepared For:

City of Manzanita
Dan Weitzel, Public Works Director
1090 Oak Street
Manzanita, Oregon 97130

Prepared by
Carlson Geotechnical

A.1.0 SUBSURFACE INVESTIGATION

Our field investigation consisted of six drilled borings completed on July 8, 2024. The exploration locations are shown on the Site Plan, attached to the geotechnical report as Figure 2. The exploration locations shown therein were determined based on measurements from existing off-site features (connecting roadways, buildings, etc.) and are approximate. Surface elevations indicated on the logs were estimated based on the topographic contours (by others) shown on schematic plans provided by our client, and are approximate. The attached figures detail the exploration methods (Figure A1), soil classification criteria (Figure A2), and present detailed logs of the explorations (Figures A3 through A8), as discussed below.

A.1.1 Drilled Borings

CGT observed the advancement of six drilled borings (B-1 through B-6) at the site using a B58 truck-mounted drill rig provided and operated by our subcontractor, PLI Systems of Hillsboro, Oregon. The borings were advanced using the hollow-stem auger drilling technique to depths of about 11½ feet below pavement surface (bps). Upon completion, the borings were backfilled with granular bentonite and the surfaces were patched with cold patch asphalt.

A.1.2 In-Situ Testing

A.1.2.1 <u>Dynamic Cone Penetrometer (DCP) Testing</u>

In each drilled boring, we performed a dynamic cone penetrometer (DCP) test. The DCP tests (DCP-1 through DCP-6) were conducted on the exposed subgrade below the pavement materials to depths up to about 3 feet bps. DCP testing was performed in general accordance with ASTM D6951, and consists of driving a 20-mm diameter, hardened steel cone on 16-mm diameter steel rods into the ground using a 8-kg drop hammer with a 460-mm, free-fall height. The number of hammer blows required to drive the DCP tip is typically recorded in 10-mm increments. The DCP index (defined as the amount of penetration per blow) is calculated by dividing the incremental penetration by the number of blows. The DCP index can be correlated to subgrade resilient modulus (M_R)[1]. Results of the DCP tests, including the DCP index and correlated resilient modulus values, are presented in the attached Appendix B.

A.1.2.2 <u>Standard Penetration Tests (SPTs)</u>

SPTs were conducted within the drilled borings using a split-spoon sampler in general accordance with ASTM D1586. The SPTs were conducted at 2½-foot intervals to the termination depths of the borings. The SPT is described on the attached Exploration Key, Figure A1.

A.1.3 Material Classification & Sampling

Soil samples were obtained at selected intervals in the borings using the referenced split-spoon (SPT) sampler and thin-walled, steel (Shelby) tube samplers, detailed on Figure A1. A qualified member of CGT's geological staff collected the samples and logged the soils in general accordance with the Visual-Manual Procedure (ASTM D2488). An explanation of this classification system is attached as Figure A2. The SPT samples were stored in sealable plastic bags and transported to our soils laboratory for further examination and testing. Our geotechnical staff visually examined all samples in order to refine the initial field classifications.

[1] Oregon Department of Transportation (ODOT) Pavement Services Unit, January 2019.

A.1.4 Subsurface Conditions

Subsurface conditions are summarized in Section 2.3 of the geotechnical report. Detailed logs of the explorations are presented on the attached exploration logs, Figures A3 through A8.

A.2.0 LABORATORY TESTING

Laboratory testing was performed on samples collected in the field to refine our initial field classifications and determine in-situ parameters. Laboratory testing included the following:

- Eight moisture content determinations (ASTM D2216).
- Two percentage passing the U.S. Standard No. 200 Sieve tests (ASTM D1140).
- One Atterberg limits (plasticity) test (ASTM D4318).

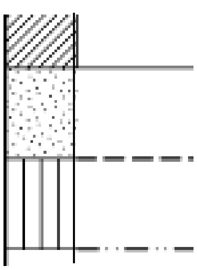
Results of the laboratory tests are shown on the exploration logs.

PL ●MC LL	Atterberg limits (plasticity) test results (ASTM D4318): PL = Plastic Limit, LL = Liquid Limit, and MC= Moisture Content (ASTM D2216)
☐ FINES CONTENT (%)	Percentage passing the U.S. Standard No. 200 Sieve (ASTM D1140)

SAMPLING

GRAB	Grab sample
BULK	Bulk sample
SPT	**Standard Penetration Test** (SPT) consists of driving a 2-inch, outside-diameter, split-spoon sampler into the undisturbed formation with repeated blows of a 140-pound, hammer falling a vertical distance of 30 inches (ASTM D1586). The number of blows (N-value) required to drive the sampler the last 12 inches of an 18-inch sample interval is used to characterize the soil consistency or relative density. The drill rig was equipped with an cat-head or automatic hammer to conduct the SPTs. The observed N-values, hammer efficiency, and N_{60} are noted on the boring logs.
MC	**Modified California** sampling consists of 3-inch, outside-diameter, split-spoon sampler (ASTM G3550) driven similarly to the SPT sampling method described above. A sampler diameter correction factor of 0.44 is applied to calculate the equivalent SPT N_{60} value per Lacroix and Horn, 1973.
CORE	**Rock Coring** interval
SH	**Shelby Tube** is a 3-inch, inner-diameter, thin-walled, steel tube push sampler (ASTM D1587) used to collect relatively undisturbed samples of fine-grained soils.
WDCP	**Wildcat Dynamic Cone Penetrometer** (WDCP) test consists of driving 1.1-inch diameter, steel rods with a 1.4-inch diameter, cone tip into the ground using a 35-pound drop hammer with a 15-inch free-fall height. The number of blows required to drive the steel rods is recorded for each 10 centimeters (3.94 inches) of penetration. The blow count for each interval is then converted to the corresponding SPT N_{60} values.
DCP	**Dynamic Cone Penetrometer** (DCP) test consists of driving a 20-millimeter diameter, hardened steel cone on 16-millimeter diameter steel rods into the ground using a 10-kilogram drop hammer with a 460-millimeter free-fall height. The depth of penetration in millimeters is recorded for each drop of the hammer.
POCKET PEN. (tsf)	**Pocket Penetrometer** test is a hand-held instrument that provides an approximation of the unconfined compressive strength in tons per square foot (tsf) of cohesive, fine-grained soils.

CONTACTS



Observed (measured) contact between soil or rock units.

Inferred (approximate) contact between soil or rock units.

Transitional (gradational) contact between soil or rock units.

ADDITIONAL NOTATIONS

Italics	Notes drilling action or digging effort
{ Braces }	Interpretation of material origin/geologic formation (e.g. { Base Rock } or { Columbia River Basalt })

CLASSIC STREET IMPROVEMENTS - MANZANITA, OREGON
Project Number G2406158

FIGURE A2

Soil Classification

Classification of Terms and Content

NAME: Group Name and Symbol
Relative Density or Consistency
Color
Moisture Content
Plasticity
Other Constituents
Other: Grain Shape, Approximate Gradation
Organics, Cement, Structure, Odor, etc.
Geologic Name or Formation

Grain Size

Grain Size		U.S. Standard Sieve
Fines		<#200 (0.075 mm)
Sand	Fine	#200 - #40 (0.425 mm)
	Medium	#40 - #10 (2 mm)
	Coarse	#10 - #4 (4.75 mm)
Gravel	Fine	#4 - 0.75 inch
	Coarse	0.75 inch - 3 inches
Cobbles		3 to 12 inches
Boulders		> 12 inches

Coarse-Grained (Granular) Soils

Relative Density

SPT N_{60}-Value	Density
0 - 4	Very Loose
4 - 10	Loose
10 - 30	Medium Dense
30 - 50	Dense
>50	Very Dense

Minor Constituents

Percent by Volume	Descriptor	Example
0 - 5%	"Trace" as part of soil description	"trace silt"
5 - 15%	"With" as part of group name	"**POORLY GRADED SAND WITH SILT**"
15 - 49%	Modifier to group name	"**SILTY SAND**"

Fine-Grained (Cohesive) Soils

SPT N_{60}-Value	Torvane tsf Shear Strength	Pocket Pen tsf Unconfined	Consistency	Manual Penetration Test
<2	<0.13	<0.25	Very Soft	Thumb penetrates more than 1 inch
2 - 4	0.13 - 0.25	0.25 - 0.50	Soft	Thumb penetrates about 1 inch
4 - 8	0.25 - 0.50	0.50 - 1.00	Medium Stiff	Thumb penetrates about ¼ inch
8 - 15	0.50 - 1.00	1.00 - 2.00	Stiff	Thumb penetrates less than ¼ inch
15 - 30	1.00 - 2.00	2.00 - 4.00	Very Stiff	Readily indented by thumbnail
>30	>2.00	>4.00	Hard	Difficult to indent by thumbnail

Minor Constituents

Percent by Volume	Descriptor	Example
0 - 5%	"Trace" as part of soil description	"trace fine-grained sand"
5 - 15%	"Some" as part of soil description	"some fine-grained sand"
15 - 30%	"With" as part of group name	"**SILT WITH SAND**"
30 - 49%	Modifier to group name	"**SANDY SILT**"

Moisture Content

Dry: Absence of moisture, dusty, dry to the touch

Moist: Leaves moisture on hand

Wet: Visible free water, likely from below water table

	Plasticity	Dry Strength	Dilatancy	Toughness
ML	Non to Low	Non to Low	Slow to Rapid	Low, can't roll
CL	Low to Medium	Medium to High	None to Slow	Medium
MH	Medium to High	Low to Medium	None to Slow	Low to Medium
CH	Medium to High	High to Very High	None	High

Structure

Stratified: Alternating layers of material or color >6 mm thick

Laminated: Alternating layers < 6 mm thick

Fissured: Breaks along definite fracture planes

Slickensided: Striated, polished, or glossy fracture planes

Blocky: Cohesive soil that can be broken down into small angular lumps which resist further breakdown

Lenses: Has small pockets of different soils, note thickness

Homogeneous: Same color and appearance throughout

Visual-Manual Classification

Major Divisions			Group Symbols	Typical Names
Coarse Grained Soils: More than 50% retained on No. 200 sieve	Gravels: 50% or more *retained* on the No. 4 sieve	Clean Gravels	GW	Well-graded gravels and gravel/sand mixtures, little or no fines
			GP	Poorly-graded gravels and gravel/sand mixtures, little or no fines
		Gravels with Fines	GM	Silty gravels, gravel/sand/silt mixtures
			GC	Clayey gravels, gravel/sand/clay mixtures
	Sands: More than 50% *passing* the No. 4 sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly-graded sands and gravelly sands, little or no fines
		Sands with Fines	SM	Silty sands, sand/silt mixtures
			SC	Clayey sands, sand/clay mixtures
Fine-Grained Soils: 50% or more Passes No. 200 Sieve	Silt and Clays Low Plasticity Fines		ML	Inorganic silts, rock flour, clayey silts
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
			OL	Organic soil of low plasticity
	Silt and Clays High Plasticity Fines		MH	Inorganic silts, clayey silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic soil of medium to high plasticity
Highly Organic Soils			PT	Peat, muck, and other highly organic soils

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References:

ASTM D2487 Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)

ASTM D2488 Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)

Terzaghi, K., and Peck, R.B., 1948, Soil Mechanics in Engineering Practice, John Wiley & Sons.

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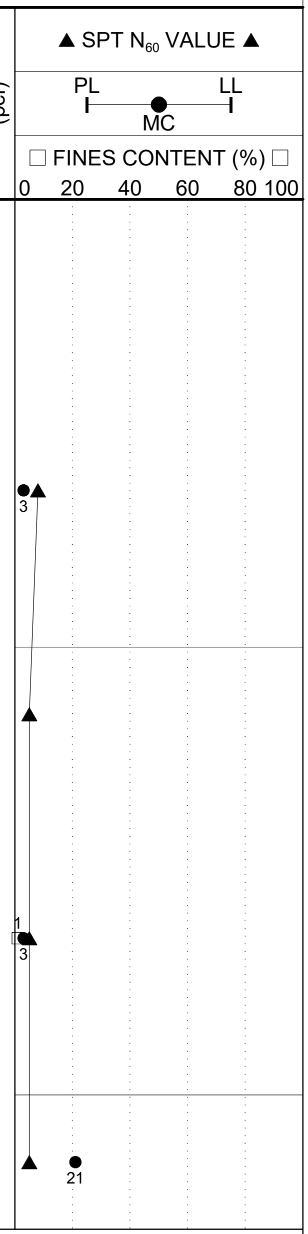
FIGURE A3

Boring B-1

CLIENT City of Manzanita - Dan Weitzel, Public Works Director
PROJECT NAME Classic Street Improvements
PROJECT NUMBER G2406158
PROJECT LOCATION Classic Street - Manzanita, Oregon
DATE STARTED 7/8/24 **GROUND ELEVATION** 52 ft
ELEVATION DATUM From schematic plans provided by client.
WEATHER Sunny, 78F **SURFACE** Asphalt Concrete
LOGGED BY BJG **REVIEWED BY** BMW
DRILLING CONTRACTOR PLI Systems, Inc.
SEEPAGE ---
EQUIPMENT Mobile B-57 Truck
GROUNDWATER DURING DRILLING 10.0 ft / El. 42.0 ft
DRILLING METHOD Hollow Stem Auger & DCP
GROUNDWATER AFTER DRILLING ---

ELEVATION (ft)	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N_{SPT} VALUE)	N_{60} VALUE ETR_{Hammer} = 77.70%	DRY UNIT WT. (pcf)
52		**ASPHALT CONCRETE:** Approximately 2 inches thick.		0					
	GP FILL	**POORLY GRADED GRAVEL FILL:** Brown, dry, angular, up to ¼-inch in diameter.							
50	SP	**POORLY GRADED SAND:** Loose, tan, moist, fine- to medium-grained, with trace low plasticity fines.							
					SPT 1	56	3-4-4 (8)	8	
				5	SPT 2	44	1-2-3 (5)	5	
45		Increased moisture content below 7 feet bgs.							
					SPT 3	56	1-1-4 (5)	5	
		Wet below 10 feet bgs.	∇	10					
					SPT 4	56	1-1-4 (5)	5	
40									



• Boring terminated at about 11½ feet bgs.
• Groundwater encountered at about 10 feet bgs.
• No caving encountered.
• Boring backfilled with crushed rock and surface patched with cold patch asphalt.

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FIGURE A4

Boring B-2

CLIENT City of Manzanita - Dan Weitzel, Public Works Director
PROJECT NAME Classic Street Improvements
PROJECT NUMBER G2406158
PROJECT LOCATION Classic Street - Manzanita, Oregon
DATE STARTED 7/8/24
GROUND ELEVATION 80 ft
ELEVATION DATUM From schematic plans provided by client.
WEATHER Sunny, 78F
SURFACE Asphalt Concrete
LOGGED BY BJG
REVIEWED BY BMW
DRILLING CONTRACTOR PLI Systems, Inc.
SEEPAGE ---
EQUIPMENT Mobile B-57 Truck
GROUNDWATER DURING DRILLING ---
DRILLING METHOD Hollow Stem Auger & DCP
GROUNDWATER AFTER DRILLING ---

ELEVATION (ft)	GROUP SYMBOL	MATERIAL DESCRIPTION	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N_{SPT} VALUE)	N_{60} VALUE ETR_{Hammer} = 77.70%	DRY UNIT WT. (pcf)
80		**ASPHALT CONCRETE:** Approximately 2 inches thick.	0					
	GP FILL	**POORLY GRADED GRAVEL FILL:** Brown, dry, angular, up to ¾-inch in diameter, with low plasticity fines.						
	SM	**SILTY SAND:** Medium dense, tan, moist, fine- to medium-grained, with low plasticity fines.		SPT 1	56	7-8-10 (18)	17	
75	SP	**POORLY GRADED SAND:** Loose, tan, moist, fine- to medium-grained, with trace low plasticity fines.	5	SPT 2	56	2-2-3 (5)	5	
				SPT 3	56	1-2-3 (5)	5	
70			10	SPT 4	56	1-2-2 (4)	4	
65								

▲ SPT N_{60} VALUE ▲ — PL / MC / LL — ☐ FINES CONTENT (%) ☐ (0 20 40 60 80 100)

- Boring terminated at about 11½ feet bgs.
- No groundwater or caving encountered.
- Boring backfilled with crushed rock and surface patched with cold patch asphalt.

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FIGURE A5

Boring B-3

CLIENT City of Manzanita - Dan Weitzel, Public Works Director
PROJECT NAME Classic Street Improvements
PROJECT NUMBER G2406158
PROJECT LOCATION Classic Street - Manzanita, Oregon
DATE STARTED 7/8/24 **GROUND ELEVATION** 82 ft
ELEVATION DATUM From schematic plans provided by client.
WEATHER Sunny, 78F **SURFACE** Asphalt Concrete
LOGGED BY BJG **REVIEWED BY** BMW
DRILLING CONTRACTOR PLI Systems, Inc.
SEEPAGE ---
EQUIPMENT Mobile B-57 Truck
GROUNDWATER DURING DRILLING ---
DRILLING METHOD Hollow Stem Auger & DCP
GROUNDWATER AFTER DRILLING ---

ELEVATION (ft)	GROUP SYMBOL	MATERIAL DESCRIPTION	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N_{SPT} VALUE)	N_{60} VALUE ETR_{Hammer} = 77.70%	DRY UNIT WT. (pcf)
		ASPHALT CONCRETE: Approximately 2 inches thick.	0					
80	GP FILL	**POORLY GRADED GRAVEL FILL:** Brown, dry, angular, up to ¾-inch in diameter, with low plasticity fines.						
	SM	**SILTY SAND:** Medium dense, tan, moist, fine- to medium-grained, with low plasticity fines.		SPT 1	56	2-6-7 (13)	13	
	SP	**POORLY GRADED SAND:** Loose, tan, moist, fine- to medium-grained, with trace low plasticity fines.	5	SPT 2	67	1-2-2 (4)	4	
75		Very loose, with trace gray mottling below 7½ feet bgs.		SPT 3	67	1-1-1 (2)	2	
		Loose below about 10 feet bgs.	10	SPT 4	67	1-2-3 (5)	5	
70								

▲ SPT N_{60} VALUE ▲
PL — MC — LL
☐ FINES CONTENT (%) ☐
0 20 40 60 80 100

- Boring terminated at about 11½ feet bgs.
- No groundwater or caving encountered.
- Boring backfilled with crushed rock and surface patched with cold patch asphalt.

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FIGURE A6

Boring B-4

CLIENT City of Manzanita - Dan Weitzel, Public Works Director
PROJECT NAME Classic Street Improvements
PROJECT NUMBER G2406158
PROJECT LOCATION Classic Street - Manzanita, Oregon
DATE STARTED 7/8/24
GROUND ELEVATION 84 ft
ELEVATION DATUM From schematic plans provided by client.
WEATHER Sunny, 78F
SURFACE Asphalt Concrete
LOGGED BY BJG
REVIEWED BY BMW
DRILLING CONTRACTOR PLI Systems, Inc.
SEEPAGE ---
EQUIPMENT Mobile B-57 Truck
GROUNDWATER DURING DRILLING ---
DRILLING METHOD Hollow Stem Auger & DCP
GROUNDWATER AFTER DRILLING ---

GROUP SYMBOL	MATERIAL DESCRIPTION
	ASPHALT CONCRETE: Approximately 2 inches thick.
GP FILL	**POORLY GRADED GRAVEL FILL:** Brown, dry, nonplastic, angular, up to ¾-inch in diameter, with low plasticity fines.
SP	**POORLY GRADED SAND:** Loose, tan, moist, fine- to medium-grained, with trace low plasticity fines.

SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N_{SPT} VALUE)	N_{60} VALUE ETR_{Hammer} = 77.70%	DRY UNIT WT. (pcf)	MC (%)
SPT 1	56	1-1-2 (3)	3		7
SPT 2	67	0-1-1 (2)	2		
SPT 3	67	0-1-1 (2)	2		
SPT 4	67	0-1-1 (2)	2		

▲ SPT N_{60} VALUE ▲ — PL, MC, LL — ☐ FINES CONTENT (%) ☐ — 0 20 40 60 80 100

ELEVATION (ft): 80, 75, 70 — DEPTH (ft): 0, 5, 10

- Boring terminated at about 11½ feet bgs.
- No groundwater or caving encountered.
- Boring backfilled with crushed rock and surface patched with cold patch asphalt.

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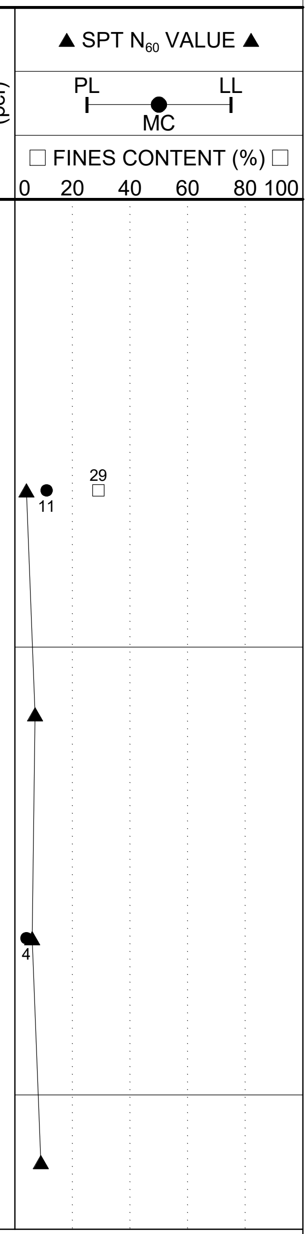
FIGURE A7
Boring B-5

CLIENT	City of Manzanita - Dan Weitzel, Public Works Director	**PROJECT NAME**	Classic Street Improvements
PROJECT NUMBER	G2406158	**PROJECT LOCATION**	Classic Street - Manzanita, Oregon
DATE STARTED	7/8/24	**GROUND ELEVATION**	76 ft
ELEVATION DATUM	From schematic plans provided by client.		
WEATHER	Sunny, 78F	**SURFACE**	Asphalt Concrete
LOGGED BY	BJG	**REVIEWED BY**	BMW
DRILLING CONTRACTOR	PLI Systems, Inc.	**SEEPAGE**	---
EQUIPMENT	Mobile B-57 Truck	**GROUNDWATER DURING DRILLING**	---
DRILLING METHOD	Hollow Stem Auger & DCP	**GROUNDWATER AFTER DRILLING**	---

ELEVATION (ft)	GROUP SYMBOL	MATERIAL DESCRIPTION	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N_{SPT} VALUE)	N_{60} VALUE ETR_{Hammer} = 77.70%	DRY UNIT WT. (pcf)
		ASPHALT CONCRETE: Approximately 2 inches thick.	0					
75	GP FILL	**POORLY GRADED GRAVEL FILL:** Brown, dry, angular, up to ¾-inch in diameter, with low plasticity fines.						
	SM	**SILTY SAND:** Loose, tan with orange mottling, moist, fine- to medium-grained, with low plasticity fines.		SPT 1	33	3-3-1 (4)	4	
70	SP	**POORLY GRADED SAND:** Loose, tan, moist, fine- to medium-grained, with trace low plasticity fines.	5	SPT 2	67	1-3-4 (7)	7	
				SPT 3	67	1-2-4 (6)	6	
65			10	SPT 4	67	3-4-5 (9)	9	



- Boring terminated at about 11½ feet bgs.
- No groundwater or caving encountered.
- Boring backfilled with crushed rock and surface patched with cold patch asphalt.

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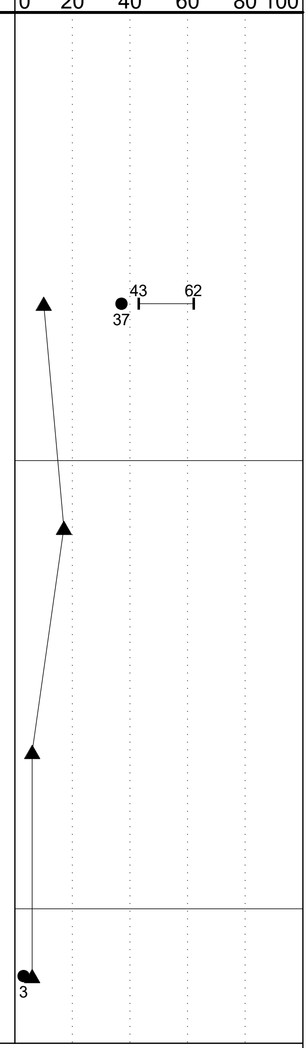
CLIENT City of Manzanita - Dan Weitzel, Public Works Director
PROJECT NAME Classic Street Improvements
PROJECT NUMBER G2406158
PROJECT LOCATION Classic Street - Manzanita, Oregon
DATE STARTED 7/8/24 **GROUND ELEVATION** 75 ft
ELEVATION DATUM From schematic plans provided by client.
WEATHER Sunny, 78F **SURFACE** Asphalt Concrete
LOGGED BY BJG **REVIEWED BY** BMW
DRILLING CONTRACTOR PLI Systems, Inc. **SEEPAGE** ---
EQUIPMENT Mobile B-57 Truck **GROUNDWATER DURING DRILLING** ---
DRILLING METHOD Hollow Stem Auger & DCP **GROUNDWATER AFTER DRILLING** ---

ELEVATION (ft)	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N_{SPT} VALUE)	N_{60} VALUE ETR_{Hammer} = 77.70%	DRY UNIT WT. (pcf)
75		**ASPHALT CONCRETE:** Approximately 2 inches thick.		0					
	GP FILL	**POORLY GRADED GRAVEL FILL:** Brown, dry, angular, up to ¾-inch in diameter, with low plasticity fines.							
	MH	**ELASTIC SILT:** Stiff, brown with multicolored mottling, moist, low to medium plasticity, with trace fine-grained sand.			SPT 1	67	3-4-6 (10)	10	
70				5					
	SP	**POORLY GRADED SAND:** Medium dense, tan with brown mottling, moist, fine- to medium-grained, with no to trace low plasticity fines.			SPT 2	78	4-8-10 (18)	17	
		Very loose below 7½ feet bgs.			SPT 3	67	2-3-3 (6)	6	
65				10					
					SPT 4	67	2-3-3 (6)	6	
60									

▲ SPT N_{60} VALUE ▲ / PL — MC — LL / ☐ FINES CONTENT (%) ☐



- Boring terminated at about 11½ feet bgs.
- No groundwater or caving encountered.
- Boring backfilled with crushed rock and surface patched with cold patch asphalt.

Appendix B: Results of DCP Tests

Classic Street Improvements
Classic Street
Manzanita, Oregon

CGT Project Number G2406158

August 16, 2024

Prepared For:

City of Manzanita
Dan Weitzel, Public Works Director
1090 Oak Street
Manzanita, Oregon 97130

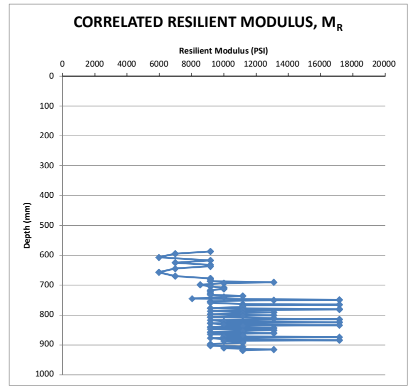
Prepared by
Carlson Geotechnical

Project:	Classic Street Improvements
Project Number:	G2406158
Date:	7/8/2024
Exploration Name:	B-1

Type of Pavement:	AC	C/AC/N (C = Portland Cement Concrete, AC = Asphaltic Concrete, N = None)
Thickness of Pavement:	2	inches
Thickness of Base Rock:	21	inches
Seating Depth:	23	(inches from ground surface to bottom of excavation)
Initial DCP reading:	625	mm

Table 2 - C_f for DCP and FWD to Convert

Layer Type & Location	C_f
Subgrade Below AC & Aggregate Base	0.35
Aggregate Base or Subbase Below AC	0.62
Subgrade Below PCC or CTB	0.25
Aggregate Base or Subbase Below PCC	0.62
None (no pavement)	0.33



Mr (average) within upper 300 mm (12 inches) of subgrade (psi) = 10854

Reading No.	No. of Blows	Depth Reading (mm)	Type of Hammer A=17.6 lb hammer B=10.1 lb hammer (only need to note change in hammer)	Hammer Blow Index	Accumulative Penetration (mm)	Middle of interval (mm)	Middle of interval (inches)	Material Type	Material Type Coefficient C_f	DCP Index mm/blow	CBR (correlation from user manual) %	Subgrade Modulus (Pg. 21 ODOT Pavement Design Guide) psf
1	1	630	A	1	5	587	23.1	Subgrade	0.35	5.00	48	9159
2	1	640		1	15	594	23.4	Subgrade	0.35	10.00	22	6990
3	1	655		1	30	607	23.9	Subgrade	0.35	15.00	14	5967
4	1	660		1	35	617	24.3	Subgrade	0.35	5.00	48	9159
5	1	670		1	45	624	24.6	Subgrade	0.35	10.00	22	6990
6	1	675		1	50	632	24.9	Subgrade	0.35	5.00	48	9159
7	1	680		1	55	637	25.1	Subgrade	0.35	5.00	48	9159
8	1	690		1	65	644	25.4	Subgrade	0.35	10.00	22	6990
9	1	705		1	80	657	25.9	Subgrade	0.35	15.00	14	5967
10	1	715		1	90	669	26.3	Subgrade	0.35	10.00	22	6990
11	1	720		1	95	677	26.6	Subgrade	0.35	5.00	48	9159
12	1	725		1	100	682	26.8	Subgrade	0.35	5.00	48	9159
13	1	730		1	105	687	27.0	Subgrade	0.35	5.00	48	9159
14	1	732		1	107	690	27.2	Subgrade	0.35	2.00	134	13094
15	1	736		1	111	693	27.3	Subgrade	0.35	4.00	62	9992
16	1	742		1	117	698	27.5	Subgrade	0.35	6.00	39	8531
17	1	747		1	122	704	27.7	Subgrade	0.35	5.00	48	9159
18	1	751		1	126	708	27.9	Subgrade	0.35	4.00	62	9992
19	1	755		1	130	712	28.0	Subgrade	0.35	4.00	62	9992
20	1	760		1	135	717	28.2	Subgrade	0.35	5.00	48	9159
21	1	765		1	140	722	28.4	Subgrade	0.35	5.00	48	9159
22	1	770		1	145	727	28.6	Subgrade	0.35	5.00	48	9159
23	1	775		1	150	732	28.8	Subgrade	0.35	5.00	48	9159
24	1	778		1	153	736	29.0	Subgrade	0.35	3.00	85	11179
25	1	782		1	157	739	29.1	Subgrade	0.35	4.00	62	9992
26	1	789		1	164	745	29.3	Subgrade	0.35	7.00	33	8033
27	1	790		1	165	749	29.5	Subgrade	0.35	1.00	292	17158
28	1	792		1	167	750	29.5	Subgrade	0.35	2.00	134	13094
29	1	797		1	172	754	29.7	Subgrade	0.35	5.00	48	9159
30	1	802		1	177	759	29.9	Subgrade	0.35	5.00	48	9159
31	1	805		1	180	763	30.0	Subgrade	0.35	3.00	85	11179
32	1	806		1	181	765	30.1	Subgrade	0.35	1.00	292	17158
33	1	809		1	184	767	30.2	Subgrade	0.35	3.00	85	11179
34	1	812		1	187	770	30.3	Subgrade	0.35	3.00	85	11179
35	1	815		1	190	773	30.4	Subgrade	0.35	3.00	85	11179
36	1	820		1	195	777	30.6	Subgrade	0.35	5.00	48	9159
37	1	821		1	196	780	30.7	Subgrade	0.35	1.00	292	17158
38	1	822		1	197	781	30.7	Subgrade	0.35	1.00	292	17158
39	1	825		1	200	783	30.8	Subgrade	0.35	3.00	85	11179
40	1	830		1	205	787	31.0	Subgrade	0.35	5.00	48	9159
41	1	832		1	207	790	31.1	Subgrade	0.35	2.00	134	13094
42	1	835		1	210	793	31.2	Subgrade	0.35	3.00	85	11179
43	1	840		1	215	797	31.4	Subgrade	0.35	5.00	48	9159
44	1	842		1	217	800	31.5	Subgrade	0.35	2.00	134	13094
45	1	845		1	220	803	31.6	Subgrade	0.35	3.00	85	11179
46	1	850		1	225	807	31.8	Subgrade	0.35	5.00	48	9159
47	1	852		1	227	810	31.9	Subgrade	0.35	2.00	134	13094
48	1	854		1	229	812	32.0	Subgrade	0.35	2.00	134	13094
49	1	855		1	230	814	32.0	Subgrade	0.35	1.00	292	17158
50	1	860		1	235	817	32.2	Subgrade	0.35	5.00	48	9159
51	1	864		1	239	821	32.3	Subgrade	0.35	4.00	62	9992
52	1	865		1	240	824	32.4	Subgrade	0.35	1.00	292	17158
53	1	870		1	245	827	32.5	Subgrade	0.35	5.00	48	9159
54	1	874		1	249	831	32.7	Subgrade	0.35	4.00	62	9992
55	1	875		1	250	834	32.8	Subgrade	0.35	1.00	292	17158
56	1	878		1	253	836	32.9	Subgrade	0.35	3.00	85	11179
57	1	883		1	258	840	33.1	Subgrade	0.35	5.00	48	9159
58	1	885		1	260	843	33.2	Subgrade	0.35	2.00	134	13094
59	1	890		1	265	847	33.3	Subgrade	0.35	5.00	48	9159
60	1	892		1	267	850	33.5	Subgrade	0.35	2.00	134	13094
61	1	895		1	270	853	33.6	Subgrade	0.35	3.00	85	11179
62	1	900		1	275	857	33.7	Subgrade	0.35	5.00	48	9159
63	1	902		1	277	860	33.9	Subgrade	0.35	2.00	134	13094
64	1	907		1	282	864	34.0	Subgrade	0.35	5.00	48	9159
65	1	910		1	285	868	34.2	Subgrade	0.35	3.00	85	11179
66	1	914		1	289	871	34.3	Subgrade	0.35	4.00	62	9992
67	1	915		1	290	874	34.4	Subgrade	0.35	1.00	292	17158
68	1	920		1	295	877	34.5	Subgrade	0.35	5.00	48	9159
69	1	924		1	299	881	34.7	Subgrade	0.35	4.00	62	9992
70	1	925		1	300	884	34.8	Subgrade	0.35	1.00	292	17158
71	1	929		1	304	886	34.9	Subgrade	0.35	4.00	62	9992
72	1	932		1	307	890	35.0	Subgrade	0.35	3.00	85	11179
73	1	935		1	310	893	35.1	Subgrade	0.35	3.00	85	11179
74	1	940		1	315	897	35.3	Subgrade	0.35	5.00	48	9159
75	1	945		1	320	902	35.5	Subgrade	0.35	5.00	48	9159
76	1	948		1	323	906	35.7	Subgrade	0.35	3.00	85	11179
77	1	952		1	327	909	35.8	Subgrade	0.35	4.00	62	9992
78	1	955		1	330	913	35.9	Subgrade	0.35	3.00	85	11179
79	1	957		1	332	915	36.0	Subgrade	0.35	2.00	134	13094
80	1	960		1	335	918	36.1	Subgrade	0.35	3.00	85	11179
81	1	962		1	337	920	36.2	Subgrade	0.35	2.00	134	13094
82	1	968		1	343	924	36.4	Subgrade	0.35	6.00	39	8531
83	1	972		1	347	929	36.6	Subgrade	0.35	4.00	62	9992
84	1	975		1	350	933	36.7	Subgrade	0.35	3.00	85	11179
85	1	979		1	354	936	36.9	Subgrade	0.35	4.00	62	9992
86	1	984		1	359	941	37.0	Subgrade	0.35	5.00	48	9159
87	1	988		1	363	945	37.2	Subgrade	0.35	4.00	62	9992
88	1	992		1	367	949	37.4	Subgrade	0.35	4.00	62	9992
89	1	995		1	370	953	37.5	Subgrade	0.35	3.00	85	11179
90	1	999		1	374	956	37.6	Subgrade	0.35	4.00	62	9992
91	1	1008		1	383	963	37.9	Subgrade	0.35	9.00	25	7283
92	1	1015		1	390	971	38.2	Subgrade	0.35	7.00	33	8033
93	1	1020		1	395	977	38.5	Subgrade	0.35	5.00	48	9159
94	1	1025		1	400	982	38.6	Subgrade	0.35	5.00	48	9159
95	1	1029		1	404	986	38.8	Subgrade	0.35	4.00	62	9992
96	1	1031		1	406	989	38.9	Subgrade	0.35	2.00	134	13094
97	1	1036		1	411	993	39.1	Subgrade	0.35	5.00	48	9159
98	1	1040		1	415	997	39.3	Subgrade	0.35	4.00	62	9992
99	1	1045		1	420	1002	39.4	Subgrade	0.35	5.00	48	9159
100	1	1050		1	425	1007	39.6	Subgrade	0.35	5.00	48	9159
101	1	1058		1	433	1013	39.9	Subgrade	0.35	8.00	28	7625
102	1	1061		1	436	1019	40.1	Subgrade	0.35	3.00	85	11179
103	1	1065		1	440	1022	40.2	Subgrade	0.35	4.00	62	9992
104	1	1069		1	444	1026	40.4	Subgrade	0.35	4.00	62	9992
105	1	1074		1	449	1031	40.6	Subgrade	0.35	5.00	48	9159
106	1	1078		1	453	1035	40.8	Subgrade	0.35	4.00	62	9992
107	1	1082		1	457	1039	40.9	Subgrade	0.35	4.00	62	9992
108	1	1090		1	465	1045	41.1	Subgrade	0.35	8.00	28	7625
109	1	1098		1	473	1053	41.5	Subgrade	0.35	8.00	28	7625
110	1	1105		1	480	1061	41.8	Subgrade	0.35	7.00	33	8033
111	1	1115		1	490	1069	42.1	Subgrade	0.35	10.00	22	6990
112	1	1120		1	495	1077	42.4	Subgrade	0.35	5.00	48	9159
113	1	1126		1	501	1082	42.6	Subgrade	0.35	6.00	39	8531
114	1	1132		1	507	1088	42.8	Subgrade	0.35	6.00	39	8531
115	1	1140		1	515	1095	43.1	Subgrade	0.35	8.00	28	7625

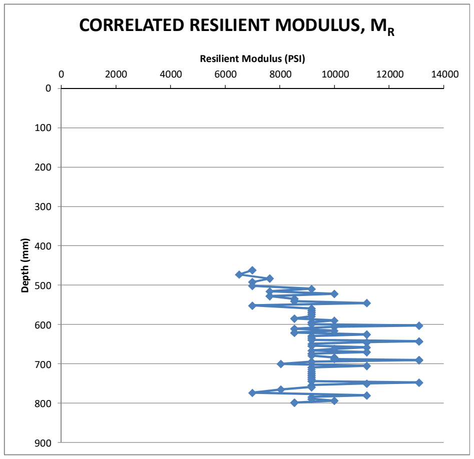
Project:	Classic Street Improvements
Project Number:	G2406158
Date:	7/8/2024
Exploration Name:	B-2

Type of Pavement:	AC	C/AC/N (C = Portland Cement Concrete, AC = Asphaltic Concrete, N = None)
Thickness of Pavement:	2	inches
Thickness of Base Rock:	16	inches
Seating Depth:	18	(inches from ground surface to bottom of excavation)
Initial DCP reading:	795	mm

Table 2 - C_f for DCP and FWD to

Layer Type & Location	C_f
Subgrade Below AC & Aggregate Base	0.35
Aggregate Base or Subbase Below AC	0.62
Subgrade Below PCC or CTB	0.25
Aggregate Base or Subbase Below PCC	0.62
None (no pavement)	0.33

Reading No.	No. of Blows	Depth Reading (mm)	Type of Hammer A=17.6 lb hammer B=10.1 lb hammer (only need to note change in hammer)	Hammer Blow Index	Accumulative Penetration (mm)	Middle of interval (mm)	Middle of interval (inches)	Material Type	Material Type Coefficient C_f	DCP Index mm/blow	CBR (correlation from user manual) %	Subgrade Modulus (Pg. 21 ODOT Pavement Design Guide) psf
1	1	805	A	1	10	462	18.2	Subgrade	0.35	10.00	22	6990
2	1	817		1	22	473	18.6	Subgrade	0.35	12.00	18	6510
3	1	825		1	30	483	19.0	Subgrade	0.35	8.00	28	7625
4	1	835		1	40	492	19.4	Subgrade	0.35	10.00	22	6990
5	1	845		1	50	502	19.8	Subgrade	0.35	10.00	22	6990
6	1	850		1	55	510	20.1	Subgrade	0.35	5.00	48	9159
7	1	858		1	63	516	20.3	Subgrade	0.35	8.00	28	7625
8	1	862		1	67	522	20.6	Subgrade	0.35	4.00	62	9992
9	1	870		1	75	528	20.8	Subgrade	0.35	8.00	28	7625
10	1	876		1	81	535	21.1	Subgrade	0.35	6.00	39	8531
11	1	882		1	87	541	21.3	Subgrade	0.35	6.00	39	8531
12	1	885		1	90	546	21.5	Subgrade	0.35	3.00	85	11179
13	1	895		1	100	552	21.7	Subgrade	0.35	10.00	22	6990
14	1	900		1	105	560	22.0	Subgrade	0.35	5.00	48	9159
15	1	905		1	110	565	22.2	Subgrade	0.35	5.00	48	9159
16	1	910		1	115	570	22.4	Subgrade	0.35	5.00	48	9159
17	1	915		1	120	575	22.6	Subgrade	0.35	5.00	48	9159
18	1	920		1	125	580	22.8	Subgrade	0.35	5.00	48	9159
19	1	926		1	131	585	23.0	Subgrade	0.35	6.00	39	8531
20	1	930		1	135	590	23.2	Subgrade	0.35	4.00	62	9992
21	1	935		1	140	595	23.4	Subgrade	0.35	5.00	48	9159
22	1	940		1	145	600	23.6	Subgrade	0.35	5.00	48	9159
23	1	942		1	147	603	23.7	Subgrade	0.35	2.00	134	13094
24	1	946		1	151	606	23.9	Subgrade	0.35	4.00	62	9992
25	1	952		1	157	611	24.1	Subgrade	0.35	6.00	39	8531
26	1	956		1	161	616	24.3	Subgrade	0.35	4.00	62	9992
27	1	962		1	167	621	24.5	Subgrade	0.35	6.00	39	8531
28	1	965		1	170	626	24.6	Subgrade	0.35	3.00	85	11179
29	1	970		1	175	630	24.8	Subgrade	0.35	5.00	48	9159
30	1	975		1	180	635	25.0	Subgrade	0.35	5.00	48	9159
31	1	980		1	185	640	25.2	Subgrade	0.35	5.00	48	9159
32	1	982		1	187	643	25.3	Subgrade	0.35	2.00	134	13094
33	1	985		1	190	646	25.4	Subgrade	0.35	3.00	85	11179
34	1	990		1	195	650	25.6	Subgrade	0.35	5.00	48	9159
35	1	995		1	200	655	25.8	Subgrade	0.35	5.00	48	9159
36	1	998		1	203	659	25.9	Subgrade	0.35	3.00	85	11179
37	1	1002		1	207	662	26.1	Subgrade	0.35	4.00	62	9992
38	1	1007		1	212	667	26.2	Subgrade	0.35	5.00	48	9159
39	1	1010		1	215	671	26.4	Subgrade	0.35	3.00	85	11179
40	1	1015		1	220	675	26.6	Subgrade	0.35	5.00	48	9159
41	1	1020		1	225	680	26.8	Subgrade	0.35	5.00	48	9159
42	1	1024		1	229	684	26.9	Subgrade	0.35	4.00	62	9992
43	1	1028		1	233	688	27.1	Subgrade	0.35	4.00	62	9992
44	1	1030		1	235	691	27.2	Subgrade	0.35	2.00	134	13094
45	1	1035		1	240	695	27.4	Subgrade	0.35	5.00	48	9159
46	1	1042		1	247	701	27.6	Subgrade	0.35	7.00	33	8033
47	1	1045		1	250	706	27.8	Subgrade	0.35	3.00	85	11179
48	1	1050		1	255	710	27.9	Subgrade	0.35	5.00	48	9159
49	1	1055		1	260	715	28.1	Subgrade	0.35	5.00	48	9159
50	1	1060		1	265	720	28.3	Subgrade	0.35	5.00	48	9159
51	1	1065		1	270	725	28.5	Subgrade	0.35	5.00	48	9159
52	1	1070		1	275	730	28.7	Subgrade	0.35	5.00	48	9159
53	1	1075		1	280	735	28.9	Subgrade	0.35	5.00	48	9159
54	1	1080		1	285	740	29.1	Subgrade	0.35	5.00	48	9159
55	1	1085		1	290	745	29.3	Subgrade	0.35	5.00	48	9159
56	1	1087		1	292	748	29.5	Subgrade	0.35	2.00	134	13094
57	1	1090		1	295	751	29.6	Subgrade	0.35	3.00	85	11179
58	1	1095		1	300	755	29.7	Subgrade	0.35	5.00	48	9159
59	1	1100		1	305	760	29.9	Subgrade	0.35	5.00	48	9159
60	1	1107		1	312	766	30.1	Subgrade	0.35	7.00	33	8033
61	1	1117		1	322	774	30.5	Subgrade	0.35	10.00	22	6990
62	1	1120		1	325	781	30.7	Subgrade	0.35	3.00	85	11179
63	1	1125		1	330	785	30.9	Subgrade	0.35	5.00	48	9159
64	1	1130		1	335	790	31.1	Subgrade	0.35	5.00	48	9159
65	1	1134		1	339	794	31.3	Subgrade	0.35	4.00	62	9992
66	1	1140		1	345	799	31.5	Subgrade	0.35	6.00	39	8531
67												



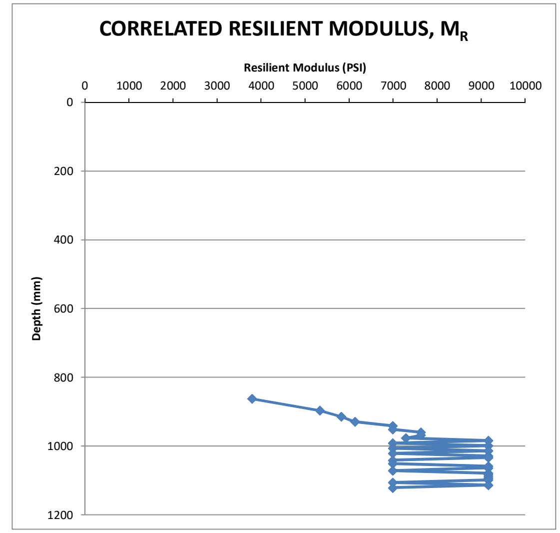
Mr (average) within upper 300 mm (12 inches) of subgrade (psi) = 9399

Project:	Classic Street Improvements
Project Number:	G2406158
Date:	7/8/2024
Exploration Name:	B-3

Type of Pavement:	AC	C/AC/N (C = Portland Cement Concrete, AC = Asphaltic Concrete, N = None)
Thickness of Pavement:	2	inches
Thickness of Base Rock:	24	inches
Seating Depth:	33	(inches from ground surface to bottom of excavation)
Initial DCP reading:	852	mm

Table 2 - C_f for DCP and FWD to

Layer Type & Location	C_f
Subgrade Below AC & Aggregate Base	0.35
Aggregate Base or Subbase Below AC	0.62
Subgrade Below PCC or CTB	0.25
Aggregate Base or Subbase Below PCC	0.62
None (no pavement)	0.33



Mr (average) within upper 300 mm (12 inches) of subgrade (psi) = 7753

Reading No.	No. of Blows	Depth Reading (mm)	Type of Hammer A=17.6 lb hammer B=10.1 lb hammer (only need to note change in hammer)	Hammer Blow Index	Accumulative Penetration (mm)	Middle of interval (mm)	Middle of interval (inches)	Material Type	Material Type Coefficient C_f	DCP Index mm/blow	CBR (correlation from user manual) %	Subgrade Modulus (Pg. 21 ODOT Pavement Design Guide) psf
1	1	900	A	1	48	862	33.9	Subgrade	0.35	48.00	4	3791
2	1	920		1	68	896	35.3	Subgrade	0.35	20.00	10	5334
3	1	936		1	84	914	36.0	Subgrade	0.35	16.00	13	5819
4	1	950		1	98	929	36.6	Subgrade	0.35	14.00	15	6130
5	1	960		1	108	941	37.1	Subgrade	0.35	10.00	22	6990
6	1	970		1	118	951	37.4	Subgrade	0.35	10.00	22	6990
7	1	978		1	126	960	37.8	Subgrade	0.35	8.00	28	7625
8	1	986		1	134	968	38.1	Subgrade	0.35	8.00	28	7625
9	1	995		1	143	977	38.5	Subgrade	0.35	9.00	25	7283
10	1	1000		1	148	984	38.7	Subgrade	0.35	5.00	48	9159
11	1	1010		1	158	991	39.0	Subgrade	0.35	10.00	22	6990
12	1	1015		1	163	999	39.3	Subgrade	0.35	5.00	48	9159
13	1	1025		1	173	1006	39.6	Subgrade	0.35	10.00	22	6990
14	1	1030		1	178	1014	39.9	Subgrade	0.35	5.00	48	9159
15	1	1040		1	188	1021	40.2	Subgrade	0.35	10.00	22	6990
16	1	1045		1	193	1029	40.5	Subgrade	0.35	5.00	48	9159
17	1	1050		1	198	1034	40.7	Subgrade	0.35	5.00	48	9159
18	1	1060		1	208	1041	41.0	Subgrade	0.35	10.00	22	6990
19	1	1070		1	218	1051	41.4	Subgrade	0.35	10.00	22	6990
20	1	1075		1	223	1059	41.7	Subgrade	0.35	5.00	48	9159
21	1	1080		1	228	1064	41.9	Subgrade	0.35	5.00	48	9159
22	1	1090		1	238	1071	42.2	Subgrade	0.35	10.00	22	6990
23	1	1095		1	243	1079	42.5	Subgrade	0.35	5.00	48	9159
24	1	1100		1	248	1084	42.7	Subgrade	0.35	5.00	48	9159
25	1	1105		1	253	1089	42.9	Subgrade	0.35	5.00	48	9159
26	1	1110		1	258	1094	43.1	Subgrade	0.35	5.00	48	9159
27	1	1115		1	263	1099	43.3	Subgrade	0.35	5.00	48	9159
28	1	1125		1	273	1106	43.6	Subgrade	0.35	10.00	22	6990
29	1	1130		1	278	1114	43.8	Subgrade	0.35	5.00	48	9159
30	1	1140		1	288	1121	44.1	Subgrade	0.35	10.00	22	6990
31												

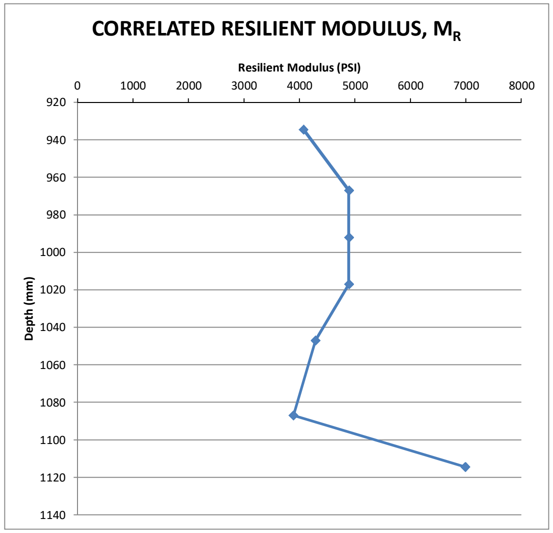
Project:	Classic Street Improvements
Project Number:	G2406158
Date:	7/8/2024
Exploration Name:	B-4

Type of Pavement:	AC	**C/AC/N (C = Portland Cement Concrete, AC = Asphaltic Concrete, N = None)**
Thickness of Pavement:	2	**inches**
Thickness of Base Rock:	34	**inches**
Seating Depth:	36	**(inches from ground surface to bottom of excavation)**
Initial DCP reading:	935	**mm**

Table 2 - C_f for DCP and FWD to

Layer Type & Location	C_f
Subgrade Below AC & Aggregate Base	0.35
Aggregate Base or Subbase Below AC	0.62
Subgrade Below PCC or CTB	0.25
Aggregate Base or Subbase Below PCC	0.62
None (no pavement)	0.33

Reading No.	No. of Blows	Depth Reading (mm)	Type of Hammer A=17.6 lb hammer B=10.1 lb hammer (only need to note change in hammer)	Hammer Blow Index	Accumulative Penetration (mm)	Middle of interval (mm)	Middle of interval (inches)	Material Type	Material Type Coefficient C_f	DCP Index mm/blow	CBR (correlation from user manual) %	Subgrade Modulus (Pg. 21 ODOT Pavement Design Guide) psf
1	1	975	A	1	40	934	36.8	Subgrade	0.35	40.00	5	4071
2	1	1000		1	65	967	38.1	Subgrade	0.35	25.00	8	4890
3	1	1025		1	90	992	39.1	Subgrade	0.35	25.00	8	4890
4	1	1050		1	115	1017	40.0	Subgrade	0.35	25.00	8	4890
5	1	1085		1	150	1047	41.2	Subgrade	0.35	35.00	5	4288
6	1	1130		1	195	1087	42.8	Subgrade	0.35	45.00	4	3888
7	1	1140		1	205	1114	43.9	Subgrade	0.35	10.00	22	6990
8												
9												
10												
11												
12												
13												
14												
15												
16												



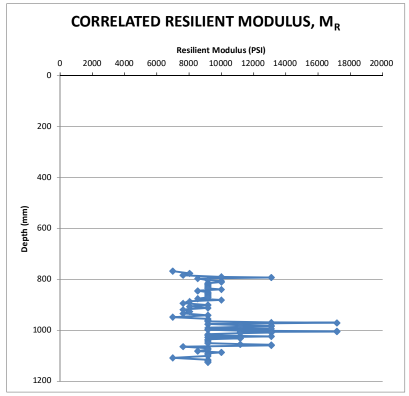
Mr (average) within upper 300 mm (12 inches) of subgrade (psi) = 4844

Project:	Classic Street Improvements
Project Number:	G2406158
Date:	7/8/2024
Exploration Name:	B-5

Type of Pavement:	AC	C/AC/N (C = Portland Cement Concrete, AC = Asphaltic Concrete, N = None)
Thickness of Pavement:	2	inches
Thickness of Base Rock:	28	inches
Seating Depth:	30	(inches from ground surface to bottom of excavation)
Initial DCP reading:	775	mm

Table 2 - C_f for DCP and FWD to Convert

Layer Type & Location	C_f
Subgrade Below AC & Aggregate Base	0.35
Aggregate Base or Subbase Below AC	0.62
Subgrade Below PCC or CTB	0.25
Aggregate Base or Subbase Below PCC	0.62
None (no pavement)	0.33



Mr (average) within upper 300 mm (12 inches) of subgrade (psi) = 10103

Reading No.	No. of Blows	Depth Reading (mm)	Type of Hammer A=17.6 lb hammer B=10.1 lb hammer (only need to note change in hammer)	Hammer Blow Index	Accumulative Penetration (mm)	Middle of interval (mm)	Middle of interval (inches)	Material Type	Material Type Coefficient C_f	DCP Index mm/blow	CBR (correlation from user manual) %	Subgrade Modulus (Pg. 21 ODOT Pavement Design Guide) psf
1	1	785	A	1	10	767	30.2	Subgrade	0.35	10.00	22	6990
2	1	792		1	17	776	30.5	Subgrade	0.35	7.00	33	8033
3	1	800		1	25	783	30.8	Subgrade	0.35	8.00	28	7625
4	1	804		1	29	789	31.1	Subgrade	0.35	4.00	62	9992
5	1	806		1	31	792	31.2	Subgrade	0.35	2.00	134	13094
6	1	812		1	37	796	31.3	Subgrade	0.35	6.00	39	8531
7	1	817		1	42	802	31.6	Subgrade	0.35	5.00	48	9159
8	1	821		1	46	806	31.7	Subgrade	0.35	4.00	62	9992
9	1	825		1	50	810	31.9	Subgrade	0.35	4.00	62	9992
10	1	830		1	55	815	32.1	Subgrade	0.35	5.00	48	9159
11	1	835		1	60	820	32.3	Subgrade	0.35	5.00	48	9159
12	1	840		1	65	825	32.5	Subgrade	0.35	5.00	48	9159
13	1	845		1	70	830	32.7	Subgrade	0.35	5.00	48	9159
14	1	850		1	75	835	32.9	Subgrade	0.35	5.00	48	9159
15	1	854		1	79	839	33.0	Subgrade	0.35	4.00	62	9992
16	1	860		1	85	844	33.2	Subgrade	0.35	6.00	39	8531
17	1	865		1	90	850	33.4	Subgrade	0.35	5.00	48	9159
18	1	870		1	95	855	33.6	Subgrade	0.35	5.00	48	9159
19	1	875		1	100	860	33.8	Subgrade	0.35	5.00	48	9159
20	1	880		1	105	865	34.0	Subgrade	0.35	5.00	48	9159
21	1	885		1	110	870	34.2	Subgrade	0.35	5.00	48	9159
22	1	891		1	116	875	34.4	Subgrade	0.35	6.00	39	8531
23	1	895		1	120	880	34.6	Subgrade	0.35	4.00	62	9992
24	1	902		1	127	886	34.9	Subgrade	0.35	7.00	33	8033
25	1	910		1	135	893	35.2	Subgrade	0.35	8.00	28	7625
26	1	915		1	140	900	35.4	Subgrade	0.35	5.00	48	9159
27	1	922		1	147	906	35.6	Subgrade	0.35	7.00	33	8033
28	1	927		1	152	912	35.9	Subgrade	0.35	5.00	48	9159
29	1	935		1	160	918	36.1	Subgrade	0.35	8.00	28	7625
30	1	942		1	167	926	36.4	Subgrade	0.35	7.00	33	8033
31	1	950		1	175	933	36.7	Subgrade	0.35	8.00	28	7625
32	1	955		1	180	940	37.0	Subgrade	0.35	5.00	48	9159
33	1	965		1	190	947	37.3	Subgrade	0.35	10.00	22	6990
34	1	970		1	195	955	37.6	Subgrade	0.35	5.00	48	9159
35	1	975		1	200	960	37.8	Subgrade	0.35	5.00	48	9159
36	1	980		1	205	965	38.0	Subgrade	0.35	5.00	48	9159
37	1	982		1	207	968	38.1	Subgrade	0.35	2.00	134	13094
38	1	983		1	208	970	38.2	Subgrade	0.35	1.00	292	17158
39	1	985		1	210	971	38.2	Subgrade	0.35	2.00	134	13094
40	1	990		1	215	975	38.4	Subgrade	0.35	5.00	48	9159
41	1	993		1	218	979	38.5	Subgrade	0.35	3.00	85	11179
42	1	995		1	220	981	38.6	Subgrade	0.35	2.00	134	13094
43	1	997		1	222	983	38.7	Subgrade	0.35	2.00	134	13094
44	1	1000		1	225	986	38.8	Subgrade	0.35	3.00	85	11179
45	1	1005		1	230	990	39.0	Subgrade	0.35	5.00	48	9159
46	1	1007		1	232	993	39.1	Subgrade	0.35	2.00	134	13094
47	1	1012		1	237	997	39.2	Subgrade	0.35	5.00	48	9159
48	1	1015		1	240	1001	39.4	Subgrade	0.35	3.00	85	11179
49	1	1016		1	241	1003	39.5	Subgrade	0.35	1.00	292	17158
50	1	1017		1	242	1004	39.5	Subgrade	0.35	1.00	292	17158
51	1	1020		1	245	1006	39.6	Subgrade	0.35	3.00	85	11179
52	1	1022		1	247	1008	39.7	Subgrade	0.35	2.00	134	13094
53	1	1025		1	250	1011	39.8	Subgrade	0.35	3.00	85	11179
54	1	1030		1	255	1015	39.9	Subgrade	0.35	5.00	48	9159
55	1	1035		1	260	1020	40.1	Subgrade	0.35	5.00	48	9159
56	1	1037		1	262	1023	40.3	Subgrade	0.35	2.00	134	13094
57	1	1042		1	267	1027	40.4	Subgrade	0.35	5.00	48	9159
58	1	1045		1	270	1031	40.6	Subgrade	0.35	3.00	85	11179
59	1	1050		1	275	1035	40.7	Subgrade	0.35	5.00	48	9159
60	1	1055		1	280	1040	40.9	Subgrade	0.35	5.00	48	9159
61	1	1060		1	285	1045	41.1	Subgrade	0.35	5.00	48	9159
62	1	1065		1	290	1050	41.3	Subgrade	0.35	5.00	48	9159
63	1	1068		1	293	1054	41.5	Subgrade	0.35	3.00	85	11179
64	1	1070		1	295	1056	41.6	Subgrade	0.35	2.00	134	13094
65	1	1072		1	297	1058	41.7	Subgrade	0.35	2.00	134	13094
66	1	1080		1	305	1063	41.9	Subgrade	0.35	8.00	28	7625
67	1	1085		1	310	1070	42.1	Subgrade	0.35	5.00	48	9159
68	1	1090		1	315	1075	42.3	Subgrade	0.35	5.00	48	9159
69	1	1096		1	321	1080	42.5	Subgrade	0.35	6.00	39	8531
70	1	1100		1	325	1085	42.7	Subgrade	0.35	4.00	62	9992
71	1	1105		1	330	1090	42.9	Subgrade	0.35	5.00	48	9159
72	1	1110		1	335	1095	43.1	Subgrade	0.35	5.00	48	9159
73	1	1115		1	340	1100	43.3	Subgrade	0.35	5.00	48	9159
74	1	1125		1	350	1107	43.6	Subgrade	0.35	10.00	22	6990
75	1	1130		1	355	1115	43.9	Subgrade	0.35	5.00	48	9159
76	1	1135		1	360	1120	44.1	Subgrade	0.35	5.00	48	9159
77	1	1140		1	365	1125	44.3	Subgrade	0.35	5.00	48	9159
78												

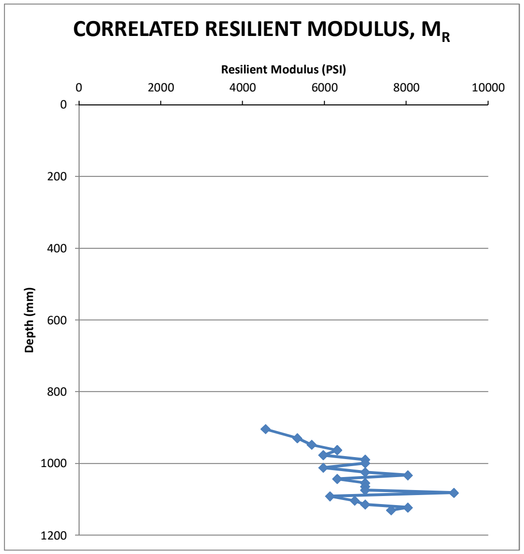
Project:	Classic Street Improvements
Project Number:	G2406158
Date:	7/8/2024
Exploration Name:	B-6

Type of Pavement:	AC	C/AC/N (C = Portland Cement Concrete, AC = Asphaltic Concrete, N = None)
Thickness of Pavement:	3	inches
Thickness of Base Rock:	32	inches
Seating Depth:	35	(inches from ground surface to bottom of excavation)
Initial DCP reading:	895	mm

Table 2 - C_f for DCP and FWD to Convert

Layer Type & Location	C_f
Subgrade Below AC & Aggregate Base	0.35
Aggregate Base or Subbase Below AC	0.62
Subgrade Below PCC or CTB	0.25
Aggregate Base or Subbase Below PCC	0.62
None (no pavement)	0.33

Reading No.	No. of Blows	Depth Reading (mm)	Type of Hammer A=17.6 lb hammer B=10.1 lb hammer (only need to note change in hammer)	Hammer Blow Index	Accumulative Penetration (mm)	Middle of interval (mm)	Middle of interval (inches)	Material Type	Material Type Coefficient C_f	DCP Index mm/blow	CBR (correlation from user manual) %	Subgrade Modulus (Pg. 21 ODOT Pavement Design Guide) psf
1	1	925	A	1	30	904	35.6	Subgrade	0.35	30.00	6	4554
2	1	945		1	50	929	36.6	Subgrade	0.35	20.00	10	5334
3	1	962		1	67	948	37.3	Subgrade	0.35	17.00	12	5683
4	1	975		1	80	963	37.9	Subgrade	0.35	13.00	17	6310
5	1	990		1	95	977	38.4	Subgrade	0.35	15.00	14	5967
6	1	1000		1	105	989	38.9	Subgrade	0.35	10.00	22	6990
7	1	1010		1	115	999	39.3	Subgrade	0.35	10.00	22	6990
8	1	1025		1	130	1012	39.8	Subgrade	0.35	15.00	14	5967
9	1	1035		1	140	1024	40.3	Subgrade	0.35	10.00	22	6990
10	1	1042		1	147	1033	40.6	Subgrade	0.35	7.00	33	8033
11	1	1055		1	160	1043	41.0	Subgrade	0.35	13.00	17	6310
12	1	1065		1	170	1054	41.5	Subgrade	0.35	10.00	22	6990
13	1	1075		1	180	1064	41.9	Subgrade	0.35	10.00	22	6990
14	1	1085		1	190	1074	42.3	Subgrade	0.35	10.00	22	6990
15	1	1090		1	195	1082	42.6	Subgrade	0.35	5.00	48	9159
16	1	1104		1	209	1091	43.0	Subgrade	0.35	14.00	15	6130
17	1	1115		1	220	1104	43.4	Subgrade	0.35	11.00	20	6735
18	1	1125		1	230	1114	43.9	Subgrade	0.35	10.00	22	6990
19	1	1132		1	237	1123	44.2	Subgrade	0.35	7.00	33	8033
20	1	1140		1	245	1130	44.5	Subgrade	0.35	8.00	28	7625
21												



Mr (average) within upper 300 mm (12 inches) of subgrade (psi) = 6595

Appendix C: Pavement Structural Capacity Evaluation

Classic Street Improvements
Classic Street
Manzanita, Oregon

CGT Project Number G2406158

August 16, 2024

Prepared For:

City of Manzanita
Dan Weitzel, Public Works Director
1090 Oak Street
Manzanita, Oregon 97130

Prepared by
Carlson Geotechnical

C.1.0 BACKGROUND

In order to evaluate the existing pavement within the subject portion[1] of Classic Street[2], and determine if structural enhancements were required to help maintain a minimum level of serviceability[3] for a design period of 20 years[4], a structural capacity evaluation was performed. We performed the structural capacity evaluation based on visual survey and materials investigation/testing in general accordance with Sections 5.3 and 5.4 of the AASHTO Guide for Design of Pavement Structures, 1993 (AASHTO). The following sections summarize the results of the visual condition survey, the results of our structural capacity analyses, and conclusions for the pavement structure.

C.2.0 PAVEMENT MATERIALS INVESTIGATION

As indicated in the geotechnical report, CGT advanced six drilled borings (B-1 through B-6) and six dynamic cone penetrometer (DCP) tests along the subject road segment. The results of our completed field investigation are briefly summarized in the following table.

Table C1 Pavement Material Thicknesses at Core Locations

Exploration	Location	Pavement Material Thickness (inches)		Correlated Subgrade Resilient Modulus (psi)[1]
		Asphalt Concrete	Aggregate Base	
B-1	See Figure 2	2	21	10854
B-2	See Figure 2	2	16	9399
B-3	See Figure 2	2	24	7753
B-4	See Figure 2	2	34	4844
B-5	See Figure 2	2	28	10103
B-6	See Figure 2	3	35	6595

[1]Average value within upper 1-foot of subgrade based on DCP testing in August 2024.

C.3.0 VISUAL CONDITION SURVEY

C.3.1 Overview

CGT engineering staff observed surface conditions of the asphalt concrete (AC) pavement within Classic Street on June 25, 2024. The purpose of the visit was to identify the type, frequency, severity, and location of any observed surface distress in the existing pavement in accordance with AASHTO procedures and the 2022 Oregon Department of Transportation Distress Survey Manual (ODOT DSM).

The following table presents a checklist of typical surface distress in flexible (asphalt) pavement. This table also includes our observations of the presence (or lack thereof) of the surface distress within the road.

1 This evaluation covers Classic Street, between Dorcas Lane and Necarney City Road.

2 Classic Street is a Minor Collector per input form the City of Manzanita.

3 Terminal serviceability assigned as 2.5 in accordance with the 2019 Oregon Department of Transportation (ODOT) pavement design manual.

4 Assumed design period for the structural capacity analysis. *If an alternative design period is warranted, please contact us.*

Table C2 Pavement Distress Type & Those Observed at Site

Distress Type	Typical Cause(s)	Observed at Site?
Rutting in the wheel paths	Ruts typically develop from consolidation or lateral movement under traffic.	None of significance observed
Fatigue (alligator) cracking	Typically caused by excessive deflection of the surface over unstable subgrade or lower courses of pavement. The unstable support usually is the result of saturated granular base or subgrade.	Yes, see discussion below
Longitudinal/transverse cracking	Typically due to poorly constructed paving joints, shrinkage of asphalt layer, daily temperature cycling, etc.	Yes, see discussion below
Patching	Typically used where the original pavement surface is removed and replaced, or additional material is applied to the pavement surface after original construction.	Yes, see discussion below
Disintegration (potholes)	Typically caused by weakness in the pavement resulting from insufficient asphalt, failure of base, and/or poor drainage.	None observed
Disintegration (raveling)	Typically caused by lack of compaction and/or improper mix proportions.	None observed
Localized Subsidence	Typically caused by poor quality subgrade materials susceptible to consolidation	Yes, see discussion below
Edge cracking	Typically due to lack of lateral (shoulder) support. Another cause of edge cracking can be settlement or yielding of subgrade or granular base.	Yes, see discussion below
Edge joint (seam) “cracking”	Typically due to poor drainage due to a shoulder being higher than the main pavement.	None observed
Corrugations (washboarding)	This form of distress typically occurs in asphalt layers that lack stability due to less than favorable mix proportions.	None observed
Upheaval	Typically caused by expansive soils and/or tree roots.	None observed

C.3.2 Fatigue Cracking

We observed fatigue (alligator) cracking within a few localized areas within the subject street. The cracks were generally ¼- to ½-inch in width and exhibited low spalling. The degree of fatigue cracking was characterized as “low to moderate” in accordance with guidelines presented in the ODOT DSM. Examples of fatigue cracking are shown on Photographs 5, 6, 11, 12, 14, and 25 on the attached Figure C2.

C.3.3 Longitudinal & Transverse Cracking

We observed longitudinal and transverse cracking within the subject street. The longitudinal cracks were generally ¼ to ½ inch in width and observed mostly along the pavement centerline (and interpreted to be attributed to asphalt shrinkage along a paving joint). The degree of longitudinal cracking was characterized as “low to moderate” in accordance with guidelines presented in the ODOT DSM. Examples of longitudinal and transverse cracks are shown on Photographs 2, 11, 18 through 23, and 29 on the attached Figure C2.

C.3.4 Patching

We observed a total of four patches within the subject street. The patches were relatively small in terms of footprint and along the edges of the street. The degree of patching was characterized as “low severity” in accordance with guidelines presented in the ODOT DSM. The patches are shown on Photographs 13 and 28 on the attached Figure C2.

C.3.5 Localized Subsidence

We observed localized subsidence (localized slumps) within three areas along the west margin of the subject street. These areas are approximated on the Site Plan (Figure 2) attached to the main body of the geotechnical report. The areas exhibiting subsidence are shown on Photographs 2, 3, 4, 6, 7, 9, 10, and 25 on the attached Figure C2. As shown therein, the east margin of each area exhibited distress (in the form of fatigue/linear cracking). Each area was relatively close to a descending slope. Additional discussion of these areas and recommendations for repairs are presented in the main body of this report.

C.3.6 Edge Cracking

We observed edge cracking at one location within the west side of the subject street (just north of one of the areas exhibiting subsidence described in the preceding section). The edge cracking is shown on Photograph 8 on the attached Figure C2.

C.4.0 STRUCTURAL CAPACITY ANALYSES

C.4.1 Methodology

We evaluated the structural capacity of the existing pavement structure using the results of the pavement materials investigation and visual condition survey in general accordance with Section 5.4.5 of AASHTO. The purpose of this evaluation was to determine whether structural enhancement (such as an overlay) was required to help manage anticipated design vehicular traffic. The methodology presented by AASHTO incorporates the use of structural numbers (SN) as follows:

- SN_{eff} = Effective structural number of the existing pavement structure, determined from the visual condition survey and investigation of the existing pavement.
- SN_f = Required structural number for future traffic.
- SN_{ol} = Required additional AC pavement thickness structural number. This value is equal to SN_f - SN_{eff}. The methodology indicates that, in the event that SN_{eff} is greater than S_f, and no functional deficiencies are observed in the existing pavement, an overlay is not required. Similarly, in the event that SN_{eff} is less than SN_f, additional AC pavement thickness is required to maintain the desired level of serviceability over the indicated design period.

C.4.2 Design Input Parameters

For the purposes of calculating the structural numbers, a number of parameters were estimated based on the results of the visual survey and pavement investigation. In addition, input parameters related to future traffic and level of serviceability were estimated based on guidelines presented by AASHTO and within the ODOT Pavement Design Guide (ODOT PDG)[5] and the Asphalt Pavement Association of Oregon (APAO)[6] manual. The parameters used in the evaluation are shown in the following table and are discussed in narrative thereafter.

5 Oregon Department of Transportation (ODOT) Pavement Design Guide, January 2019.
6 Asphalt Pavement Association of Oregon (APAO) Asphalt Pavement Design Guide, Revised October 2003.

Table C3 Design Input Parameters

Structural Number	Required Input Parameter	Value Used in Evaluation
SN_{eff}	a_1 = Structural layer coefficient, AC layer	0.35
	a_2 = Structural layer coefficient, base layer	0.10
	a_3 = Structural layer coefficient, subbase layer	---
	D_1 = Thickness of existing pavement, surface layer[1]	2 inches
	D_2 = Thickness of existing pavement, base layer[1]	16 inches
	D_3 = Thickness of existing pavement, subbase layer	---
	M_2 = Drainage coefficient for granular base	1.0
	M_3 = Drainage coefficient for granular subbase	---
SN_f	N_f = Design period	20 years
	$ESAL_f$ = Design 18-kip ESAL over design period[2]	100,000
	M_R = Design resilient modulus[3]	8,200 psi
	Design Serviceability (PSI) Loss (Initial = 4.2, Terminal = 2.5)	1.7
	R = Design Reliability	85 percent
	S_o = Design Standard Deviation	0.49

1 Value based on typical AC thickness observed in boring B-2 (representing the thinnest pavement section identified during drilling).
2 Value selected based on street classification (Minor Collector) per APAO manual. Additional discussion presented below.
3 Value selected based on results of DCP testing (average value used for design purposes).

The following summarizes additional comments on the values presented in Table C3:

- Layer coefficients (a_1, a_2, and a_3) were determined based on results of visual condition survey discussed in Section C.3 above and Table 5.2 of AASHTO.
- Layer thicknesses (D_1, D_2, and D_3) were based on results of our pavement materials investigation.
- A design period of 20 years was assigned for the subject street in accordance with current standard of practice for new construction.
- The design 18-kip equivalent single axle load (ESAL) was assigned based on Table 3.1 of the APAO manual considering a "Level III (Moderate)" traffic classification. This value is at the upper limit of the anticipated traffic demand. The APAO manual includes "Urban Minor Collectors" under Level III traffic classification. *Detailed traffic loading information was not provided for our review*. *If an increased traffic load is estimated, please contact us so that we may refine the traffic loading and revise our recommendations, if warranted.*
- The value used for drainage coefficients (m_n) was selected in accordance with Table 2.4 of the referenced AASHTO manual, based on "good" drainage characteristics of the base and subgrade materials. This quality of drainage was selected based on the unsaturated nature of the pavement materials during our investigation in August 2024.
- The value used for design reliability (R) and standard deviation (S_o) was selected in accordance with Table 11 and Section 5.3.3, respectively, of the referenced ODOT design manual.

C.4.3 Results of Analyses

Using the above inputs and procedures presented by AASHTO, we calculated the structural numbers for the subject street. The following table summarizes the results of our analyses:

Table C4 Calculated Structural Numbers for Classic Street

Area of Interest	Calculated Structural Number		
	SN_{eff}	SN_f	SN_{ol}
Classic Street, between Dorcas Lane and Necarney City Road	2.3	2.35	0.05

C.5.0 REVIEW & DISCUSSION

As indicated above, we completed a structural capacity evaluation of the subject portion of the roadway (Classic Street) to determine whether structural enhancement (such as an overlay) was required to accommodate design vehicular traffic when considering the 20-year design period. Our analyses indicated that, for the modeled design ESAL, the effective structural number (SN_{eff}) for the existing pavement is slightly below the required future structural number (SN_f). Accordingly, structural enhancement is required to accommodate the indicated vehicular traffic and maintain the desired level of serviceability.

Within the context of this assignment, it is our opinion that improvement to the pavement structure within the existing roadway is warranted to support the indicated vehicular traffic over the design period of 20 years. Recommendations for roadway improvements are presented in the main body of the geotechnical report.

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FIGURE C1

Site Plan

LEGEND

1 Orientation of site photographs shown on Figure C2

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NOTES: Drawing based on observations made while on site. 2023 aerial image from ArcGIS (www.arcgis.com). All photograph locations are approximate.

1 Inch = 250 Feet

0 250 500

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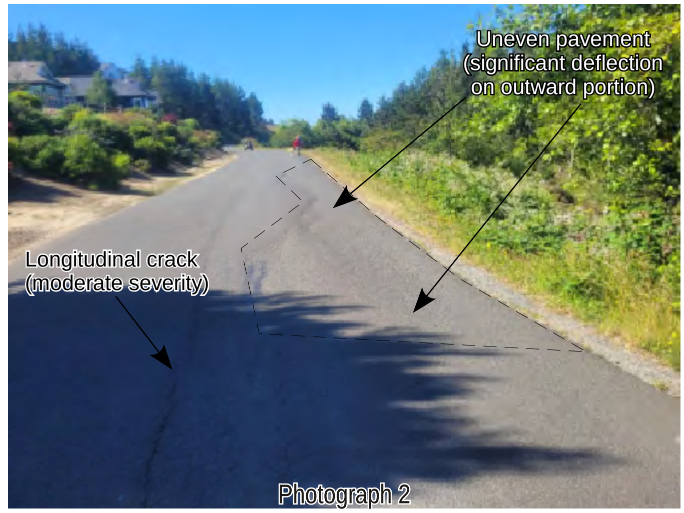
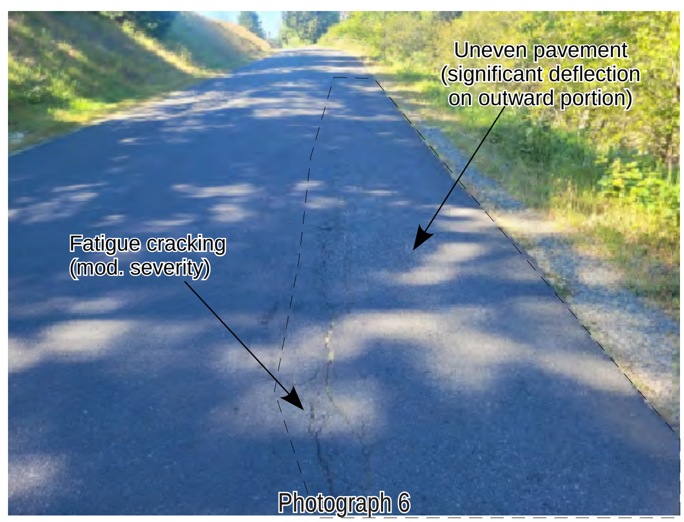
FIGURE C2

Site Photographs





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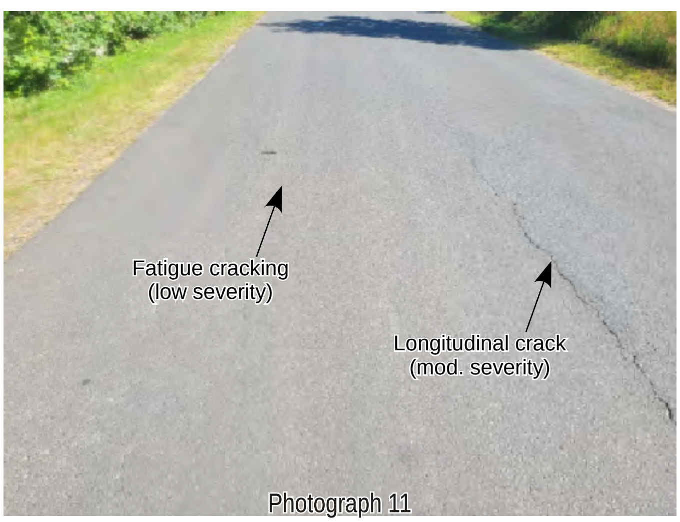
Photographs were taken at the time of our fieldwork on June 25, 2024.

Photograph 7

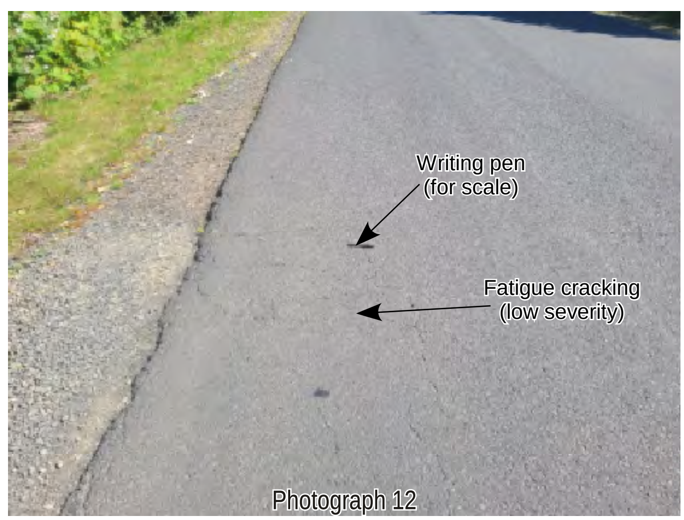
Photograph 8

Photograph 9

Photograph 10

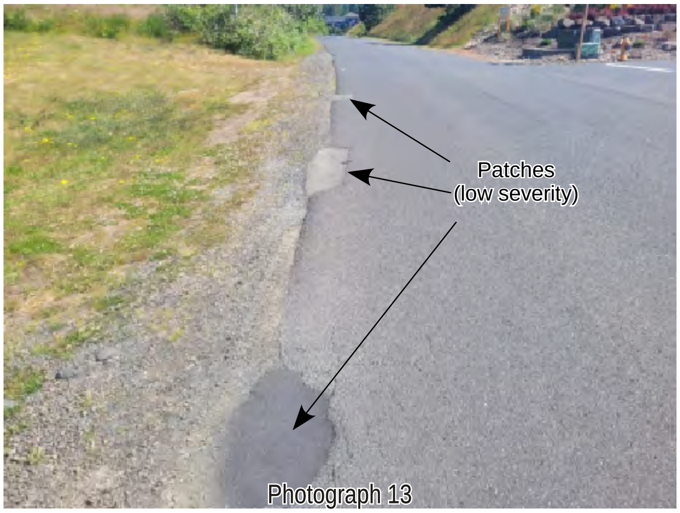


Photograph 11

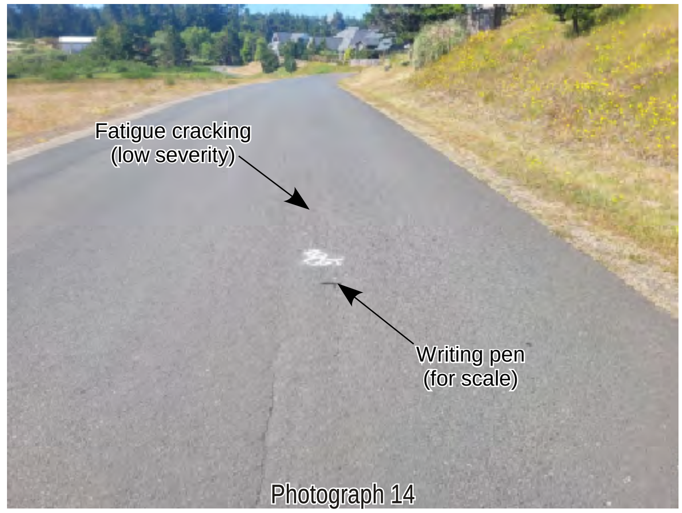


Photograph 12

Photographs were taken at the time of our fieldwork on June 25, 2024.



Photograph 13



Photograph 14

Photograph 15

Photograph 16

Photograph 17

Photograph 18

Photographs were taken at the time of our fieldwork on June 25, 2024.

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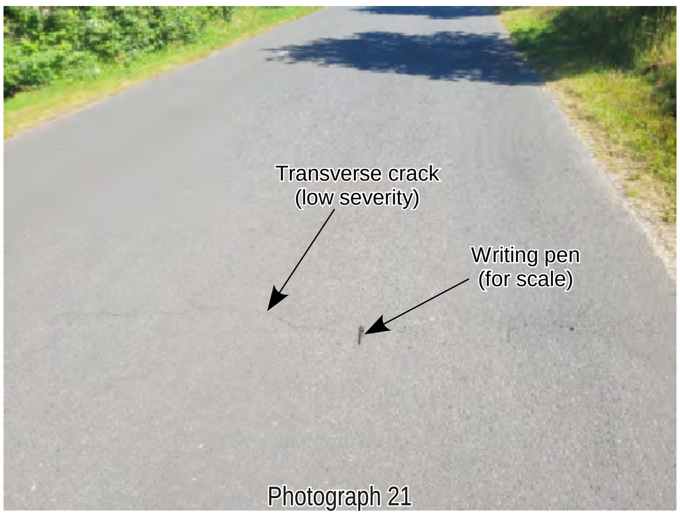
FIGURE C2 (cont.)
Site Photographs

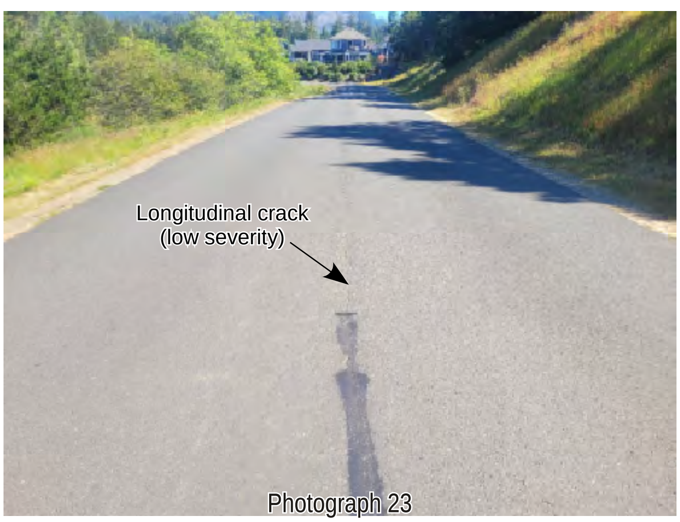
Photograph 19

Photograph 20



Photograph 21

Photograph 22



Photograph 23

Photograph 24

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FIGURE C2 (cont.)

Site Photographs

Photograph 25

Photograph 26

Photograph 27

Photograph 28

Photograph 29

Photograph 30

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Photographs were taken at the time of our fieldwork on June 25, 2024.