30% CD
STORMWATER REPORT

Third Street Improvements (NE Adams to NE Johnson Street)
McMinnville, OR, 97218

April 28, 2025

Prepared by:

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JAN. 12, 2021
JASON T. WHITE

EXPIRES: 6/30/2026

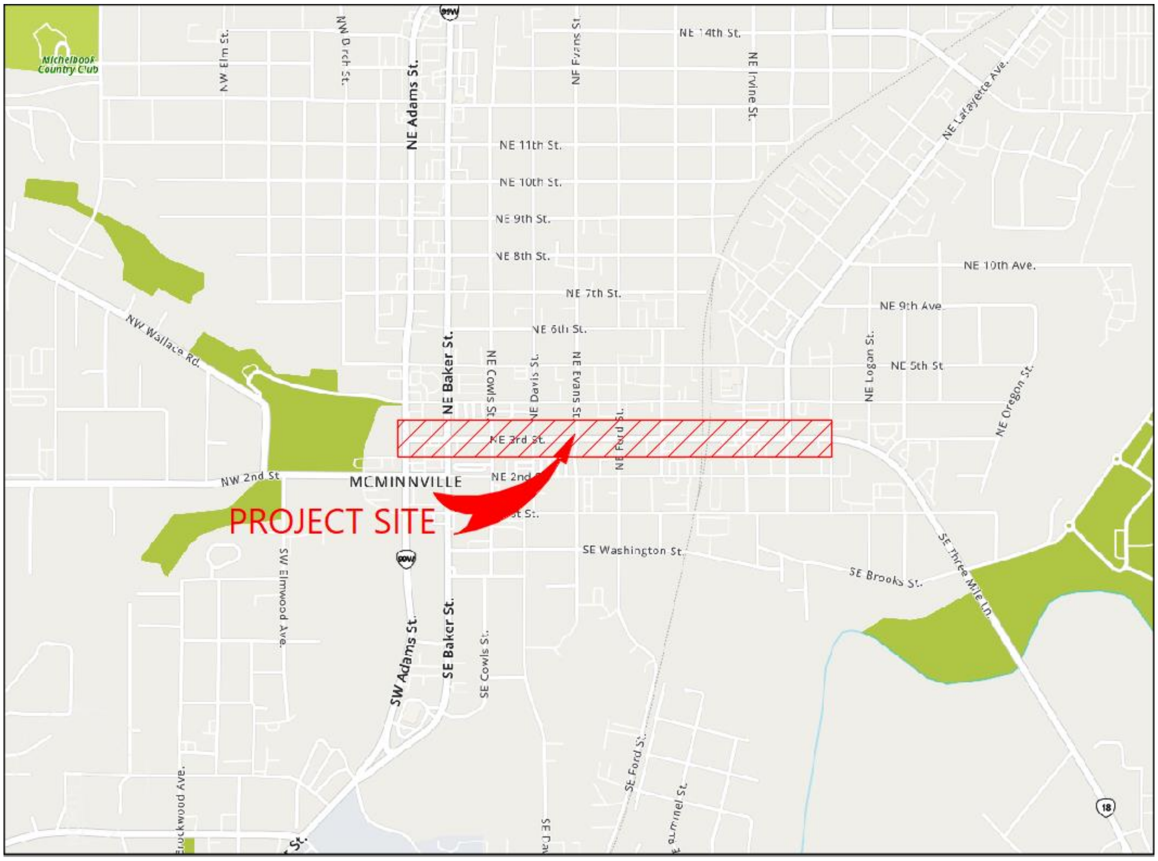
TABLE OF CONTENTS

Attachments

I. PROJECT DESCRIPTION

The proposed Third Street Improvement project consists of rehabilitating nine blocks of the downtown area from Adams Street to Johnson Street, with surface and utility improvements throughout. Complete surface improvements are proposed along the entirety of the nine blocks of Third Street and within adjacent intersections, including new sidewalk, paver and permeable paver pedestrian flexible space, raised concrete roadway and parking with a flush curb condition and concrete valley gutter, pedestrian crossings, new trees and other plantings, and other furnishings throughout. Utility improvements to the storm, water, sanitary sewer, gas, and electrical are proposed throughout the project limits. This report will focus on the proposed stormwater improvements.

Existing storm mains are to be replaced due to age, condition, and location. New main lines will be relocated into the public street and out of sidewalks and curb lines and new inlets will be located at low points in the roadway. Where feasible, stormwater will be routed into soil cell systems below the sidewalk to be treated prior to it entering the downstream system.



Vicinity Map: Limits of Third Street Improvements

II. EXISTING CONDITIONS

The existing land is 9-blocks of fully developed downtown corridor area from NE Adams Street to NE Johnson Street. There are four existing ultimate discharge locations associated with the project. NE Adams Street to Baker Street discharge into Cozine Creek via a storm main that extends west, past Third Street. From NE Baker Street to NE Evans Street, the existing storm system discharges to NE 4th Street. From NE Evans to NE Galloway Street the existing storm system discharges to NE

1st Street. The existing storm drain systems in both 1st and 4th Streets are conveyed to their eventual discharge locations in North Cozine Creek. NE Irvine Street to NE Johnson Street are conveyed east to an existing storm system in Third Street where it flows east to its eventual discharge in the South Yamhill River. See Attachment A in the appendix for the City of McMinnville Storm Sewer Map and enlargement of the project area.

Due to the developed nature of the existing land, limited infiltration testing has been conducted at this time. The two borings that were completed, show limited infiltration rates at approximately 3 feet of depth, ranging from 1.5-2.3 inches per hour. The on-site soil at 1-3 feet of depth was found to be dark lean clay, the clay is underlain by soft to medium stiff silt and elastic silt with variable amounts of sand extending to depths of up to 11.5 below ground surface (bgs). Due to low unfactored infiltration rates and low hydraulic conductivity, the geotechnical engineer does not recommend infiltration as a stormwater management approach (Attachment D). In general, groundwater was not encountered during shallow explorations, however, one boring A-11 (near the existing railroad corridor) encountered groundwater at approximately 9 feet below ground surface (bgs). Results of geotechnical laboratory moisture content testing did not imply presence of groundwater in in other explorations. Based on Oregon Water Resources Department records, the groundwater in the project corridor is typically 3 to 10 feet bgs, and likely fluctuates seasonally during the wet season (November-May). The Geotechnical Report documenting this information is included as Attachment D of this report.

III. PROPOSED CONDITIONS

This report and stormwater design are preliminary and based on 30% design plans. The proposed Third Street design is a curb-less design with a crowned centerline. All stormwater from the roadway and sidewalks will flow towards a concrete valley gutter at the roadway/parking limits. Storm inlets will be located at low points along the valley gutters, not exceeding ODOT's maximum inlet spacing requirements.

Where feasible, stormwater will then be routed into soil cell systems below the sidewalk to be treated prior to it entering the downstream system. As infiltration is not recommended, the soil cell systems will be lined and include underdrains, allowing the stormwater to enter the outlet pipes once treated. Although the proposed project will be designed to treat all stormwater runoff before it enters the downstream public storm system, there are some locations where soil cells are not feasible and the specifics for treatment in those areas are still to be determined. This will be studied further in later stages of the design.

After treatment, stormwater will enter the City's conveyance system and generally will remain in the same watersheds as existing conditions. Pipes have been sized to account for additional storage that could typically be accommodated in the gutter of a curbed roadway.

The total tributary area for these improvements is approximately 12 acres – this area includes the improved right-of-way, roof drains, and all adjacent property that drains into the project area.

IV. REFERENCES

City of McMinnville Storm Drainage Design and Construction Standards (COM SDDCS)

City of McMinnville Stormwater Standards (COM SS)

City of McMinnville Storm Drainage Master Plan (COM SDMP)

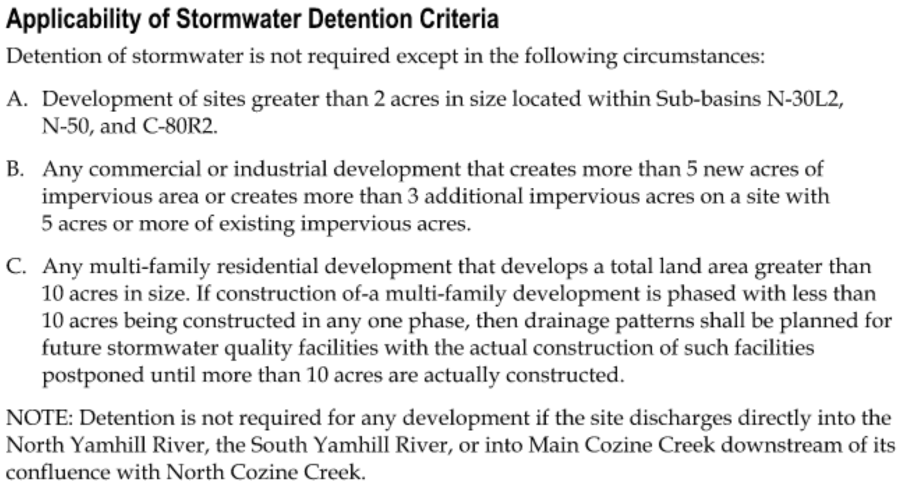
ODOT Hydraulics Manual (Chapter 7)

Geotechnical Report: Report of Geotechnical Engineering Services for Third Street Improvements: NE Third Street to NE Johnson Street, prepared by Haley and Aldrich for BKF Engineers, dated July 2024.

V. DESIGN AND CONSTRUCTION STANDARDS

The proposed development will provide stormwater management in accordance with Chapter 7 of the ODOT Hydraulics Manual, as well as Appendix E of the COM SDDCS. Per Appendix E of the COM SDDCS, detention of stormwater is not required since circumstances A, B, and C do not apply (see detention criteria below). Compliance with the COM SDMP has also been implemented by proposing drainage improvements for known conveyance inadequacies described in the report.

Per the COM SDDCS Appendix E, this project is not subject to stormwater detention requirements.



Applicability of Stormwater Detention Criteria

Detention of stormwater is not required except in the following circumstances:

A. Development of sites greater than 2 acres in size located within Sub-basins N-30L2, N-50, and C-80R2.

B. Any commercial or industrial development that creates more than 5 new acres of impervious area or creates more than 3 additional impervious acres on a site with 5 acres or more of existing impervious acres.

C. Any multi-family residential development that develops a total land area greater than 10 acres in size. If construction of-a multi-family development is phased with less than 10 acres being constructed in any one phase, then drainage patterns shall be planned for future stormwater quality facilities with the actual construction of such facilities postponed until more than 10 acres are actually constructed.

NOTE: Detention is not required for any development if the site discharges directly into the North Yamhill River, the South Yamhill River, or into Main Cozine Creek downstream of its confluence with North Cozine Creek.

Per the COM SDDCS standards, this basin is <100 acres in size, and therefore requires the Rational Method Design procedure.

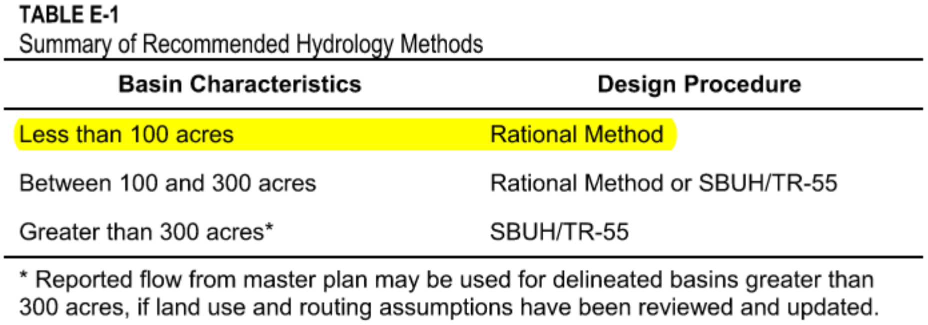


TABLE E-1
Summary of Recommended Hydrology Methods

Basin Characteristics	Design Procedure
Less than 100 acres	Rational Method
Between 100 and 300 acres	Rational Method or SBUH/TR-55
Greater than 300 acres*	SBUH/TR-55

* Reported flow from master plan may be used for delineated basins greater than 300 acres, if land use and routing assumptions have been reviewed and updated.

VI. HYDROLOGY AND CONVEYANCE

The City of McMinnville SDDCS and ODOT Hydraulics Manual (Chapter 7) were used to design the proposed stormwater conveyance systems. Given the COM SDDCS design storm frequency requirements (upstream watershed <100 acres), the proposed conveyance system is sized to handle the 10 to 50-year storm events using the rational method. A proposed basin exhibit/block map and calculations have been provided in the Attachment A of this report.

Conveyance Sizing Requirements:

The Rational Method is used to size pipes per COM SDDCCS Appendix E. Per the City of McMinnville stormwater standards, the proposed project is classified as a "small and moderate pipe system" within the public right-of-way that serves an upstream watershed no more than 320 aces and is therefore required to accommodate the 10-year storm event. A portion of the system also contains sag curves within the ODOT right-of-way, and therefore a 50-year design event would be applied in those areas. Per the COM SDMP, existing downstream storm main deficiencies have been identified. To mitigate for potential downstream deficiencies, a hydraulic grade line (HGL) analysis was performed and applied to the conveyance design to ensure pipes would adequately convey the 50-year storm event, without backing up into any portions of the proposed surface improvements. Until a more thorough investigation of the downstream conditions can be identified, some conservative assumptions have been made and applied to the conveyance design (see below).

Conveyance Sizing Method and Findings:

For the conveyance design, the peak rainfall intensity and duration for 50-year storm was used from the ODOT IDF Curve table, Zone 8 (ODOT Hydrology Manual, Appendix A). The runoff coefficient of 0.85 for "City Business Areas" was selected from Table 1 in Appendix F of Chapter 7 of the ODOT Hydrology Manual. For this preliminary analysis, two (2) project areas were selected to be assessed which would represent the most conservative sizing for the project. Based on the preliminary grading and drainage patterns, the largest tributary drainage area to Cozine Creek was identified as Block D (from Davis Street to Evans Street). The largest tributary drainage area to Yamhill River was identified as Blocks H through J (east of the railroad, to Johnson street). Per the proposed basin exhibit/block map in Attachment A, the drainage area for Block D represents approximately 1.62 acres. The drainage area for Blocks H-J represents approximately 3.04 acres. A time of concentration (TC) of 5 minutes was assumed for each tributary drainage area, and a minimum pipe slope of 0.5% was assumed for existing and proposed conditions. For this exercise, a user defined tailwater elevation was set at the existing downstream pipe connection point for each basin, assuming that the existing pipe would be half full. Based on the assumptions and parameters outlined above, the HGL for the 50-year storm event was modeled using Bentley StormCAD software (see calculations in Attachment C). Based on this analysis, it was determined that all proposed storm mains from Adams Street to the railroad will need to be 18" in order to convey the required amount of runoff for the proposed development in the Third Street right-of-way, without backing up into the street for the selected storm event.

From the railroad east to the connection point in Johnson Street, the storm main from blocks H-J will need to be 24" diameter, given its larger tributary area (see Block H-J Profile, attachment C). Further studies will need to be conducted to confirm the capacity of the existing downstream storm mains, and also to further refine the proposed storm main sizes for each proposed block/basin as the project progresses to the next phases.

DESIGN STORM INTENSITIES AND RUNOFF COEFFICIENT (ODOT HYDROLOGY MANUAL, CHAPTER 7)		
Design Storm (per Zone 8 IDF Curves)	**Intensity (i) (in/hr)**	**Runoff Coefficient (C) (City Business Area)**
10-year 24-hour	2.3	0.85
50-year 24-hour	3	0.85

VII. STORMWATER RUNOFF TREATMENT

The City of McMinnville SDDCS was used to design the proposed stormwater treatment facilities. For water quality treatment, the soil cell facilities are designed using a 6% simplified sizing factor. The simplified sizing factor is being used for the preliminary water quality sizing only. Soil cells have been designated by the Washington Department of Ecology as being functionally equivalent to a bioretention facility.

Water Quality Requirements:

Due to anticipation of the project being federally funded and portions of the project being within the ODOT right-of-way, stormwater treatment is proposed to manage all stormwater runoff.

Stormwater Management (SWM) Facility Types:

For the 30% construction documents, proprietary soil cell water quality treatment technology is proposed. The soil cells have been proposed in an effort to provide as much growing medium for large trees in the right-of-way as possible. Large trees are proposed throughout the proposed Third Street Improvements Plan – soil cells provide room for large root systems to grow while also providing low impact storm water quality treatment. Preliminary footprints for the soil cell systems are shown in Attachment C of this report.

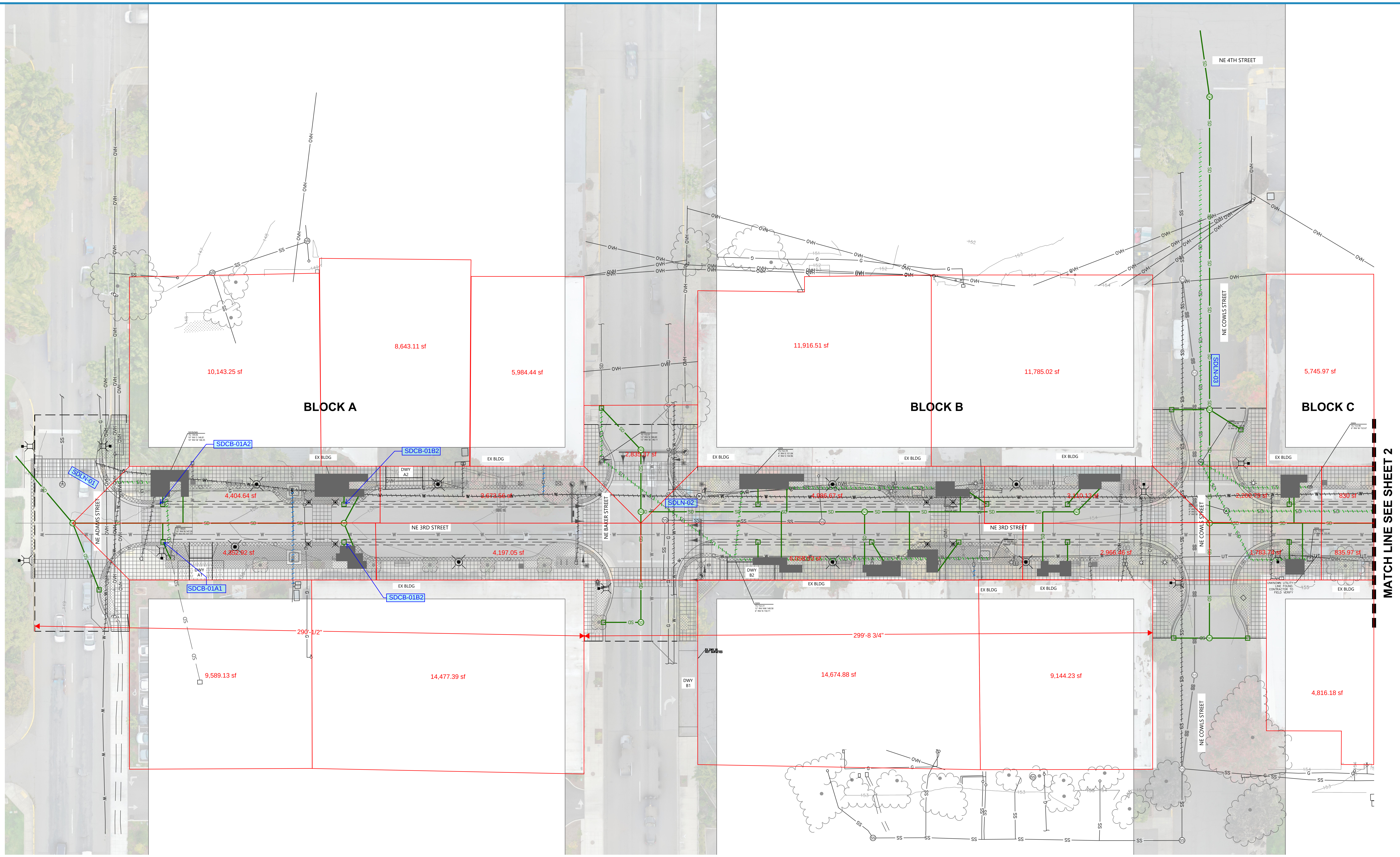
Other treatment options will be explored as the design progresses through the construction documents phase.

VIII. SUMMARY

Due to existing downstream deficiencies described in the City of McMinnville Stormwater Master Plan, and portions of the project site existing within sag curves in the ODOT right-of-way, all existing storm mains within the 9-block project area will be replaced. For the 30% construction documents, conservative assumptions have been made for the proposed conveyance design – a more robust downstream analysis is required to assess all downstream conditions. A hydraulic grade line (HGL) analysis was performed and applied to the conveyance design to ensure pipes can adequately convey the 50-year storm event, without backing up into any portions of the proposed surface improvements. This study results in a proposed 18” storm main from Adams Street to the Railroad tracks, and a 24” storm main from the Railroad east to the downstream connection point in Johnson Street.

Per City of McMinnville SDDCS requirements, this project is not subject to detention requirements. All stormwater runoff will be treated to meet ODOT and federal funding requirements – for the 30% construction documents, proprietary soil cell technology is proposed as the primary treatment method. The proposed soil cells have been preliminary sized using a 6% sizing factor.

ATTACHMENT A



LEGEND:

— — —	PROPERTY LINE	●	PROPOSED STORM DRAIN MANHOLE
—— – ——	CENTER LINE	☐	PROPOSED STORM DRAIN INLET
—W—	PROPOSED WATER MAIN	☐	HISTORICAL CISTERN
—W—	PROPOSED WATER LATERAL	◉	HISTORICAL WATER SUPPLY WELL
—FW—	PROPOSED FIRE SERVICE LATERAL	⊠	PROPOSED STREET LIGHTS
—SS—	PROPOSED SANITARY SEWER LINE	☐	HISTORICAL CISTERN
—SD—	PROPOSED STORM DRAIN LINE	◉	HISTORICAL WATER SUPPLY WELL
\\\\\\\\	REMOVE OR ABANDON EXISTING WATER LINE		
\\\\\\\\	REMOVE OR ABANDON EXISTING SANITARY SEWER LINE		
\\\\\\\\	REMOVE OR ABANDON EXISTING STORM DRAIN LINE		

20 0 20 40
scale 1" = 20' feet

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THIRD STREET IMPROVEMENT PROJECT
NE THIRD STREET (ADAMS STREET TO JOHNSON STREET), MCMINNVILLE, OREGON 97128
YAMHILL COUNTY

STORM SIZING EXHIBIT

DATE 2/16/24	SCALE 1" = 20'
CLIENT CITY OF MCMINNVILLE	
EXHIBIT NO. 20221310-000	
SHEET 1 OF 4	

Plot Apr 28, 2025 at 4:17pm

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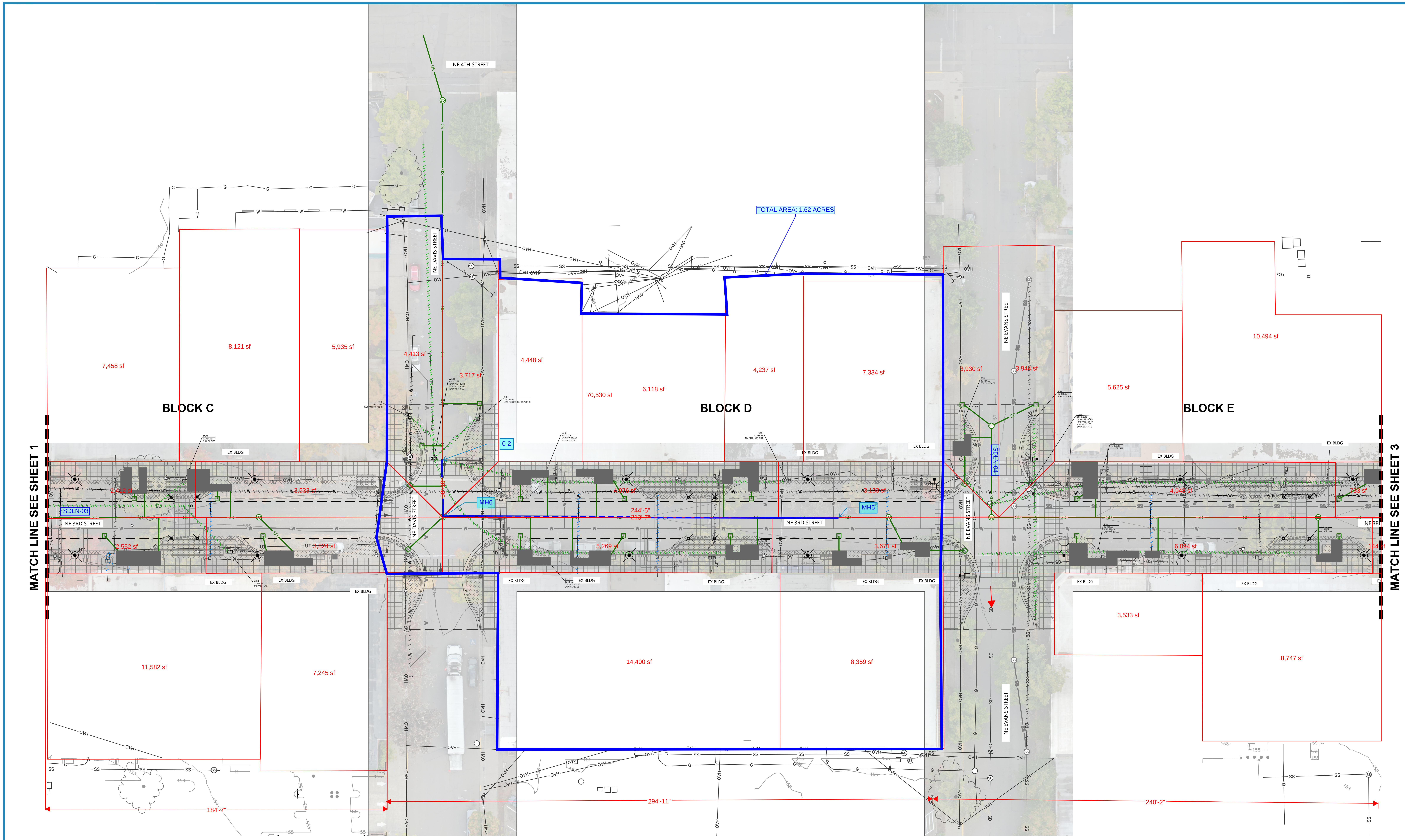

LEGEND:

—— — — ——	PROPERTY LINE	◎	PROPOSED STORM DRAIN MANHOLE
——— — ———	CENTER LINE	▣	PROPOSED STORM DRAIN INLET
——W——	PROPOSED WATER MAIN	□	HISTORICAL CISTERN
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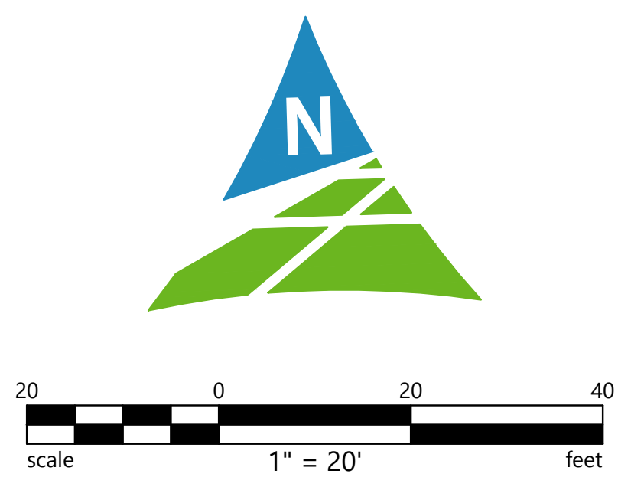

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SHEET 2 OF 4	

Plot Apr 28, 2025 at 4:18pm

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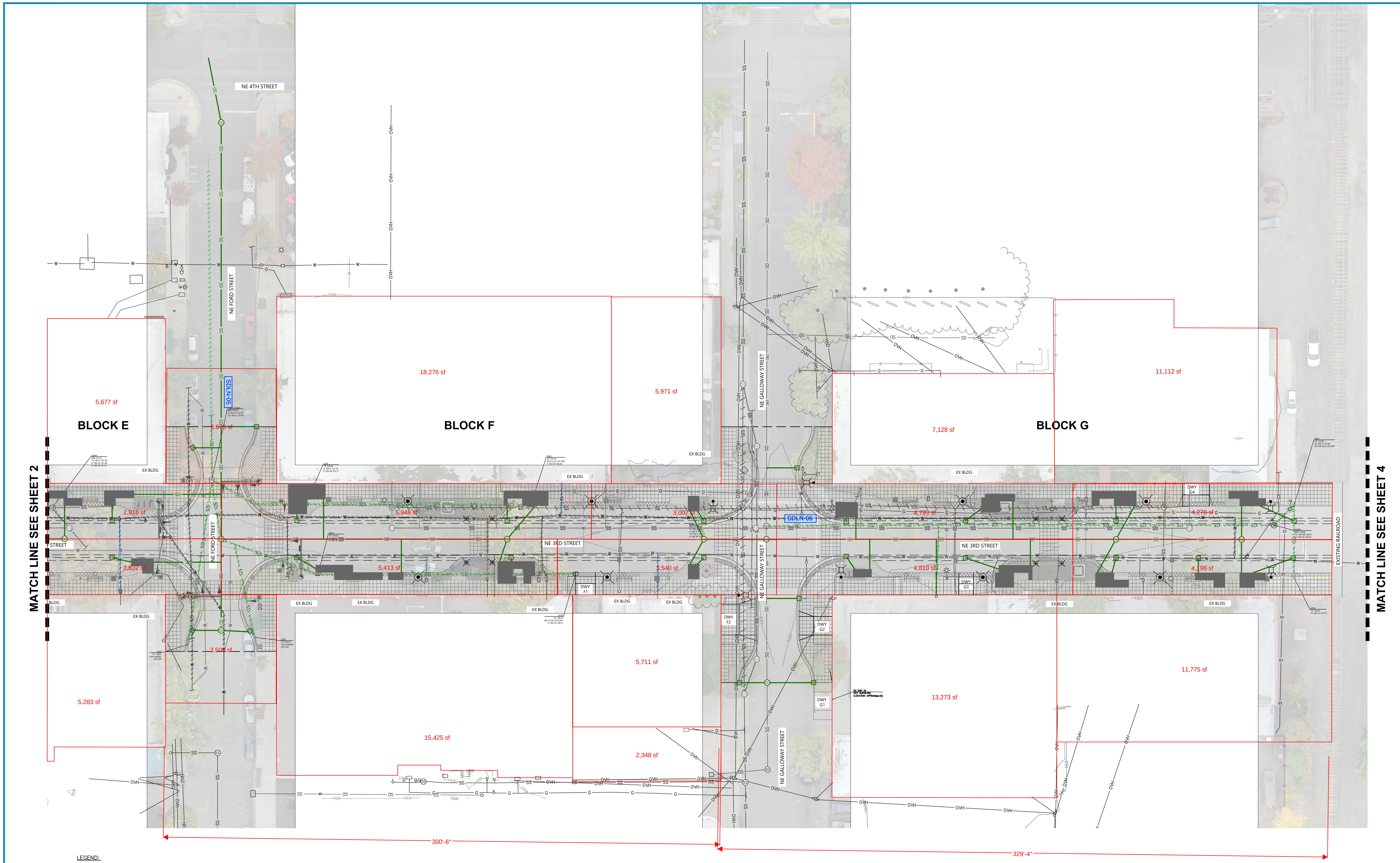

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—— — ——	CENTER LINE
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—W—	PROPOSED WATER LATERAL
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20221310-000
SHEET
3 OF 4

Plot Apr 28, 2025 at 4:18pm
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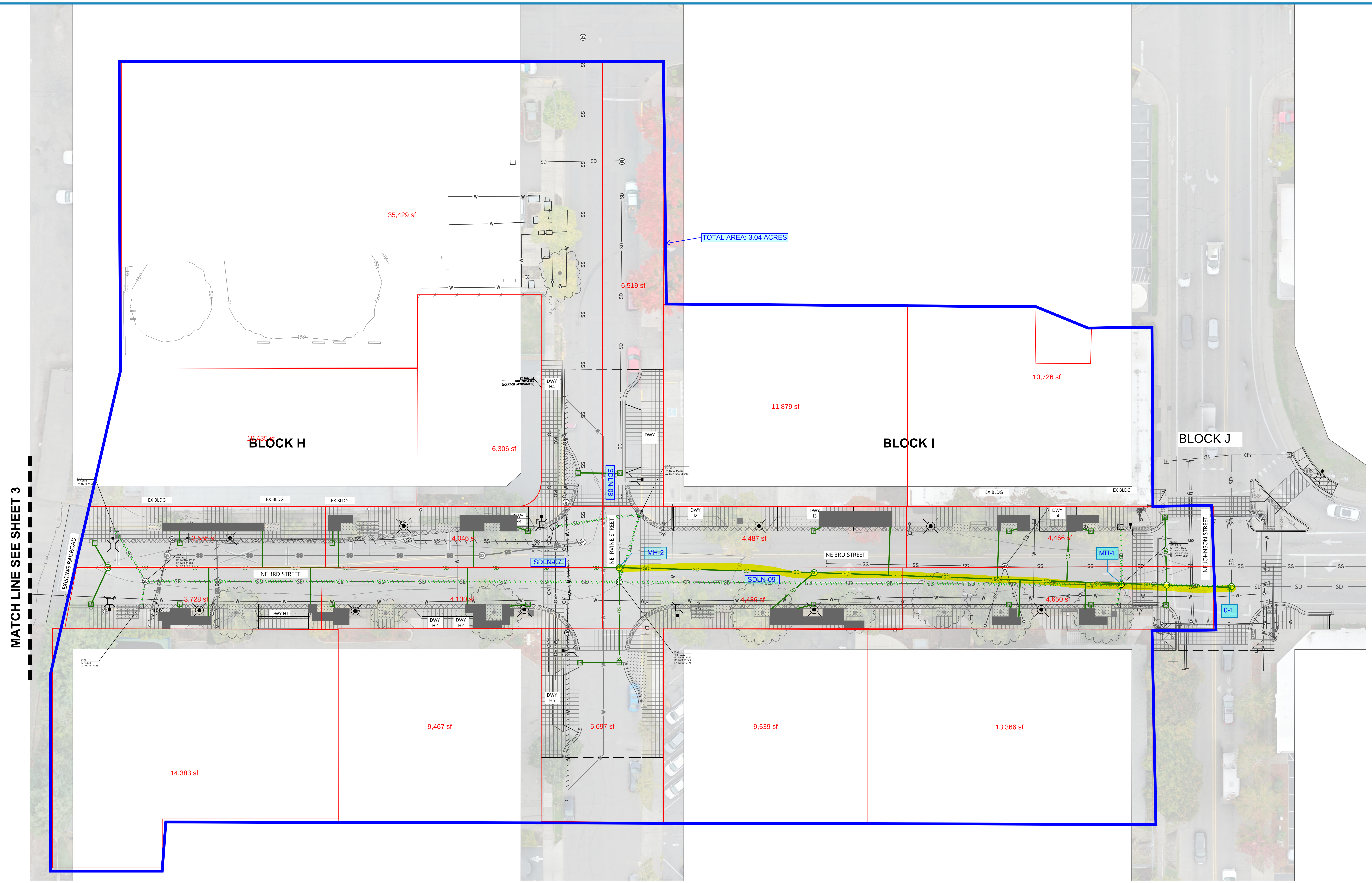

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CLIENT CITY OF MCMINNVILLE	
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SHEET 4 OF 4	

Plot Apr 28, 2025 at 4:18pm

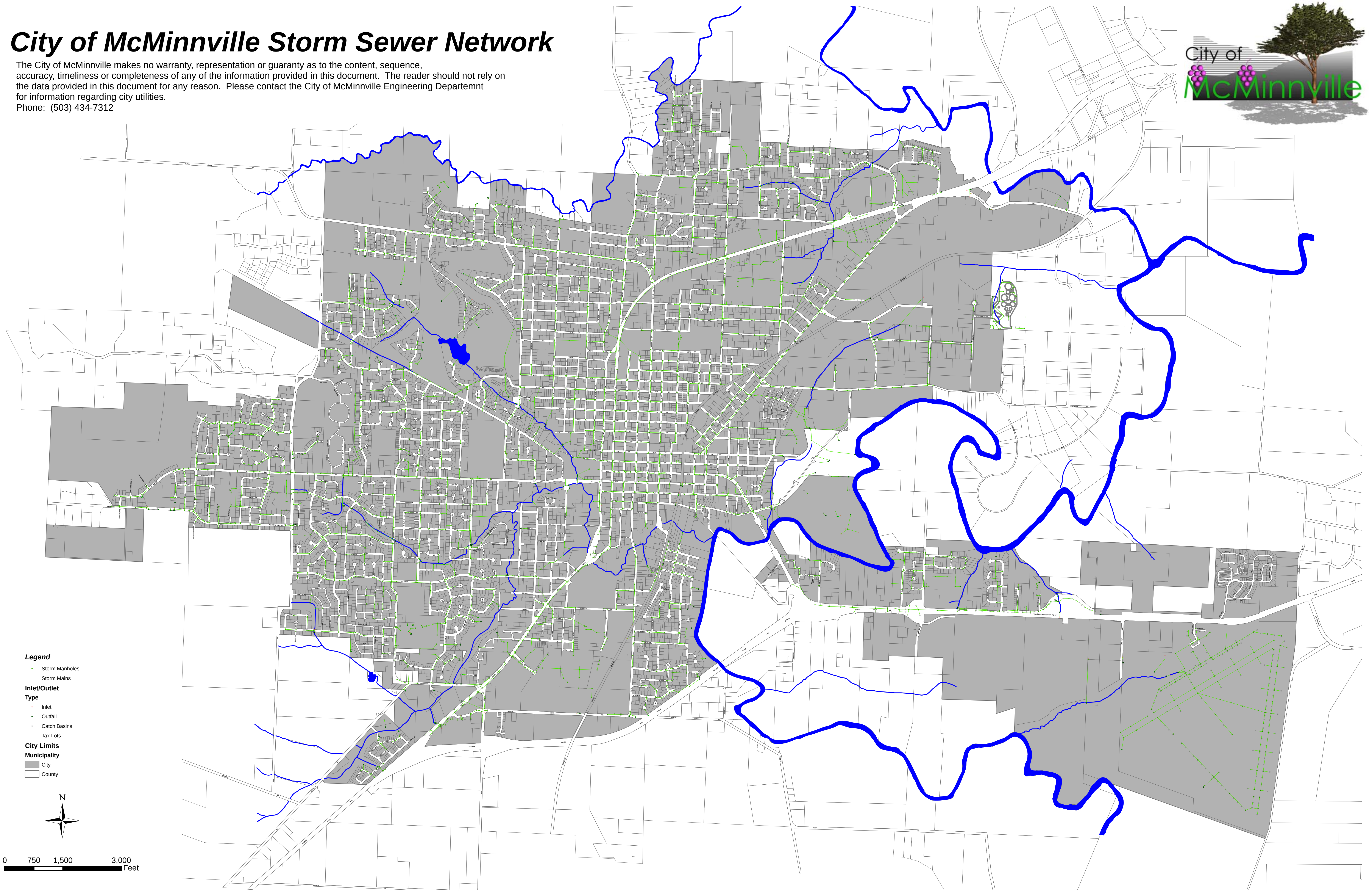
221310_Utility Sizing Exhibit.dwg

ATTACHMENT B

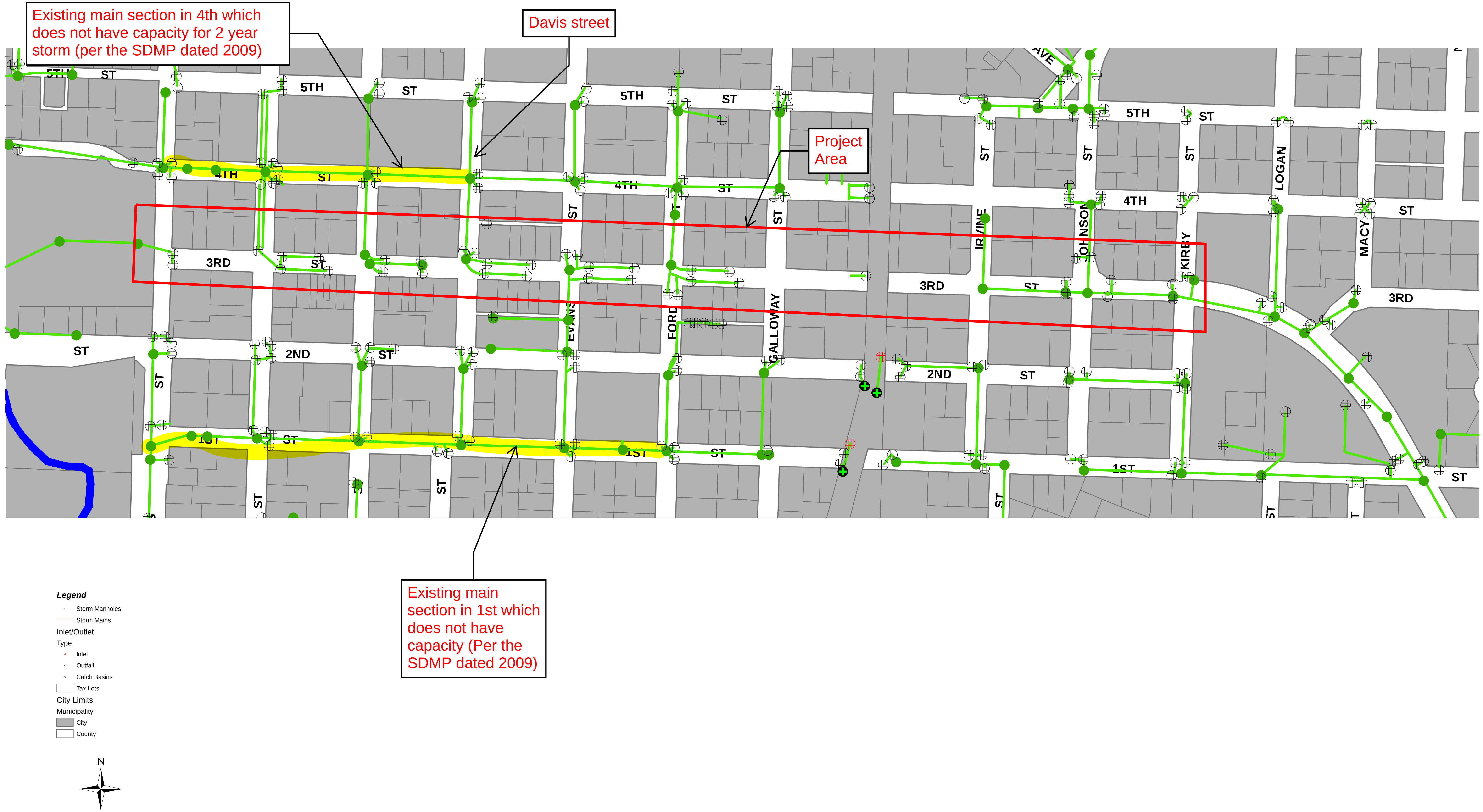
City of McMinnville Storm Sewer Network

The City of McMinnville makes no warranty, representation or guaranty as to the content, sequence, accuracy, timeliness or completeness of any of the information provided in this document. The reader should not rely on the data provided in this document for any reason. Please contact the City of McMinnville Engineering Departemnt for information regarding city utilities.
Phone: (503) 434-7312

City of McMinnville



City of McMinnville Storm Sewer Network ***ENLARGMENT***



Existing main section in 4th which does not have capacity for 2 year storm (per the SDMP dated 2009)

Davis street

Project Area

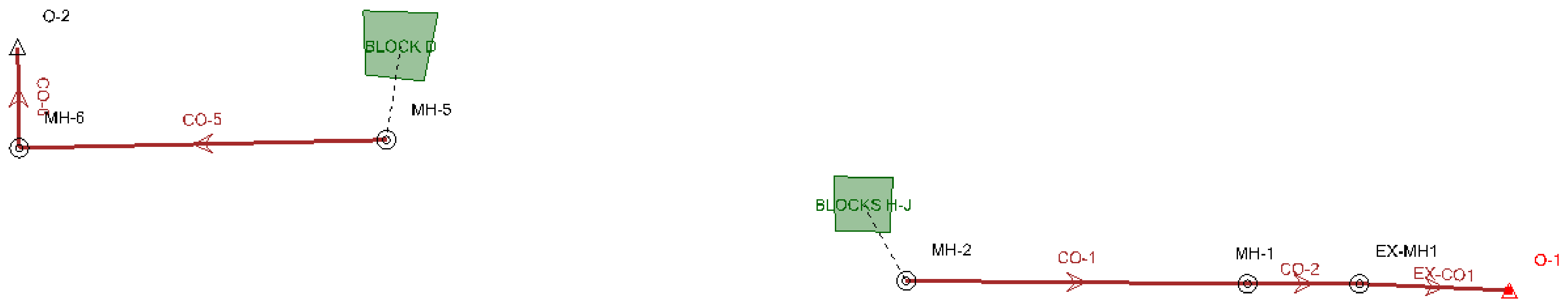
Existing main section in 1st which does not have capacity (Per the SDMP dated 2009)

ATTACHMENT C

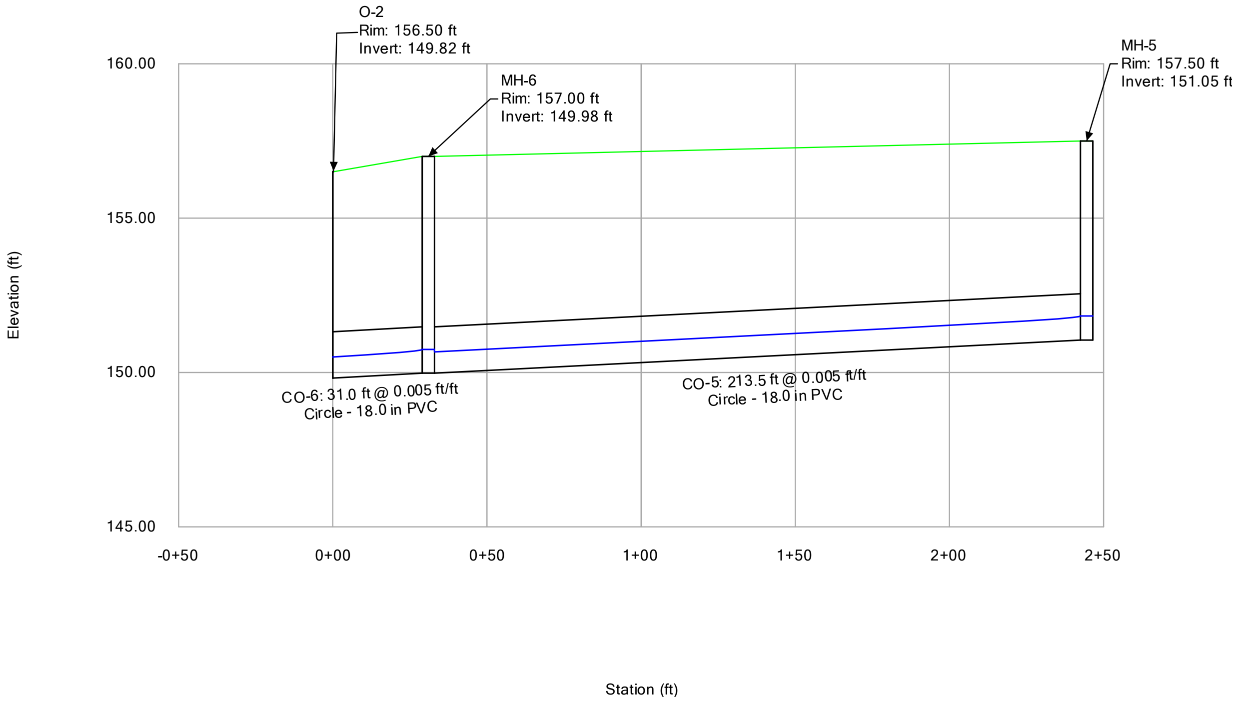
Conduit Flex Table - 50 year storm event																				
ID	Label	Start Node	Set Invert to Start?	Invert (Start) (ft)	Stop Node	Set Invert to Stop?	Invert (Stop) (ft)	Has User Defined Length?	Length (User Defined) (ft)	Length (Scaled) (ft)	Slope (Calculated) (ft/ft)	Section Type	Diameter (in)	Manning's n	Flow (cfs)	Velocity (ft/s)	Depth (Out) (ft)	Capacity (Full Flow) (cfs)	Flow / Capacity (Design) (%)	Depth (Normal) / Rise (%)
33	CO-1	MH-2	FALSE	153.05	MH-1	FALSE	151.82	TRUE	247	109.6	0.005	Circle	24	0.01	11.72	3.73	6.06	20.75	56.5	53.8
52	CO-5	MH-5	TRUE	151.05	MH-6	TRUE	149.98	TRUE	213.5	117.9	0.005	Circle	18	0.01	4.16	5.26	0.98	9.67	43.1	45.9
53	CO-6	MH-6	TRUE	149.98	O-2	TRUE	149.82	TRUE	31	32.6	0.005	Circle	12	0.01	4.03	5.13	0.85	3.33	121.2	(N/A)
56	CO-2	MH-1	TRUE	151.82	EX-MH1	FALSE	151.55	TRUE	54	36	0.005	Circle	24	0.01	11.37	3.62	6.25	20.79	54.7	52.7
60	EX-CO1	EX-MH1	TRUE	151.55	O-1	TRUE	151.05	TRUE	100	48.2	0.005	Circle	12	0.01	11.29	14.37	1	3.27	344.7	(N/A)

Manhole Flex Table- 50 year storm event													
ID	Label	Elevation (Ground) (ft)	Set Rim to G	Elevation (Rim) (ft)	Bolted Cover	Elevation (Invert in 1) (ft)	Flow (Total In)	Flow (Total Out) (cfs)	Depth (Out) (ft)	Hydraulic Grade Line (Out) (ft)	Headloss Method	Hydraulic Grade Line (In) (ft)	Notes
31	MH-1	158.02	TRUE	158.02	FALSE	151.82	11.72472	11.37	6.06	157.88	Absolute	157.88	
32	MH-2	159.2	TRUE	159.2	FALSE	(N/A)	11.72472	11.72	5.65	158.27	Absolute	158.27	
49	MH-5	157.5	TRUE	157.5	FALSE	(N/A)	4.164048195	4.16	0.78	151.83	Absolute	151.83	
50	MH-6	157	TRUE	157	FALSE	149.98	4.164048195	4.03	0.98	150.96	Absolute	150.96	
55	EX-MH1	157.8	TRUE	157.8	FALSE	151.55	11.36981106	11.29	6.25	157.8	Absolute	157.8	

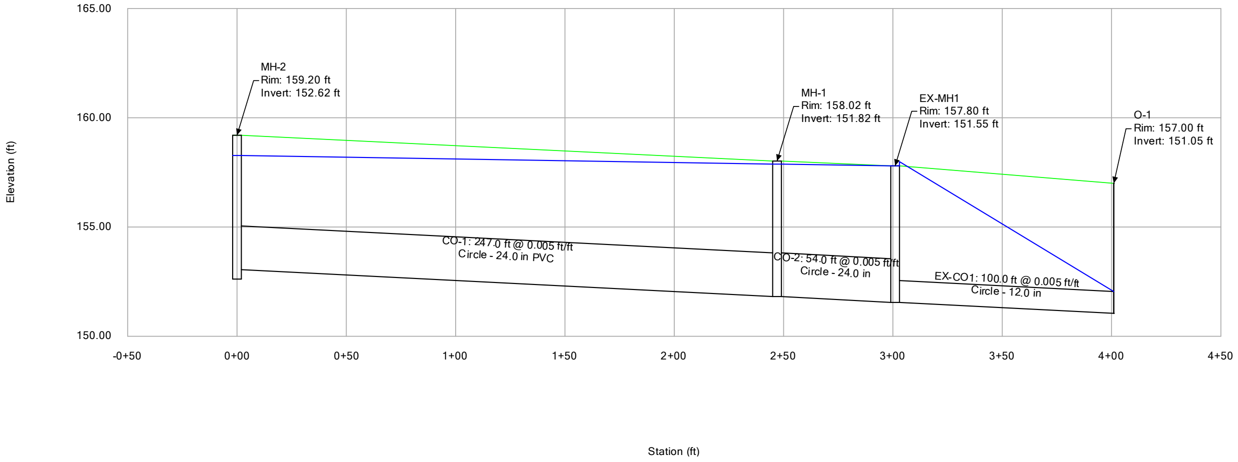
Outfall Flex Table- 50 year storm event										
ID	Label	Elevation (Ground) (ft)	Set Rim to Ground Elevation?	Elevation (Invert) (ft)	Boundary Condition Type	Boundary Element	Elevation (User Defined Tailwater) (ft)	Hydraulic Grade (ft)	Flow (Total Out) (cfs)	Notes
51	O-2	156.5	TRUE	149.82	User Defined	<None>	150.48	150.67	4.01	
59	O-1	157	TRUE	151.05	User Defined	<None>	152.05	152.05	11.25	



Profile Report
Engineering Profile - BLOCK D (PRELIM.stsw)



Profile Report
Engineering Profile - BLOCK H-J (PRELIM.stsw)



ATTACHMENT D

REPORT OF GEOTECHNICAL ENGINEERING SERVICES FOR
THIRD STREET IMPROVEMENTS
NE THIRD STREET - NE ADAMS STREET TO NE JOHNSON STREET
MCMINNVILLE, OREGON

by
Haley & Aldrich, Inc.
Portland, Oregon

for
BKF Engineers
Portland, Oregon

File No. 0208183-000
July 2024

HALEY & ALDRICH, INC.
6420 S. Macadam Avenue
Suite 100
Portland, OR 97239-3517
503.620.7284

25 July 2024
File No. 0208183-000

BKF Engineers
1125 NW Couch Street, Suite 420
Portland, Oregon 97209

Attention: Jason White, P.E., LEED AP
Principal/Vice President

Subject: Report of Geotechnical Engineering Services
Third Street Improvement Project
NE Third Street - NE Adams Street to NE Johnson Street
McMinnville, Oregon

Dear Jason White:

Haley & Aldrich, Inc. (Haley & Aldrich) is pleased to submit to BKF Engineers our report of geotechnical engineering services for the Third Street Improvement Project (Project) in McMinnville, Oregon.

We understand the City of McMinnville intends to perform improvements to NE Third Street between NE Adams Street and NE Johnson Street (Project Corridor) in downtown McMinnville. The Project limits span a total of nine city blocks. The street sections within the Project Corridor typically consist of two bi-directional traffic lanes with parking lanes and a center turn lane in the eastern portion of the Project Corridor. While specific project details have not yet been developed, construction activities in the Project Corridor will likely include street and sidewalk reconstruction, underground infrastructure improvements, above-ground street furnishings, and landscaping. The current street alignment is entirely asphalt-paved, and options for repaving include full reconstruction and rehabilitation.

The primary geotechnical issues affecting the design and construction of the planned improvements include the presence of soft and fine-grained subgrade soils, shallow groundwater, and an existing asphalt concrete over Portland cement concrete pavement section, which can present challenges for rehabilitation. Our recommendations regarding roadway and flatwork construction, stormwater infiltration features, site grading, and other geotechnical aspects of this project are presented in this report.

We appreciate the opportunity to provide our services to you on this project. If you have any questions, please contact the undersigned at 971.327.9115.

Sincerely yours,
HALEY & ALDRICH, INC.

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Enclosure

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SIGNATURE PAGE FOR

REPORT OF GEOTECHNICAL ENGINEERING SERVICES FOR
THIRD STREET IMPROVEMENTS
NE THIRD STREET - NE ADAMS STREET TO NE JOHNSON STREET
MCMINNVILLE, OREGON

PREPARED FOR
BKF ENGINEERS
PORTLAND, OREGON

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1. Introduction

Haley & Aldrich, Inc. (Haley & Aldrich) is pleased to submit this report to BKF Engineers (BKF) summarizing our geotechnical engineering services for the Third Street Improvements Project (Project) on NE Third Street between NE Adams Street and NE Johnson Street (Project Corridor), located in McMinnville, Oregon. We completed our work in general accordance with our agreement dated 18 August 2023.

While specific Project details have not yet been developed, construction activities for the Project will likely include street and sidewalk reconstruction, underground infrastructure improvements, above-ground street furnishings, and landscaping improvements to the City of McMinnville's (City's) historic downtown Third Street. The existing asphalt concrete pavements present throughout the Project Corridor are in poor to fair condition, showing signs of fatigue cracking, trench cut repairs, and undesired low points inhibiting drainage. Existing sidewalks consist of a mixture of concrete, tile, and brick, in variable conditions. Based on the City's 29 July 2022 Final Concept Design document, concepts for the improved Project Corridor favor repaving the street with Portland cement concrete and including on-street parallel parking zones, raised mid-block crosswalks, curb extension areas, and street and building frontage zones. Existing mature trees are likely to be removed and replaced with new trees, possibly taking advantage of buried soil cells to reduce potential for root damage to pavements. The typical 60-foot-wide right-of-way conceptually features 12 feet of sidewalk and 8 feet of curb extension or parking on each side of the street, as well as one 10-foot-wide vehicle travel lane running in each direction.

This report contains the results of our analyses and provides recommendations for design and construction of the proposed improvements. The first section of this report provides an overview of the Project information. The main body of the report presents our geotechnical engineering findings and design recommendations in detail. Figures are presented at the end of the text. The location of the site is shown on Figure 1, Vicinity Map, and the existing and proposed site layout is shown on Figure 2, Site and Exploration Plan. Appendix A describes our field exploration procedures and presents field data and logs. Appendix B describes our laboratory testing procedures and results. Appendix C presents photographs of the pavement cores and dynamic cone penetrometer (DCP) probe data and correlations.

2. Scope of Services

The purpose of our services was to evaluate the subsurface conditions along the Project Corridor and to develop geotechnical engineering recommendations for design and construction of the Project elements. We completed the following tasks in general accordance with the Consulting Agreement between BKF and Haley & Aldrich dated 18 August 2023:

- Reviewed readily available geologic, groundwater, and soil survey maps that cover the Project vicinity;
- Conducted a reconnaissance of the Project Corridor;
- Performed the following exploratory work to characterize as-built pavement and subsurface soil and groundwater conditions;
  - Completed 15 pavement cores at approximately 100 to 250 feet on center.
  - Conducted DCP testing at ten core locations through the underlying base rock and soil subgrade to depths of about 3 feet below surface grade to evaluate pavement subgrade strength.
  - Advanced six borings to about 11 feet below grade adjacent to select cores to characterize subsurface soils.
  - Completed two infiltration tests in borings along the alignment at a depth of 2.5 to 3 feet below the existing grade.
- Maintained a log of the materials encountered in the explorations and collected representative soil samples for laboratory testing;
- Conducted a program of laboratory testing on select soil samples including grain-size distribution, percent fines (percent passing the No. 200 sieve), moisture content, and Atterberg Limits determinations;
- Conducted engineering analyses to evaluate pavement design and rehabilitation alternatives and infiltration characteristics along the Project Corridor; and
- Prepared this geotechnical report summarizing the results of the subsurface exploration and laboratory testing programs and presenting our recommendations and conclusions.

3. Site Conditions

3.1 GEOLOGIC, SOILS, AND GROUNDWATER MAPPING

The geology of the Project Corridor is mapped as Willamette Silt in the study entitled *Preliminary geologic map of the McMinnville and Dayton Quadrangles* (Brownfield and Schlicker, 1981). These soils are described as consisting of poorly sorted semi-consolidated deposits of silt, clay, and sand with varying amounts of fine gravel. Our subsurface investigation suggests that these soils are present along the Project Corridor .

The U.S. Department of Agriculture (USDA) has mapped one near-surface soil unit, *Woodburn Silt Loam 0 to 3 percent slopes*, in the immediate vicinity of the Project Corridor in the Soil Survey of Yamhill County Area (USDA, 2024). The Woodburn Silt Loam spans the entire Project Corridor and extends across much of downtown McMinnville. Sourced from silty glaciolacustrine deposits and formed via river terraces, the typical profile described for this soil unit consists of silt loam extending to 17 inches below ground surface (bgs), followed by silty clay loam to 32 inches bgs, then silt loam to 68 inches bgs, then transitioning to a stratified fine sandy loam to silt loam to depths of at least 92 inches bgs. The unit is described as moderately well-drained and a depth to water table of 25 to 32 inches is reported.

The USDA provides the index properties presented in Table 1 for the Woodburn Silt Loam soil unit.

Table 1. Summary of USDA Soil Data

Soil Unit	Clay (percent)	Silt (percent)	Sand (percent)	Liquid Limit, LL	Plasticity Index	Corrosivity to Steel	Saturated Hydraulic Conductivity (inches/hour)
Woodburn Silt Loam	23.5	58.2	19.5	34.1	10.8	High	0.2 – 2.0

3.2 SURFACE CONDITIONS

3.2.1 General

Between NE Adams Street and NE Irvine Street, the Project Corridor is defined as a two-lane road with curb and gutter, and features curbside parallel parking stalls and sidewalks along each side of the road. Between NE Irvine Street and NE Johnson Street, a center turn lane substitutes the curbside parking stalls. A rail crossing is present between the intersections with NE Galloway Street and NE Irvine Street. The roadway is relatively flat with grade changes of less than 2 percent. Drainage is currently facilitated by curbside catch basins. This downtown area of McMinnville is developed with commercial developments generally consisting of retail and restaurant businesses on either side of the road for the entire Project Corridor. The sides of the Project Corridor include at-grade landscape strips and planter boxes landscaped with trees, bushes, and flowers.

3.2.2 Pavement Condition

We conducted a pavement condition survey in accordance with the 2010 Oregon Department of Transportation (ODOT) GFP Pavement Condition Rating Manual (ODOT, 2010). The pavement conditions were judged to range from Very Good to Poor as defined below.

- **Very Good:** No noticeable cracking or fatigue, rut depths less than 1/4-inch.
- **Good:** Low- to medium-severity transverse cracking, low-severity alligator cracking.
- **Fair:** Medium-severity transverse cracking, low- to medium-severity alligator cracking, slight rutting under traffic path.
- **Moderate:** Medium- to high-severity transverse cracking, medium- to high-severity alligator cracking, moderate rutting under traffic path.
- **Poor:** High-severity alligator cracking, deep rutting under traffic path.

A summary of our observations is provided in Table 2.

Table 2. Third Street Pavement Condition Summary

Approximate Location	Pavement Condition
NE Adams St to NE Baker St	Fair – Moderate trench cut repairs, moderate longitudinal cracking, minor transverse and alligator cracking
NE Baker St to NE Cowls St	Moderate – Heavy trench cut repairs, moderate longitudinal cracking, moderate alligator cracking (especially in WB lane), some patched potholes
NE Cowls St to NE Davis St	Moderate – Heavy trench cut repairs, heavy longitudinal cracking, localized areas of alligator cracking (especially in WB lane)
NE Davis St to NE Evans St	Fair – Moderate longitudinal cracking, minor transverse cracking, initiation of alligator cracking in localized areas
NE Evans St to NE Ford St	Fair – Moderate longitudinal cracking, minor transverse cracking, initiation of alligator cracking in localized areas
NE Ford St to NE Galloway St	Fair – Moderate longitudinal and transverse cracking, minor trench cut repairs
NE Galloway St to NE Irvine St	Poor – Heavy trench cut repairs, moderate longitudinal cracking, heavy alligator cracking and pothole repairs near RR crossing, especially east of railroad crossing
NE Irvine St to NE Johnson St	Fair – Moderate trench cut repairs, moderate longitudinal cracking, minor transverse cracking
NE Johnson St/3rd Street Intersection	Moderate – Moderate longitudinal, transverse, and alligator cracking

Note:
WB = Westbound

3.3 SUBSURFACE CONDITIONS

3.3.1 General

We explored subsurface soil and groundwater conditions along the Project Corridor by advancing solid stem auger borings and DCP probes, drilling pavement cores, and performing in situ infiltration tests. The locations of the explorations are shown on Figures 2 through 10. Soil conditions interpreted from geologic maps and our explorations, in conjunction with soil properties inferred from field observations and laboratory tests, formed the basis for the conclusions and recommendations contained within this report. Appendix A describes our field exploration procedures and presents field data and boring logs. Appendix B describes our laboratory soil testing procedures and results. Appendix C presents photographs of the pavement cores and DCP probe data and correlations.

3.3.2 Pavement

Pavement and soil conditions along NE 3rd Street were evaluated by drilling 15 pavement cores (designated A-1 through A-15) distributed between parking and drive lanes within the Project Corridor limits. DCP testing was performed at 10 of these core locations to estimate soil strength and California Bearing Ratio (CBR) values. Solid stem auger soil borings were drilled at six core locations to depths of up to 11.5 feet bgs to develop a better understanding of subgrade conditions at depth.

The asphalt concrete (AC) pavement sections encountered within the cores generally ranged from 2.5 to 8 inches thick, though sections are typically 3 to 6 inches thick (a section thickness of 11 inches was encountered at A-15, as discussed later). The AC appeared to be layered, with deeper layers showing distinct differences in aggregate composition, suggesting that the layers were placed at different points in time. The entire Project Corridor between NE Adams Street and NE Johnson Street is underlain by older Portland cement concrete (PCC). Core A-15, which was performed in the Third Street westbound left turn lane east of the intersection with NE Johnson Street, did not appear to encounter an underlying layer of PCC pavement beneath the AC pavement, which may explain why a thicker AC section was identified at this location. The A-15 location may coincide with a former trench cut that was repaired with full-depth asphalt.

At all cores except A-15, the AC section was underlain by PCC pavement ranging in thickness from 3.5 to 5 inches (average thickness of about 4 inches). Table 3 summarizes the layering of the pavement cores.

Table 3. Pavement Cores Data

Core Location	Lane	AC Thickness (inches)	Number of AC Layers Present	PCC Thickness (inches)
A-1	Parking Lane	4.5	3	4.5
A-2	Parking Lane	6	3	3.5
A-3	Parking Lane	3	2	3.5
A-4	Parking Lane	4	4	4
A-5	Parking Lane	4.5	2	4
A-6	Drive Lane	5	3	3.5
A-7	Parking Lane	3	3	3.5
A-8	Parking Lane	5.5	4	3.5
A-9	Parking Lane	4	3	4
A-10	Drive Lane	5	3	3.5
A-11	Drive Lane	2.5	2	4.75
A-12	Drive Lane	5	3	5
A-13	Drive Lane	6.5	2	4
A-14	Drive Lane	8	5	4
A-15	Drive Lane	11	4	None

3.3.3 Soils

Below the pavement sections, the explorations typically encountered a 3-inch-thick layer of a rounded gravel fill serving as a base aggregate. The soil subgrade beneath the base aggregate typically consisted of a 1- to 3-foot-thick layer of dark brown lean clay, which based on the presence of some minor brick fragments was judged to be fill, though is also possibly re-worked native soils. Moisture contents within the clay layer ranged from 19 to 34 percent. One Atterberg Limits test performed on the clay revealed a plasticity index of 12. The clay was typically underlain by soft to medium stiff silt and elastic silt with variable amounts of sand extending to depths of up to 11.5 feet bgs. The silty soils had laboratory-measured moisture contents between 19 and 41 percent. Three Atterberg Limits tests performed on the silts revealed plasticity indexes ranging from 2 to 20. Fines contents of the silts and clays ranged from 88 to 97 percent.

Two utility trenches were encountered during our exploration activities, near explorations A-8 and A-10. Trench backfill in these locations was characterized by 3- to 4-inch sub-rounded cobbles mixed with dark brown lean clay. The trench near boring A-10 included a 1-inch diameter steel pipe running perpendicular to NE 3rd Street.

3.3.4 Groundwater

In general, we did not observe groundwater in our shallow explorations; however, boring A-11 encountered water at approximately 9 feet bgs. Furthermore, we note that the results of our laboratory moisture content testing did not imply presence of groundwater in our explorations.

Review of water well records filed with the Oregon Water Resources Department indicate that depth to groundwater in the Project Corridor vicinity is typically 3 to 10 feet bgs. The depth to groundwater beneath the Project Corridor likely fluctuates seasonally and may be present at shallower depths during the wet season, which is typically between November and May.

3.3.5 Infiltration Testing

We performed two in situ encased falling head infiltration tests within two of the boring locations at depths of approximately 3 feet bgs (boring A-3) and 2.5 feet bgs (boring A-10). The field-measured “drawdown” rates (i.e., the vertical drop in the water level with time) are shown in Table 4 below. The fines contents of samples collected from the test locations are also shown in Table 4. The pre-soak period of each test was limited to 1 hour to limit traffic lane closures.

Table 4. Infiltration Test Data

Infiltration Test No.	Approximate Test Depth (feet)	Field Drawdown Rate (inches/hour)	Fines Content (percent)
IT-1 (at boring A-3)	3	2.3	91.9
IT-2 (at boring A-10)	2.5	1.5	88.2

Refer to Section 7, Infiltration Systems, for a discussion of our findings and recommendations regarding the design of infiltration systems.

3.3.6 DCP Probes

A U.S. Army Corps of Engineers Dual Mass DCP was advanced in select pavement cores to help characterize soil stiffness and obtain an estimate of the in situ CBR and resilient modulus of the native soils up to 3 feet below surface grade. The DCP consists of a steel extension shaft assembly with a 60-degree hardened steel cone tip attached to one end, which is driven into the subgrade by means of a sliding dual mass (10.1 pound) hammer. The DCP was simply lifted out of the ground under manual force upon completion of testing. Testing was conducted in accordance with ASTM International (ASTM) D6951/D6951M-18 (2018). DCP testing indicated resilient moduli ranging from 4,000 to 6,000 pounds per square inch (psi) were typical throughout the Project Corridor, with some outlier readings both greater and lesser than these values appearing at some depths in some locations.

4. Conclusions

Based on our explorations, testing, and analyses, it is our opinion that the site is suitable for the proposed roadway improvements, provided the recommendations in this report are included in design and construction. We offer the following general summary of our conclusions.

- The AC pavements throughout the Project Corridor are typically in poor to fair condition. We recommend complete replacement of the AC pavement with a new AC or PCC section to achieve a 20-year or greater design life. Alternatively, pavement rehabilitation consisting of a complete removal and replacement of existing AC down to the PCC layer may be completed for a 15-year design life. Mill and overlay rehabilitation is not recommended due to the potential for reflective cracking through the PCC layer to the pavement surface.
- The subgrade soils that blanket the Project area beneath the existing pavements consist of medium stiff, moist, clays and silts.
    - These materials will be easily disturbed by construction equipment, causing rutting, pumping, and general deterioration of subgrades when trafficked. We recommend that the contractor limit trafficking on the subgrade and/or employ wet weather/soil practices during all seasons.
    - Due to the fine-grained nature of the subgrade soils, it will not be reasonably possible to compact new aggregate base materials on top of the native subgrade without employing stabilization techniques, such as thickened rock sections or cement amendment.
    - The subgrade soils will require significant aeration/drying to use as structural fill. For planning purposes, they should be considered unsuitable for reuse as structural fill.
- While extensive groundwater is not expected to be encountered in shallow subgrade excavations, deeper utility trenches may encounter groundwater. Additionally, if construction occurs during wet weather the water is likely to pond in low areas and excavations and shallow seepage may occur. The contractor shall be prepared to control surface runoff and seepage into excavations, particularly during wet weather.
- Subgrade and shallow soils generally have low to moderate bearing capacity and moderate compressibility. Soft conditions may be encountered during pavement and utility subgrade preparation, and stabilization measures may be required.
- The hydraulic conductivity of the soils on site is low. As such, we do not recommend the use of infiltration systems unless further testing is completed that indicates that they are feasible.

The following sections present our recommendations for geotechnical aspects of the Project design. We have developed our conclusions and recommendations based on our current understanding of the Project. If the nature of the Project or location-specific Project elements are altered from those described in this report, Haley & Aldrich should be notified so we can confirm or modify our recommendations.

5. Pavement Design and Considerations

5.1 GENERAL

Paving for the Project includes new and/or rehabilitated AC and/or PCC pavements. Pavements should be constructed in accordance with City standards and Oregon Standard Specifications for Construction (OSSC) 00744 – Asphalt Concrete Pavement or OSSC 00756 – Plain Concrete Pavement, as applicable.

5.2 ROADWAY TRAFFIC

The following traffic loading criteria for the pavement design were based on guidelines found in the ODOT Pavement Design Guide (ODOT, 2019) and on traffic estimates provided by BKF.

- A 20- or 30-year design life for new and rebuilt AC pavement sections and a 30 or 50-year design life for new PCC pavement sections.
- A 15-year design life for rehabilitated AC pavement sections
- Average daily traffic (ADT) of 2,949 vehicles total, consisting of 1,232 vehicles in the eastbound direction and 1,717 vehicles in the westbound direction. The more conservative westbound ADT values were used for design. Vehicle types were broken down into the following FHWA classifications:
  - Types 1, 2, and 3: 83.5 percent of ADT Total
  - Types 4, 5, 6, and 7: 16 percent of ADT Total
  - Types 8, 9, and 10: 0.5 percent of ADT Total
- Assumed traffic growth rate of 2.5 percent per year.

Based on the data outlined above and equivalent single-axle load (ESAL) factors in ODOT (2019), we calculated the following design traffic loadings.

- New AC Pavements: 800,000 ESALs (20-year life); 1,500,000 ESALS (30-year life).
- New PCC Pavements: 1,600,000 ESALs (30-year life); 4,300,000 ESALs (50-year life).
- Rehabilitated Pavement Section: 530,000 ESALs (15-year life).

5.3 DESIGN PARAMETERS

The following pavement design parameters were based on guidelines found in ODOT (2019) and American Association of State Highway and Transportation Officials (AASHTO) Guide for Design of Pavement Structures (AASHTO, 1993).

- Design resilient modulus of 4,000 psi for in situ soil subgrade.
- A resilient modulus of 20,000 psi for base rock.
- Structural coefficients of 0.42 and 0.10 for new asphalt and base rock layers, respectively.
- Structural coefficient of 0.25 and 0.275 for existing AC and concrete pavement layers, respectively.

- Structural coefficient of 0.075 for a native soil subgrade that has been cement treated or for stabilization rock with a geotextile fabric.
- An effective modulus of subgrade reaction of 240 pounds per cubic inch for the design of new PCC pavement.

5.4 PAVEMENT SECTIONS

The following section describes options for new and/or rehabilitated pavements. Due to generally soft to medium stiff and moist subgrade conditions, where new pavement sections are installed, it should be assumed that subgrade stabilization work will be required. We have prepared recommendations for new AC pavement sections that include either 18 inches of stabilization rock or 10 inches of cement-amended soil subgrade. The recommended new AC pavement sections are for 20-year and 30-year design lives; recommendations for new PCC pavement sections use 30-year and 50-year design lives. Rehabilitated (e.g., crack-and-seat) pavements have been designed for a 15-year design life, in accordance with ODOT (2019).

The existing AC is underlain by PCC pavement. Due to joints, utility cuts, and cracks in the PCC, reflective cracking may transmit through new AC if the PCC is left in place in its current condition. Therefore, we recommend the use of the “crack-and-seat” method of breaking up the PCC to reduce the potential for reflective cracking. Refer to Section 5.5, Pavement Materials and Construction, for additional discussion regarding crack-and-seat. However, because the PCC bears directly on the native subgrade, we consider it to be somewhat risky to attempt cracking-and-seating, since there is a potential that the native subgrade may become disturbed during the process.

The existing pavement section along the Project Corridor is in fair to poor condition. The existing AC thickness varies from 2.5 to 11.0 inches and is typically underlain by PCC, except where demolished for excavation of utility trenches and in the small portion of the Project Corridor located east of NE Johnson Street. Where present, the PCC bears on a thin layer of rounded gravel underlain by several feet of lean clay fill.

We recommend that the entire pavement section either be rebuilt, or that new AC be placed atop the existing PCC after undergoing a crack-and-seat procedure. Table 4 summarizes pavement section options for the Project Corridor.

Table 4. Pavement Design Options

Classification	Design Life (years)	Pavement Thickness (inches)	Aggregate Base (inches)	Subgrade
New AC Pavement Section	20	5.5	5.0	18 inches stabilization rock over geotextile
	30	6.0	7.0	
	20	5.5	4.0	10 inches of cement amended soil
	30	6.0	5.5	
New PCC Pavement Section	30	8.0	6.0	In situ soil subgrade
	50	9.0	6.0	
New AC with Crack and Seat PCC	15	8.0	n/a	Crack-and-seat existing PCC

Note: *The ODOT Pavement Design Guide (PDG) recommends a minimum design life of 20 years for AC pavements in “rural” areas and 30 years in urban areas. The PDG recommends a typical design life of 50 years for PCC pavements, though a 30-year design life is permissible for “design of short segments with low traffic.”*

Parking lanes (not including bus stops or lanes expected to experience heavy truck traffic) may be designed with reduced pavement sections. Rigid PCC sections for parking lanes may consist of 6 inches of PCC over 6 inches of aggregate base. AC pavement sections in parking lanes may consist of 3.5 inches of AC over 3.5 inches of aggregate base.

5.5 PAVEMENT MATERIALS AND CONSTRUCTION

5.5.1 AC

The AC should conform to the specifications provided in OSS 00745 – Asphalt Concrete Pavement. The wearing and base course layers should be 1/2-inch Level 3 dense-graded hot mix asphalt with minimum and maximum lift thicknesses of 2 and 3 inches, respectively. The AC should be compacted to 92 percent of Rice Density of the mix, as determined in accordance with ASTM D 2041.

The AC binder should be PG 70-22 Performance Grade Asphalt Cement according to 00745.11 – Asphalt Cement and Additives.

5.5.2 Rigid PCC

Rigid PCC used for pavement should meet the specifications provided in OSS 00756 – Plain Concrete Pavement. The installed concrete should be Class 4000 1.5-inch paving concrete per OSS 02001 – Concrete. The PCC joints should have a maximum spacing of 12 feet and be constructed in accordance with OSS 00756.48 – Joints and ODOT standard details DET 1600 and 1602. Joints should not be located in wheel tracks.

PCC should be interlocked at contraction joints (e.g., continuous slab with no dowels), though dowels should be used at construction and expansion joints. Dowels should have a minimum diameter equal to 1/8-inch per 1 inch of PCC thickness.

Where the pavement is tied into the curb and gutters to reduce the PCC thickness, tie bars should be installed at the longitudinal joints using dowels equal in diameter to those used for construction and expansion joints.

5.5.3 Aggregate Base

Imported granular material used as aggregate base (base rock) beneath conventional AC pavement should meet the criteria specified in Section 6.2, Structural Fill and Backfill. A subgrade geotextile conforming to OSS 02320 (Table 4) shall be placed atop the soil subgrade before aggregate base is installed. However, a subgrade geotextile is not required atop a cement-treated subgrade or stabilization material that is underlain by a subgrade geotextile.

5.5.4 AC Grinding

Grinding of existing AC should be completed in conformance with OSSC 620 – Cold Plane Pavement Removal.

5.5.5 Cement-Treated Subgrade

If the in situ fine-grained subgrade soils are stabilized via cement treatment, then the soil amending should be conducted in accordance with the Section 6.2.7, Cement-Treated Soil. The treatment depth shall be a minimum of 10 inches and a target 28-day unconfined compressive strength of 100 psi for the treated soil shall be achieved.

5.5.6 Crack-and-Seat PCC

PCC underlies AC pavements throughout the Project Corridor. One pavement rehabilitation option is provided that allows the PCC to remain in place and an AC overlay to be installed. If this option is chosen, then the PCC will need to be prepared via the “crack-and-seat” methodology to help reduce the potential for future reflective cracking. Where existing PCC will be left in place, the panels shall be cut into pieces ranging from 1 to 3 feet in greatest dimension. Due to the soft, easily disturbed nature of the soil subgrade, “rubblization” should not be attempted. The panels shall only be broken down by saw cutting, as dynamic action, such as hydraulic hammers, will also disturb the subgrade. After the slabs have been saw cut, they should be “seated” by several passes of an approximately 25- to 35-ton pneumatic (rubber-tired) roller.

6. Earthwork Recommendations

Based on available information, we estimate mass grading for the site will be limited, with shallow areal excavations being made to accommodate aggregate base sections for reconstructed or newly constructed roadways, curbs, and sidewalks. However, because of soft, moist soil conditions, proper planning and execution of earthwork will have significant implications for successful completion of the Project.

All earthwork should be conducted in accordance with City requirements and the OSSC (ODOT, 2024). Project-specific recommendations for earthwork are provided in the following sections.

6.1 SITE PREPARATION

6.1.1 General

The site soils are highly susceptible to moisture-related disturbance. Wet soil construction practices will be necessary throughout most of the year, particularly during periods of wet weather. Wet soil construction practices include limiting trafficking on exposed subgrades and using equipment, such as smooth excavator buckets and tracked equipment, to limit subgrade disturbance, etc. Due to widespread soft, wet conditions, the existing soil subgrade will not be suitable for placement of aggregate base or fill material without prior stabilization, as discussed later.

6.1.2 Demolition

Demolition should include complete removal of existing site improvements within areas to receive new pavements, curbs, or sidewalks. Underground utility lines, vaults, or tanks that are to be abandoned should be completely removed or grouted full if left in place.

Voids resulting from removal of below-grade structures or loose soil in utility lines should be backfilled with compacted structural fill, as discussed in Section 6.2, Structural Fill and Backfill. The bases of such excavations should be completed to a firm subgrade before filling, and their sides sloped slightly to allow for more uniform compaction at the edges of the excavations.

Materials generated during demolition of existing improvements should be transported off site for disposal or stockpiled in areas designated by the City. In general, these materials will not be suitable for reuse as engineered fill. However, asphalt, concrete, and base rock material may be crushed and recycled for use as general fill. Such recycled material should meet the specifications for imported granular material, as described in Section 6.2, Structural Fill and Backfill.

Refer to Section 5, Pavement Design and Considerations, for discussion regarding AC grinding and “crack-and-seat” preparation of existing PCC.

6.1.3 Stripping

We anticipate most of the improvements will not require stripping of the existing ground surface; however, some stripping will be required in localized areas throughout the Project. Actual stripping depths should be evaluated, based on observations during the stripping operation. The prepared subgrade should be observed and approved by the engineer. Generally, visible organic material (sod, roots larger than 1/4-inch diameter, and/or other plant material), debris, and other unsuitable materials should be removed from the subgrade areas. Such material will not be suitable for use as structural fill and should be hauled off site as designated by the City.

6.1.4 Subgrade Preparation

Wherever possible, the contractor should work from existing paved surfaces and limit trafficking onto exposed soil subgrades. As noted in Section 5, Pavement Design and Considerations, the new pavement designs assume the pavement subgrade will be augmented by the installation of an 18-inch-thick layer of stabilization rock or by 10 inches of cement amended subgrade. These measures are to provide a stable subgrade for compaction of aggregate base and AC. The contractor is responsible for providing additional stabilization material, if needed, to protect the subgrade from disturbance caused by construction traffic. The contractor should not attempt to compact the native soil subgrade prior to implementing one of the stabilization measures discussed above.

Following subgrade excavation and prior to implementing stabilization measures, the suitability of the subgrade should be evaluated by Haley & Aldrich. It is unlikely that the typical proof-rolling with a fully loaded dump truck or similar heavy rubber-tired construction equipment will be suitable due to soft conditions, though this method should be utilized if deemed feasible by Haley & Aldrich. Otherwise, the evaluation will likely include use of a steel T-probe to identify any excessively soft areas. If excessively soft zones are identified during the evaluation, then additional subgrade excavation may be required.

6.1.5 Utility Considerations

Underground utilities should be installed prior to installation of geotextile and stabilization rock or cement amendment of the subgrade; otherwise, utility construction will disturb these stabilization measures. Additionally, care should be taken in the selection of the subgrade stabilization measure to assure that existing utilities are not disturbed. For planning purposes, a minimum of 1 foot of clearance between the base of a cement-treated soil (currently proposed to be 10 inches thick) and any buried utilities should be maintained. Individual contractors may require greater clearance or release of liability from potential damage to buried utilities.

6.2 STRUCTURAL FILL AND BACKFILL

Structural fill includes fill for embankments; for slab and pavement support, such as aggregate base; and other fill within the influence zone of structures adjacent to the improvement area. Fill should only be placed over a subgrade that has been prepared in accordance with Section 6.1, Site Preparation. A variety of soils may be used as structural fill, provided they are free of debris, clay balls, roots, organic matter, frozen soil, man-made contaminants, particles exceeding 4 inches in size, and other deleterious material. Structural fill should meet the appropriate specifications provided in OSSC 00330.12 – Borrow Material, 00330.13 – Selected General Backfill, 00330.14 – Selected Granular Backfill or others as appropriate.

Fill and backfill material should be placed and compacted in lifts with maximum uncompacted thicknesses and relative densities as recommended in the OSSC and the tables that follow.

6.2.1 On-Site Soils

In general, the native materials in the Project Corridor consist of moist, fine-grained soils and will not be suitable for reuse as structural fill. However, they may be reused when they are directly incorporated into a cement-amended subgrade.

6.2.2 Recycled AC, PC, and Aggregate Base

Existing AC, PCC, and aggregate base from the site can be used in general structural fill, provided these materials are thoroughly and uniformly crushed with no particles greater than 3 inches. If used as trench backfill, this material should not be used within the pipe zone. The recycled materials should meet the specifications provided in OSSC 00330.13 – Selected General Backfill.

6.2.3 Aggregate Base

Imported granular material used as aggregate base beneath pavements or slabs should be clean, crushed rock or crushed gravel and sand that is fairly well graded between coarse and fine. The base aggregate should meet the specifications provided in OSSC 02630.10 – Dense Graded Base Aggregate, depending upon application. For use beneath sidewalks, we generally recommend the rock have a maximum particle size of 0.75 or 1 inch.

Aggregate base should be separated from the base of untreated fine-grained subgrades with a layer of subgrade geotextile that meets the specifications provided in OSSC 02320.20 – Geotextile Property Values for subgrade geotextile (separation). The geotextile should be installed in conformance with the specifications provided in OSSC 00350 – Geosynthetic Installation.

6.2.4 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 12 inches above utility lines (i.e., the pipe zone) should meet City and ODOT specifications and consist of well-graded granular material with a maximum particle size of 3/4 inch and less than 10 percent by dry weight passing the U.S. Standard No. 200 Sieve, and should meet the specifications provided in OSSC 00405.13 – Pipe Zone Material. Within roadway alignments, the remainder of the trench backfill up to the subgrade elevation should consist of well-graded granular material with a maximum particle size of 3 inches, have less than 10 percent by dry weight passing the U.S. Standard No. 200 Sieve, and meet the specifications provided in OSSC 00405.14 – Trench Backfill, Class B or D.

Outside of structural improvement areas, trench backfill placed above the pipe zone may consist of general fill materials that are free of organics and material over 6 inches in diameter and meet the specifications provided in OSSC 00330.12 – Borrow Material.

6.2.5 Imported Select Structural Fill

Imported granular material used as structural fill during periods of wet weather should be pit or quarry run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in OSSC 00330.14 – Selected Granular Backfill, 00330.15 – Selected Stone Backfill, or 00330.16 – Selected Stone Embankment. The imported granular material should also be angular, fairly well-graded between coarse and fine material, have less than 5 percent by dry weight passing the U.S. Standard No. 200 Sieve, and have at least two mechanically fractured faces.

6.2.6 Stabilization Material

If imported granular material is used to stabilize subgrade excavations, we recommend that material consist of pit or quarry run rock, or crushed rock. The material should generally be sized between 2 and 6 inches, have less than 5 percent by dry weight passing the U.S. Standard No. 4 Sieve, and have at least two mechanically fractured faces. The material should be free of organic matter and other deleterious material. Material meeting the specifications of OSSC 00330.16 – Stone Embankment, though with the maximum size noted above is generally acceptable, as are smaller quarry spalls, ballast, and other similar clean angular materials.

Stabilization material should be separated from the base of soft or fine-grained subgrades with a layer of subgrade geotextile that meets the specifications provided in OSSC 02320.20 – Geotextile Property Values for subgrade geotextile (separation). The geotextile should be installed in conformance with the specifications provided in OSSC 00350 – Geosynthetic Installation.

Stabilization material should be placed atop the geotextile in an initial 12-inch loose lift. The rock should be compacted with a 2- to 3-ton, smooth, dual-drummed roller operating in “static” mode to a well “keyed” condition. The remaining 6 inches of rock should be placed and compacted to 95 percent of the rock’s maximum dry density as determined by ASTM D 1557/AASHTO T-180.

6.2.7 Cement-Treated Soil

As an alternative to the use of stabilization material for subgrade protection, an experienced contractor may be able to amend the on-site soils with Portland cement to obtain suitable support properties. Successful use of soil amendment depends on the use of correct mixing techniques, soil moisture content, and amendment quantities. Specific recommendations for soil amendment, based on exposed site conditions, can be provided if necessary. Soil amendment should be conducted in accordance with the specifications provided in OSSC 00344 – Treated Subgrade.

For budgeting purposes, we recommend 6 percent cement (by dry weight) be used for soil treatment. A dry weight of 110 pounds per cubic foot should be assumed for the soil. Actual percentages of cement will need to be based on in situ soil moisture contents and other field conditions at the time of amendment.

Portland cement-amended soils are hard and have low permeability. These soils do not drain well, nor are they suitable for planting. Future planted areas should not be cement amended, if practical, or accommodations should be made for drainage and planting. Moreover, cement amending of soil within building areas must be done carefully to avoid trapping water under floor slabs. We should be contacted if this approach is considered. Cement amendment should not be used if runoff during construction cannot be directed away from adjacent wetlands.

To protect the cement-treated surfaces from abrasion, “slickening,” or other damage, the treated surface should be covered with 4 to 6 inches of imported granular material before it is tracked by equipment. The crushed rock can become contaminated with soil during construction if not properly protected. If the rock becomes contaminated, it should be removed and replaced with clean rock prior to paving.

It is not possible to amend soils during heavy or continuous rainfall. Work should be completed during suitable conditions. To prevent strength loss during curing, cement-amended soil should be allowed to cure for a minimum of four days prior to access by construction traffic.

6.3 FILL PLACEMENT AND COMPACTION

Structural fill should be placed and compacted in accordance with OSSC 00330.43 – Earthwork Compaction requirements and the following guidelines.

- Place fill and backfill on a prepared subgrade that consists of firm, inorganic native soils or approved structural fill.
- Place fill or backfill in uniform horizontal lifts with a thickness appropriate for the material type and compaction equipment. Table 5, below, provides general guidance for uncompacted lift thicknesses.
- Do not place fill and backfill until the required tests and evaluation of the underlying materials have been made and the appropriate approvals have been obtained.
- Limit the maximum particle size within the fill to two-thirds of the loose lift thickness.
- Control the moisture content of the fill to within 3 percent of the optimum moisture content based on laboratory Proctor tests. The optimum moisture content corresponds to the moisture content at the maximum attainable Proctor dry density.
- Perform a representative number of in-place density tests on structural fill in the field to verify adequate compaction.

Table 5. Guidelines for Uncompacted Lift Thickness

Compaction Equipment	Guidelines for Uncompacted Lift Thickness (inches)	
	Granular and Crushed Rock Maximum Particle Size less than or equal to 1½ inch	Crushed Rock Maximum Particle Size greater than 1½ inch
Plate Compactors and Jumping Jacks	4 to 8	Not Recommended
Rubber-Tire Equipment	8 to 12	6 to 8
Light Roller	8 to 12	8 to 10
Heavy Roller	12 to 18	12 to 16
Hoe Pack Equipment	18 to 24	12 to 16

Note:

The above table is based on our experience and is intended to serve as a guideline. The information provided in this table should not be included in the Project specifications.

During structural fill placement and compaction, a sufficient number of in-place density tests should be completed by Haley & Aldrich to verify that the specified degree of compaction is being achieved. For structural fill with more than 30 percent retained on the 3/4-inch sieve, proper compaction should be verified with a proof roll or other performance methods.

6.4 EXCAVATION

6.4.1 General

Site soils within expected excavation depths generally consist of moist clay and silt. In our opinion, conventional earthmoving equipment in proper working condition should be capable of making necessary general excavations for utilities and other earthwork. The earthwork contractor should be responsible for providing equipment and following procedures as needed to excavate the site soils, as described in this report, while protecting the subgrade.

6.4.2 Temporary Cut Stability

Because of the variables involved, actual slope angles required for stability in temporary cut areas can only be estimated before construction. We recommend that stability of the temporary slopes used for construction be the responsibility of the contractor, since the contractor is in control of the construction operation and is continuously at the site to observe the nature and condition of the subsurface.

All temporary soil cuts associated with site excavations (greater than 4 feet in depth) should be adequately sloped back to prevent sloughing and collapse, in accordance with Occupational Safety and Health Administration (OSHA) guidelines. The stability and safety of cut slopes depend on a number of factors, including:

- The type and density of the soil;
- The presence and amount of any seepage;
- Depth of cut;
- Proximity and magnitude of the cut to any surcharge loads, such as stockpiled material, traffic loads, or structures;
- Duration of the open excavation; and
- Care and methods used by the contractor.

All excavations should be made in accordance with all local, state, and federal safety requirements. According to OSHA guidelines, we expect that the existing site soils would be considered Type C.

Where groundwater seepage is encountered within excavation slopes, the cut slope inclination may have to be flatter than 1.5 horizontal to 1 vertical. However, appropriate inclinations will ultimately depend on the actual soil and groundwater seepage conditions exposed in the cuts at the time of construction. It is the responsibility of the contractor to ensure that the excavation is properly sloped or braced for worker protection, in accordance with OSHA guidelines. To assist with this effort, for planning purposes only, we make the following recommendations regarding temporary excavations.

- Protect excavations from erosion with plastic sheeting for the duration of the excavation to minimize surface erosion and raveling.
- Limit the maximum duration of the open excavation to the shortest time period possible.
- Place no surcharge loads (equipment, materials, etc.) within 10 feet of the top of excavations.

More restrictive requirements may apply depending on specific site conditions, which should be continuously assessed by the contractor.

If temporary sloping is not feasible based on site spatial constraints, excavations could be supported by internally braced shoring systems, such as a trench box or other temporary shoring. There are a variety of options available. We recommend that the contractor be responsible for selecting the type of shoring system to apply.

6.5 DEWATERING AND TEMPORARY DRAINAGE

Groundwater may be present within the depths of utility excavations; therefore, trenching operations may require dewatering. Refer to Section 3.3.4, Groundwater, for a discussion of groundwater conditions at the site. Dewatering is typically the responsibility of the contractor. Due to the fine-grained nature of the site soils, pumping from sumps located within the excavation will likely be effective in removing water resulting from seepage. Deeper trenching and excavation work may require well point dewatering, though this is not anticipated. Failure to dewater can result in issues, such as base heave, sidewall caving and sloughing, increased backfill and haul off requirements, and Project delays.

During grading at the site, the contractor should be made responsible for temporary drainage of surface water as necessary to prevent standing water and/or erosion of the working surface. During rough and finished grading of the roadway alignment, the contractor should keep subgrades free of water.

7. Infiltration Systems

The results of on-site field infiltration testing are described in Section 3.3.5, Infiltration Testing. In general, we found that the soils are not suitable for infiltration with unfactored hydraulic conductivity values between 1.5 and 2.3 inches per hour. Further, mapping from the USDA Web Soil Survey indicates that the hydraulic conductivity of on-site soils is typically between 0.2 and 2 inches per hour. Based on the high fines content of the soils (89 to 97 percent), we consider the field testing to be unrepresentative of the actual, long-term infiltration capacity of the site soils. Longer-term in situ field testing may be required to better characterize the actual infiltration capacity. We would anticipate long-term rates to be on the order of 0.2 inches per hour or less. As such, we recommend against the use of stormwater infiltration facilities. If further consideration for such systems is desired, then additional field testing will be necessary.

8. Design Support and Construction Observations

The final Project plans and specifications, particularly those including pavement and subsurface improvements, should be reviewed by Haley & Aldrich prior to construction to check that they are in general conformance with the intent of our recommendations.

Satisfactory pavement and earthwork performance depends to a large degree on quality of construction. Sufficient monitoring 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. Recognition of changed conditions often requires experience; therefore, Haley & Aldrich or their representative should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

We recommend that Haley & Aldrich be retained to monitor construction at the site to confirm that subsurface conditions are consistent with the site explorations and to confirm that the intent of Project plans and specifications relating to earthwork and paving are being met. In particular, we recommend that stripping and subgrade preparation/stabilization, as well as placement and compaction of structural backfill, aggregate base, and asphalt pavement be observed and/or tested by Haley & Aldrich.

9. Limitations

We have prepared this report for the exclusive use of BKF, the City, and their authorized agents for the proposed Third Street Improvements Project in McMinnville, Oregon in accordance with our subconsultant agreement. Our report is intended to provide our opinion of geotechnical parameters for design and construction of the proposed Project based on exploration locations that are believed to be representative of site conditions. However, conditions can vary significantly between exploration locations and our conclusions should not be construed as a warranty or guarantee of subsurface conditions or future site performance.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. No warranty, express or implied, should be understood.

Any electronic form, facsimile, or hard copy of the original document (email, text, table, and/or figure), if provided, and any attachments are only a copy of the original document. The original document is stored by Haley & Aldrich and will serve as the official document of record.

References

1. American Association of State Highway and Transportation Officials , 1993. Guide for Design of Pavement Structures.

2. Brownfield, M.E., and Schlicker, H.G., 1981. Preliminary geologic map of the McMinnville and Dayton quadrangles: Oregon Department of Geology and Mineral Industries, Open-File Report 81-6, scale 1:24,000.

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4. ODOT, 2019. ODOT Pavement Design Guide.

5. ODOT, 2010. GFP Pavement Condition Rating Manual, ODOT Pavement Management Unit, updated July.

6. United States Department of Agriculture, 2024. Soil Survey Staff, Natural Resources Conservation Service, United States Department of Agriculture Web Soil Survey. Available online at the following link: http://websoilsurvey.sc.egov.usda.gov/. Accessed 07/9/2024.

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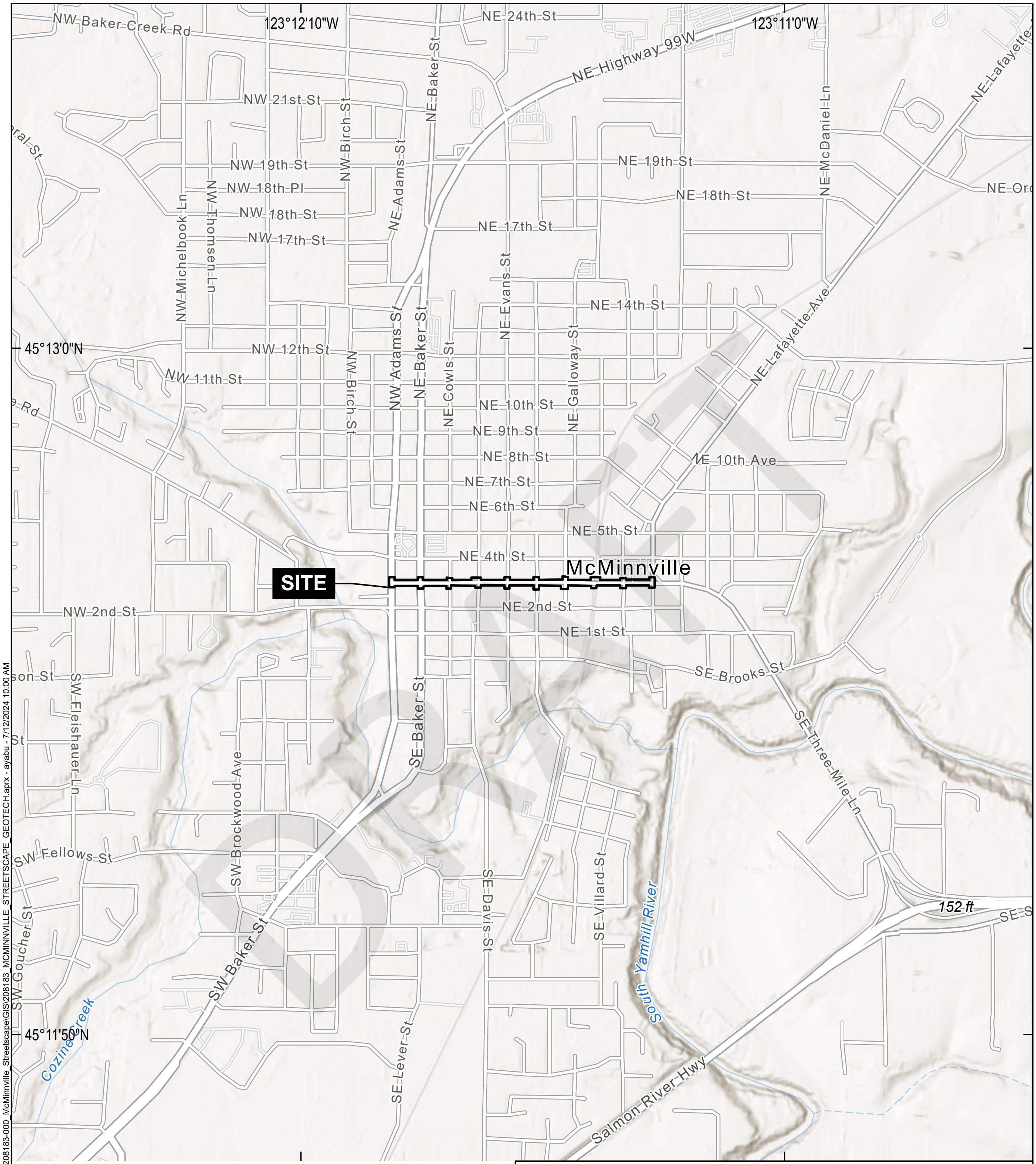
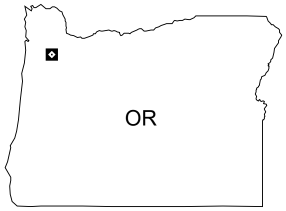
FIGURES





MAP SOURCE: ESRI
SITE COORDINATES: 45°12'36"N, 123°11'37"W

HALEY ALDRICH

THIRD STREET IMPROVEMENTS
MCMINNVILLE, OREGON

PROJECT LOCUS

APPROXIMATE SCALE: 1 IN = 2000 FT
JULY 2024

FIGURE 1

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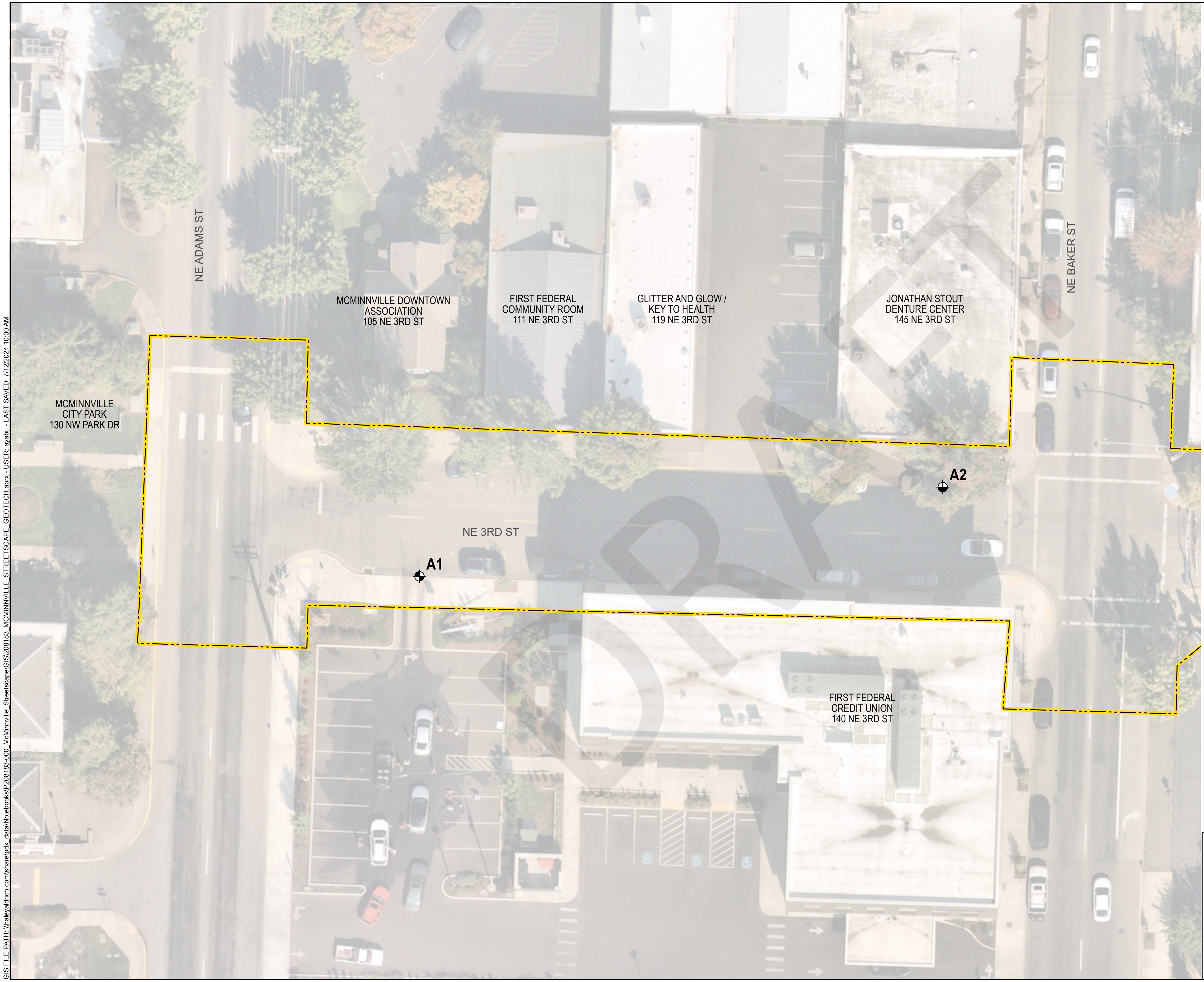

LEGEND

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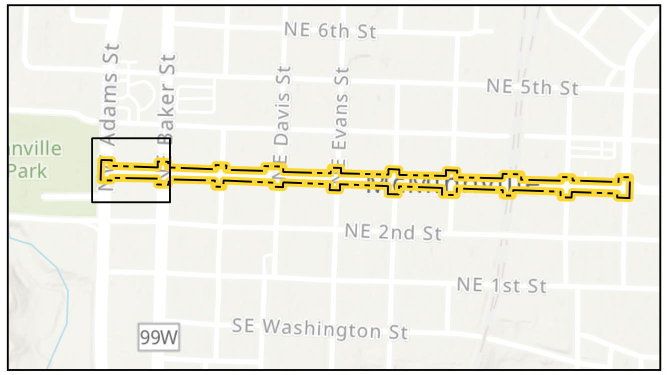
- ⌖ DCP AND SOLID STEM AUGER BORING / INFILTRATION TEST
- ⌖ PAVEMENT CORE
- ⌖ PAVEMENT CORE / INFILTRATION TEST
- ▭ SITE BOUNDARY

NOTES

1. ALL LOCATIONS AND DIMENSIONS ARE APPROXIMATE.
2. AERIAL IMAGERY SOURCE: NEARMAP, 16 SEPTEMBER 2023



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HALEY ALDRICH

THIRD STREET IMPROVEMENTS
MCMINNVILLE, OREGON

SITE AND EXPLORATION PLAN
NE ADAMS ST TO NE BAKER ST

JULY 2024

FIGURE 2

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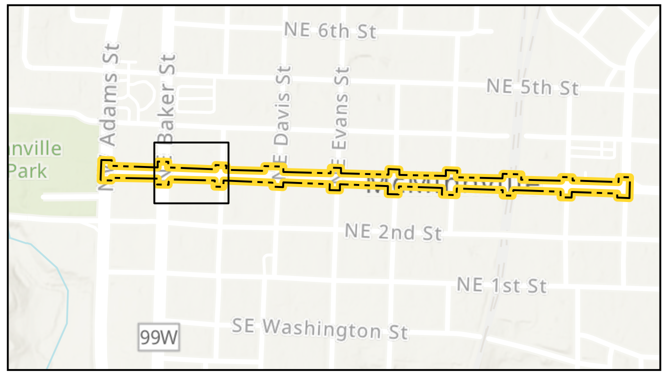

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THIRD STREET IMPROVEMENTS
MCMINNVILLE, OREGON

SITE AND EXPLORATION PLAN
NE BAKER ST TO NE COWLS ST

JULY 2024

FIGURE 3

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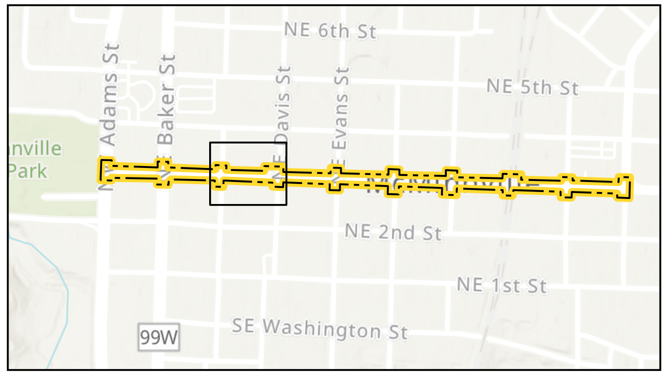

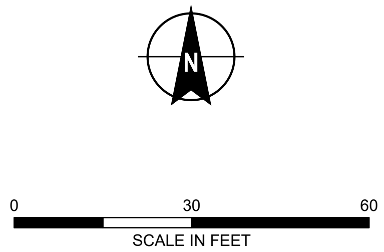

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THIRD STREET IMPROVEMENTS
MCMINNVILLE, OREGON

SITE AND EXPLORATION PLAN
NE COWL ST TO NE DAVIS ST

JULY 2024

FIGURE 4

LEGEND

⊕ DCP AND SOLID STEM AUGER BORING

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⊕ PAVEMENT CORE

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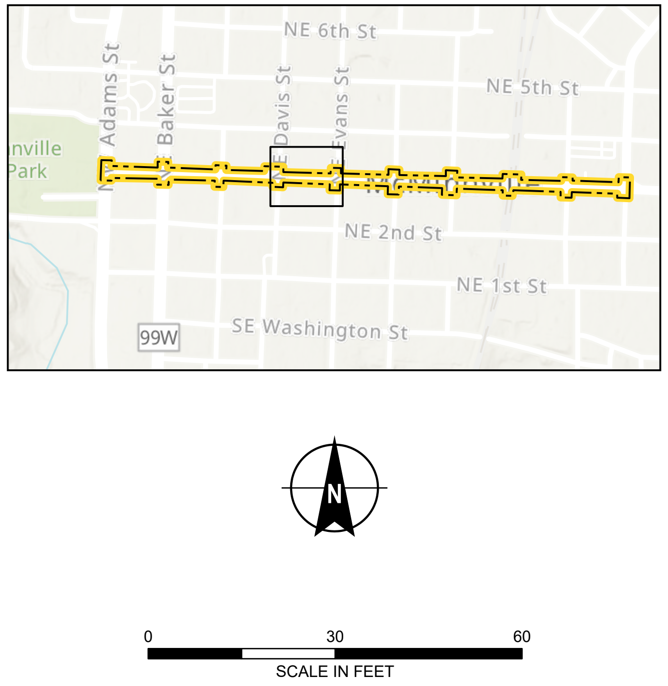

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THIRD STREET IMPROVEMENTS
MCMINNVILLE, OREGON

SITE AND EXPLORATION PLAN
NE DAVIS ST TO NE EVANS ST

JULY 2024

FIGURE 5

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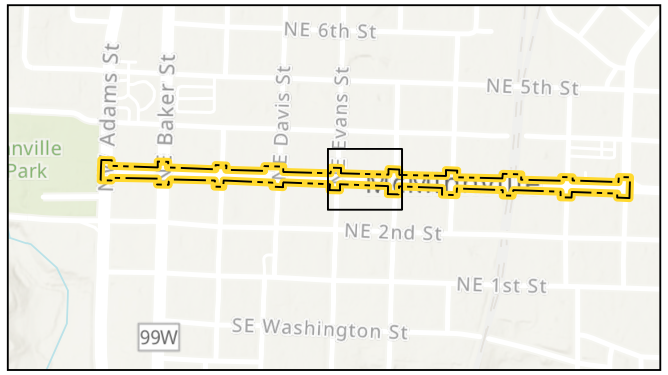

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THIRD STREET IMPROVEMENTS
MCMINNVILLE, OREGON

SITE AND EXPLORATION PLAN
NE EVANS ST TO NE FORD ST

JULY 2024

FIGURE 6

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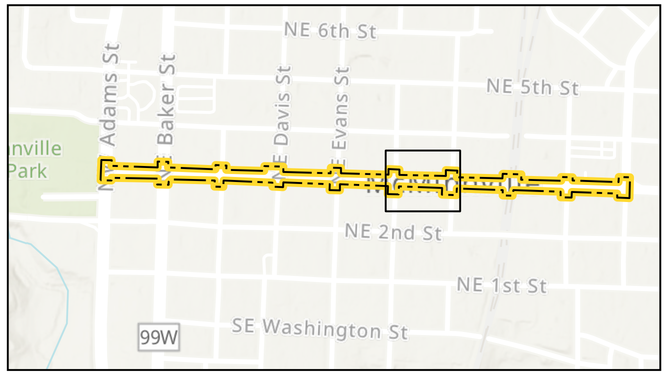

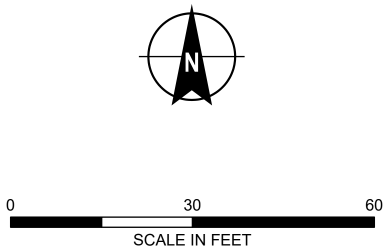

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THIRD STREET IMPROVEMENTS
MCMINNVILLE, OREGON

SITE AND EXPLORATION PLAN
NE FORD ST TO NE GALLOWAY ST

JULY 2024

FIGURE 7

LEGEND

- ⊕ DCP AND SOLID STEM AUGER BORING
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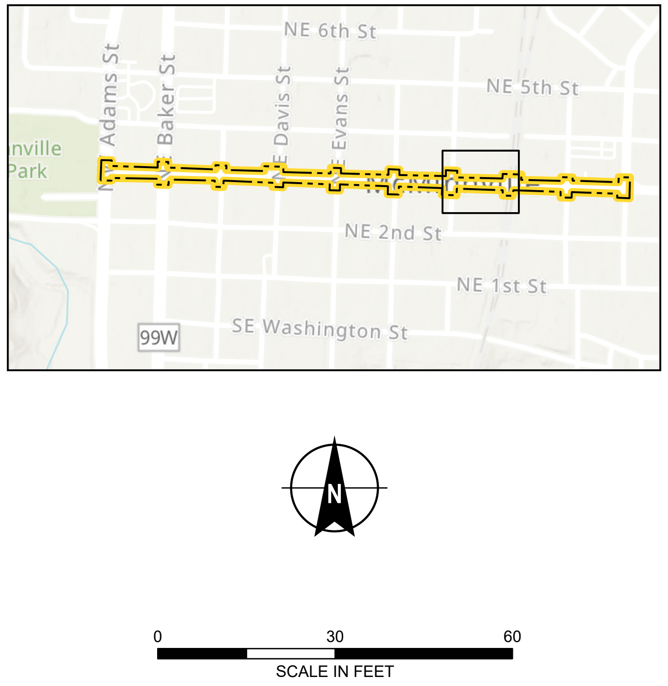

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MCMINNVILLE, OREGON

SITE AND EXPLORATION PLAN
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FIGURE 8

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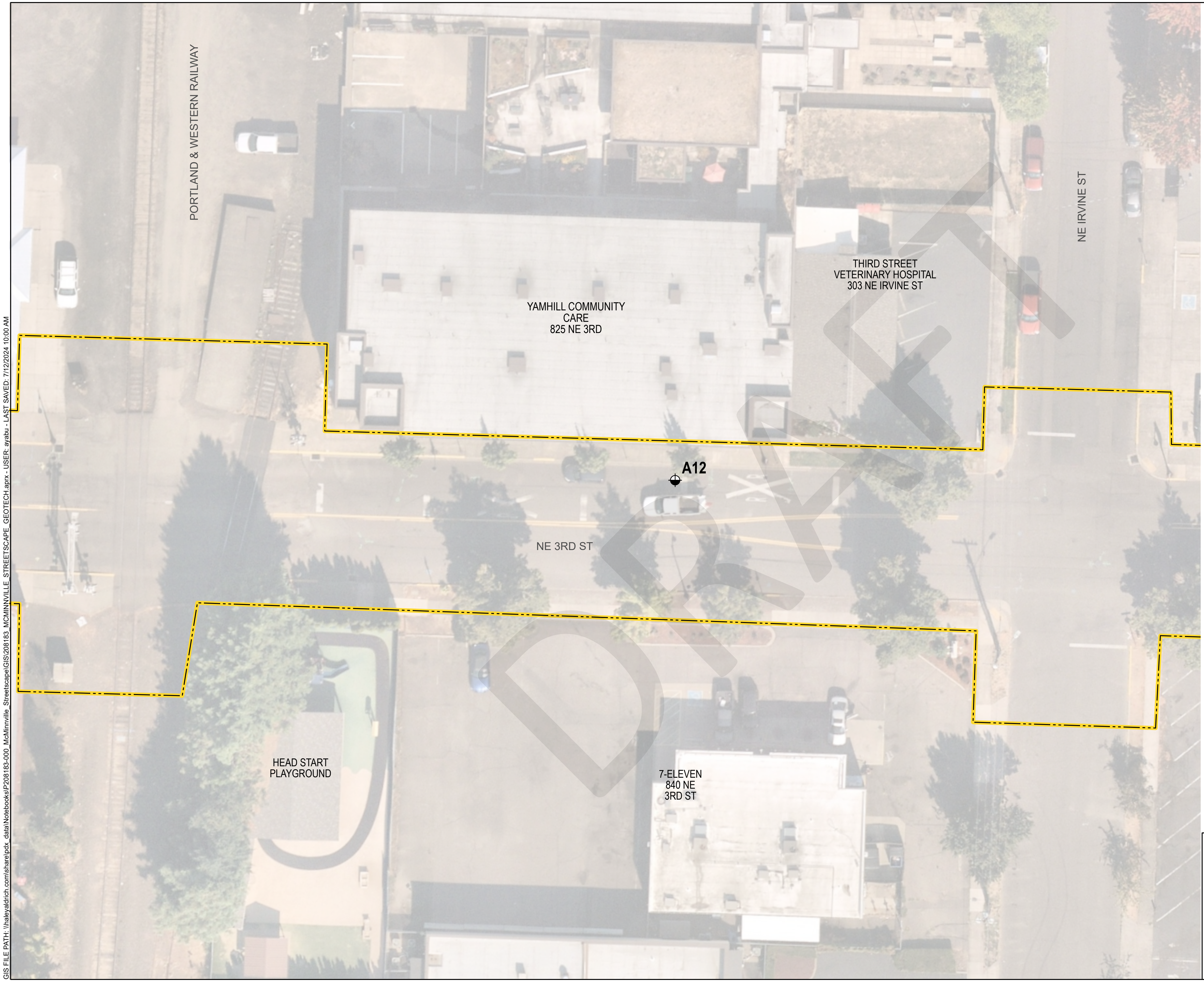

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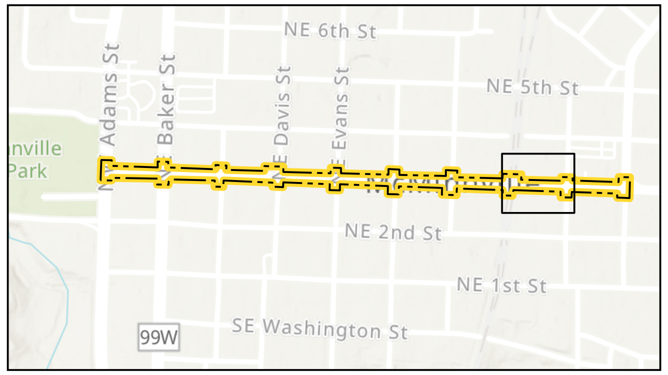

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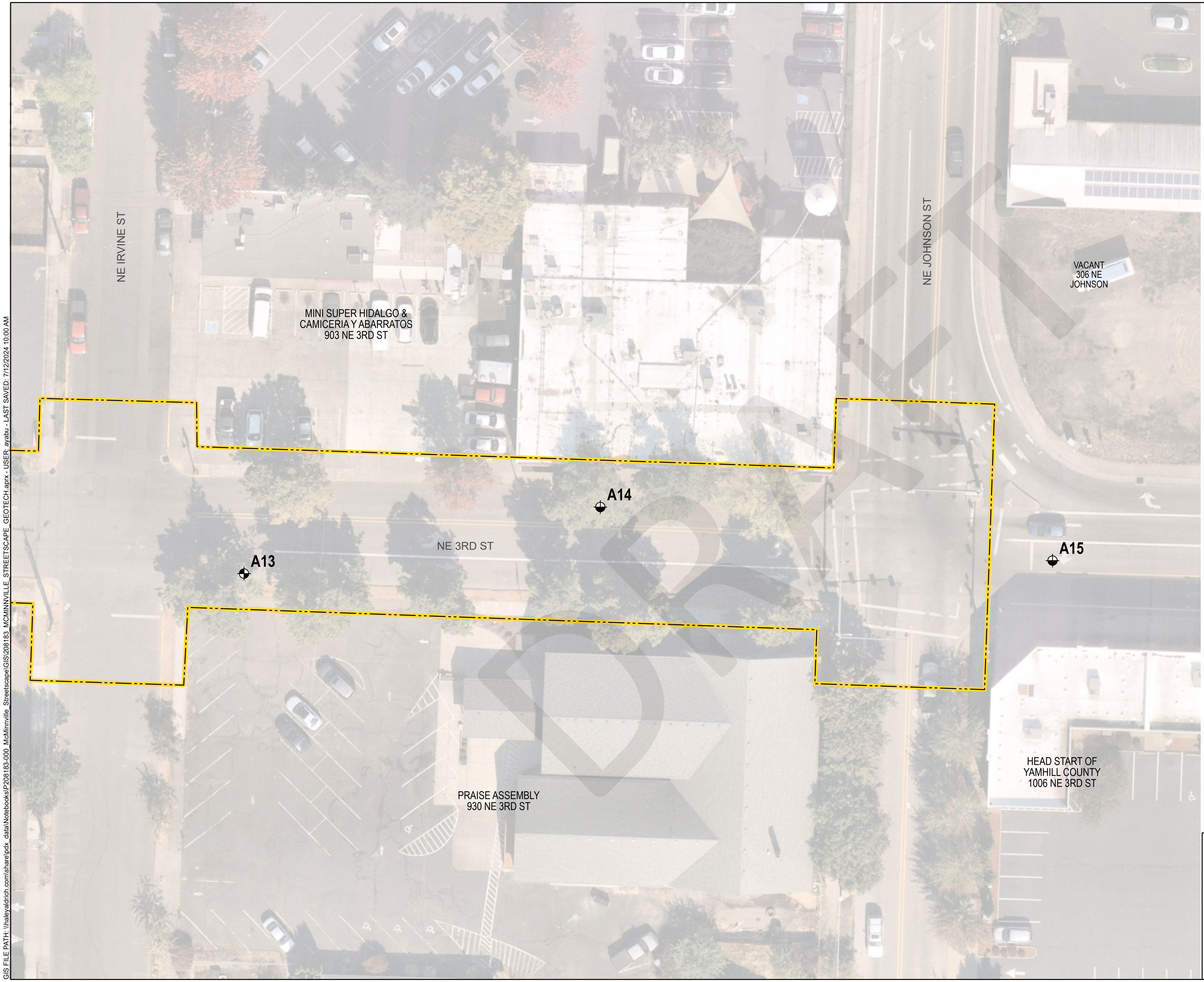
THIRD STREET IMPROVEMENTS
MCMINNVILLE, OREGON

SITE AND EXPLORATION PLAN
RAILROAD TO NE IRVINE ST

JULY 2024

FIGURE 9

GIS FILE PATH: \\haleyaldrich.com\share\pdx_data\Notebooks\P208183-000_McMinnville_Streetscape\GIS\208183_MCMINNVILLE_STREETSCAPE_GEOTECH.aprx - USER: ayabu - LAST SAVED: 7/12/2024 10:00 AM



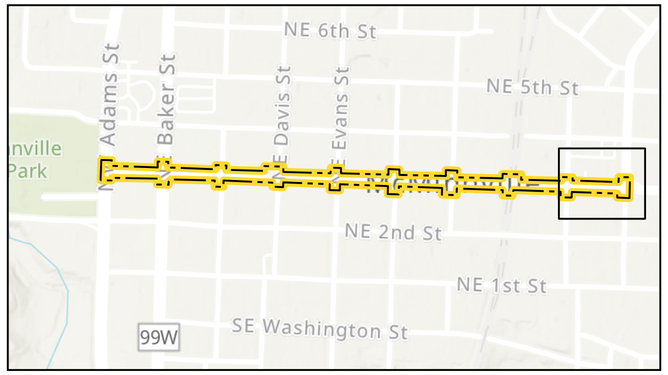
GIS FILE PATH: \\haleyaldrich.com\share\pdx_data\Notebooks\P208183-000_McMinnville_Streetscape\GIS\208183_MCMINNVILLE_STREETSCAPE_GEOTECH.aprx - USER: ayabu - LAST SAVED: 7/12/2024 10:00 AM

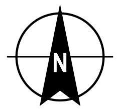
LEGEND

⊕	DCP AND SOLID STEM AUGER BORING
⊕	DCP AND SOLID STEM AUGER BORING / INFILTRATION TEST
⊕	PAVEMENT CORE
⊕	PAVEMENT CORE / INFILTRATION TEST
▭	SITE BOUNDARY

NOTES

1. ALL LOCATIONS AND DIMENSIONS ARE APPROXIMATE.
2. AERIAL IMAGERY SOURCE: NEARMAP, 16 SEPTEMBER 2023





0 30 60
SCALE IN FEET

HALEY ALDRICH

THIRD STREET IMPROVEMENTS
MCMINNVILLE, OREGON

SITE AND EXPLORATION PLAN
NE IRVINE ST TO NE JOHNSON ST

JULY 2024

FIGURE 10

APPENDIX A

Exploration Logs

APPENDIX A

Field Explorations

This appendix documents the processes Haley & Aldrich, Inc. (Haley & Aldrich) used to determine the nature and quality of the soil and groundwater underlying the project site addressed by this report.

Explorations and Their Location

We investigated subsurface conditions at the site by completing six borings, 10 Dynamic Cone Penetrometer (DCP) probes, 15 pavement cores, and two infiltration tests. The borings were advanced to depths of 11.5ft bgs. The borings were advanced using a solid-stem auger on a trailer-mounted Big Beaver drill rig operated by Dan Fischer Excavating of Forest Grove, Oregon under subcontract to Haley & Aldrich. The locations of the explorations are shown on Figures 2 through 10.

The field explorations were coordinated by one of our geotechnical engineering staff members, who classified the various soil units encountered, obtained representative soil samples for geotechnical testing, and maintained a detailed log of the borings. The exploration logs are included in this appendix. Results of the laboratory testing are indicated on the exploration logs and are included in Appendix B.

The exploration logs within this appendix show our interpretation of the drilling, sampling, and testing data. They indicate the approximate depth where the soils change. Note that the change may be gradual. In the field, we classified the samples taken from the exploration per the methods presented on Figure A-1, Key to Exploration Logs, in this appendix. This figure also provides a legend explaining the symbols and abbreviations used in the log.

Soil Sampling Procedures

Materials encountered in the explorations were classified in the field in general accordance with ASTM International (ASTM) Standard Practice D 2488 “Standard Practice for the Classification of Soils (Visual Manual Procedure).” Soil classification and sampling intervals are shown in the exploration log in this appendix.

Soil samples are obtained from the borings using the following methods.

- Soil samples were obtained from the boring using a standard penetration testing sampler completed in general conformance with ASTM Test Method D 1586 "Standard Method for Penetration Test and Split-Barrel Sampling of Soils". The sampler was driven with a 140-pound cat head hammer falling 30 inches. The N value, or number of blows required to drive the sampler 1 foot or as otherwise indicated into the soils, is shown adjacent to the sample symbols on the boring log. Disturbed samples were obtained from the sampler for subsequent classification and testing.

Pavement Cores

AC cores were collected during the boring explorations using core barrels mounted on the drill rods and delivered to our laboratory. The core information is included in the main body of the report and photographs of the cores are presented in Appendix C.

DCP Testing

We performed a total of 10 DCP probes at explorations A-1, A-2, A-3, A-5, A-6, A-8, A-11, A-12, A-13, and A-14. The DCP consists of a steel extension shaft assembly with a 60-degree hardened steel cone tip attached to one end, which is driven into the subgrade by means of a sliding dual mass hammer. Testing was conducted in accordance with ASTM D 6951/D 6951M-09. Testing provides an evaluation of in-place California Bearing Ratio and Resilient Modulus values for the subgrade. DCP testing was conducted by a member of Haley & Aldrich's geotechnical staff. DCP logs are presented in Appendix C.

Sample Description

Identification of soils in this report is based on visual field and laboratory observations which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field nor laboratory testing unless presented herein. ASTM D 2488 visual-manual identification methods were used as a guide. Where laboratory testing confirmed visual-manual identifications, then ASTM D 2487 was used to classify the soils.

Relative Density/Consistency

Soil density/consistency in borings is related primarily to the standard penetration resistance (N). Soil density/consistency in test pits and probes is estimated based on visual observation and is presented parenthetically on the logs.

SAND or GRAVEL Relative Density	N (Blows/Foot)	SILT or CLAY Consistency	N (Blows/Foot)
Very loose	0 to 4	Very soft	0 to 1
Loose	5 to 10	Soft	2 to 4
Medium dense	11 to 30	Medium stiff	5 to 8
Dense	31 to 50	Stiff	9 to 15
Very dense	>50	Very stiff	16 to 30
		Hard	>30

Moisture

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

USCS Soil Classification Chart (ASTM D 2487)

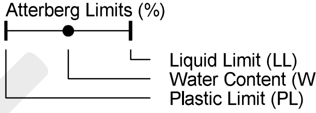
Major Divisions			Symbols		Typical Descriptions
			Graph	USCS	
Coarse Grained Soils More than 50% of Material Retained on No. 200 Sieve	Gravel and Gravelly Soils More than 50% of Coarse Fraction Retained on No. 4 Sieve	Clean Gravels *(<5% fines)*	▨	GW	Well-Graded Gravel; Well-Graded Gravel with Sand
			▨	GP	Poorly Graded Gravel; Poorly Graded Gravel with Sand
		Gravels *(5-12% fines)*	▨	GW-GM	Well-Graded Gravel with Silt; Well-Graded Gravel with Silt and Sand
			▨	GW-GC	Well-Graded Gravel with Clay; Well-Graded Gravel with Clay and Sand
			▨	GP-GM	Poorly Graded Gravel with Silt; Poorly Graded Gravel with Silt and Sand
			▨	GP-GC	Poorly Graded Gravel with Clay; Poorly Graded Gravel with Clay and Sand
		Gravels with Fines *(>12% fines)*	▨	GM	Silty Gravel; Silty Gravel with Sand
			▨	GC	Clayey Gravel; Clayey Gravel with Sand
	Sand and Sandy Soils More than 50% of Coarse Fraction Passing No. 4 Sieve	Sands with few Fines *(<5% fines)*	▨	SW	Well-Graded Sand; Well-Graded Sand with Gravel
			▨	SP	Poorly Graded Sand; Poorly Graded Sand with Gravel
		Sands *(5-12% fines)*	▨	SW-SM	Well-Graded Sand with Silt Well-Graded Sand with Silt and Gravel
			▨	SW-SC	Well-Graded Sand with Clay; Well-Graded Sand with Clay and Gravel
			▨	SP-SM	Poorly Graded Sand with Silt; Poorly Graded Sand with Silt and Gravel
			▨	SP-SC	Poorly Graded Sand with Clay; Poorly Graded Sand with Clay and Gravel
		Sands with Fines *(>12% fines)*	▨	SM	Silty Sand; Silty Sand with Gravel
			▨	SC	Clayey Sand; Clayey Sand with Gravel
Fine Grained Soils More than 50% of Material Passing No. 200 Sieve	Silts		▨	ML	Silt; Silt with Sand or Gravel; Sandy or Gravelly Silt
			▨	MH	Elastic Silt; Elastic Silt with Sand or Gravel; Sandy or Gravelly Elastic Silt
	Silty Clay *(based on Atterberg Limits)*		▨	CL-ML	Silty Clay; Silty Clay with Sand or Gravel; Gravelly or Sandy Silty Clay
	Clays		▨	CL	Lean Clay; Lean Clay with Sand or Gravel; Sandy or Gravelly Lean Clay
			▨	CH	Fat Clay; Fat Clay with Sand or Gravel; Sandy or Gravelly Fat Clay
	Organics		▨	OL/OH	Organic Soil; Organic Soil with Sand or Gravel; Sandy or Gravelly Organic Soil
Highly Organic *(>50% organic material)*			▨	PT	Peat - Decomposing Vegetation - Fibrous to Amorphous Texture

Minor Constituents

	Estimated Percentage
Sand, Gravel	
Trace	<5
Few	5 - 15
Cobbles, Boulders	
Trace	<5
Few	5 - 10
Little	15 - 25
Some	30 - 45

Soil Test Symbols

%F	Percent Passing No. 200 Sieve
AL	Atterberg Limits (%)



CA	Chemical Analysis
CAUC	Consolidated Anisotropic Undrained Compression
CAUE	Consolidated Anisotropic Undrained Extension
CBR	California Bearing Ratio
CIDC	Consolidated Drained Isotropic Triaxial Compression
CIUC	Consolidated Isotropic Undrained Compression
CK0DC	Consolidated Drained k0 Triaxial Compression
CK0DSS	Consolidated k0 Undrained Direct Simple Shear
CK0UC	Consolidated k0 Undrained Compression
CK0UE	Consolidated k0 Undrained Extension
CRSCN	Constant Rate of Strain Consolidation
DS	Direct Shear
DSS	Direct Simple Shear
DT	In Situ Density
GS	Grain Size Classification
HYD	Hydrometer
ILCN	Incremental Load Consolidation
K0CN	k0 Consolidation
kc	Constant Head Permeability
kf	Falling Head Permeability
MD	Moisture Density Relationship
OC	Organic Content
OT	Tests by Others
P	Pressuremeter
PID	Photoionization Detector Reading
PP	Pocket Penetrometer
SG	Specific Gravity
TRS	Torsional Ring Shear
TV	Torvane
UC	Unconfined Compression
UUC	Unconsolidated Undrained Triaxial Compression
VS	Vane Shear
WC	Water Content (%)

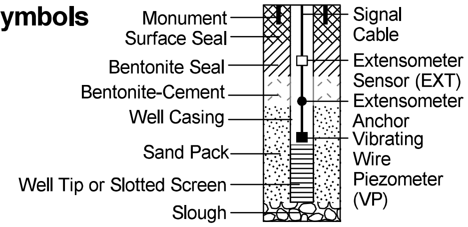
Groundwater Indicators

▽	Groundwater Level on Date or At Time of Drilling (ATD)
▼	Groundwater Level on Date Measured in Piezometer
♀	Groundwater Seepage (Test Pits)

Sample Symbols

☒ 1.5" I.D. Split Spoon
◪ 3.0" I.D. Split Spoon
◨ Modified California Sampler
▣ Rock Core Run
▩ Sonic Core
◫ Thin-walled Sampler
☒ Grab
▥ Cuttings
▥ Push Probe

Well Symbols



HALEY ALDRICH

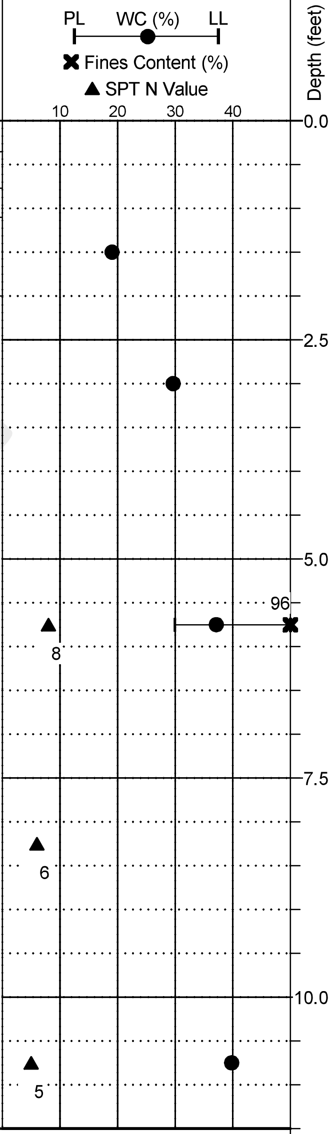
Project: Third Street Improvements
Location: McMinnville, Oregon
Project No.: 0208183-000

Key to Exploration Logs

Figure **A-1**
Sheet **1 of 8**

Date Started: 05/20/2024
Date Completed: 05/20/2024
Drilling Contractor/Crew: Dan J. Fischer Excavating, Inc. / Tyler J.
Logged by: T. Merlin
Checked by: M. Hintz
Drilling Method: Solid Stem Auger
Location: Lat: 45.209974 Long: -123.198898 (OR State Plane N, NAD 83, intn'l ft.)
Rig Model/Type: Trailer-mounted Little Beaver drill rig
Ground Surface Elevation: 151.67 feet (NAVD 88)
Hammer Type: Safety-hammer/Cathead
Comments: Hand augered to 5 feet bgs
Hammer Weight (pounds): 140
Hammer Drop Height (inches): 30
Measured Hammer Efficiency (%): Not Available
Hole Diameter: 6 inches
Well Casing Diameter: NA
Total Depth: 11.5 feet
Depth to Groundwater: Not Identified

Elevation (feet)	Depth (feet)	Sample Data Blow Count	Type	Recovery	Length (inches)	Number Tests	Graphic Log	Material Description
	0.0							Asphalt concrete (4.25-inches thick).
								Portland cement concrete (5-inches thick).
								POORLY GRADED GRAVEL (GP), (dense), moist, gray, up to 3-inch rounded gravel. [FILL]
150.0						G-1 WC		LEAN CLAY (CL), trace fine sand, (soft), moist, dark brown, trace brick pieces
147.5	2.5					G-2 WC		ELASTIC SILT (MH), trace fine sand, soft, moist, brown to yellow-brown. [NATIVE] light brown, iron oxide staining
145.0	5.0	2 3 5		18in.	18	S-1 AL, GS, PP, TV, WC		medium stiff, medium plasticity, no staining (PP = 1.5 tsf, TV = 900 tsf)
142.5	7.5	3 2 4		16in.	18	S-2 PP, TV		brown (PP = 1.25 tsf, TV = 1200 tsf)
	10.0	3 2 3		18in.	18	S-3 PP, TV, WC		(PP = 1.6 tsf, TV = 1250 tsf)
140.0								Bottom of Borehole at 11.5 feet.
137.5	12.5							



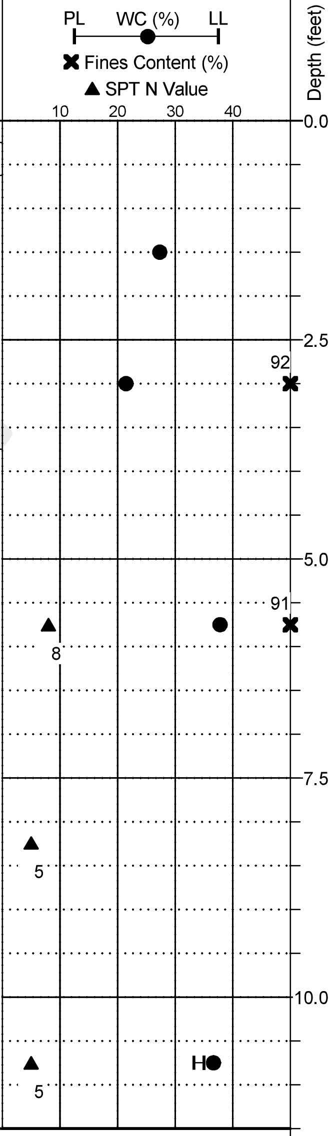
General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

Date Started: 05/20/2024
Date Completed: 05/20/2024
Logged by: T. Merlin
Checked by: M. Hintz
Location: Lat: 45.209975 Long: -123.197681 (OR State Plane N, NAD 83, intn'l ft.)
Ground Surface Elevation: 154.98 feet (NAVD 88)
Comments: Hand augered to 5 feet bgs

Drilling Contractor/Crew: Dan J. Fischer Excavating, Inc. / Tyler J.
Drilling Method: Solid Stem Auger
Rig Model/Type: Trailer-mounted Little Beaver drill rig
Hammer Type: Safety-hammer/Cathead
Hammer Weight (pounds): 140
Hammer Drop Height (inches): 30
Measured Hammer Efficiency (%): Not Available
Hole Diameter: 6 inches
Well Casing Diameter: NA
Total Depth: 11.5 feet
Depth to Groundwater: Not Identified

Elevation (feet)	Depth (feet)	Sample Data: Blow Count	Recovery	Length (inches)	Number / Tests	Material Description
	0.0					Asphalt concrete (3-inches thick).
						Portland cement concrete (4.5-inches thick).
						POORLY GRADED GRAVEL (SP), (dense), moist, gray, up to 3-inch rounded gravel. [FILL]
					G-1 WC	LEAN CLAY (CL), (soft), moist, very dark brown, roots and organics.
152.5	2.5				G-2 GS, WC	dark brown, iron oxide staining, trace brick fragments
						SILT (ML), trace fine sand, soft, moist, brown to yellow-brown. [NATIVE]
150.0	5.0	4 4 4	18in.	18	S-1 GS, PP, TV, WC	medium stiff, brown (PP = 2.6 tsf, TV = 1100 tsf)
147.5	7.5	2 2 3	16in.	18	PP, TV S-2a S-2b	(PP = 1.75 tsf, TV = 1400 tsf) gray-brown, iron oxide staining hydrocarbon odor
145.0	10.0	2 2 3	14in.	18	S-3 AL, PP, TV, WC	less iron oxide staining (PP = 1.6 tsf, TV = 1200 tsf)
142.5	12.5					

Bottom of Borehole at 11.5 feet.



General Notes:
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2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

HALEY ALDRICH

Project: McMinnville Third Street Improvements
Location: McMinnville, Oregon
Project No.: 0208183-000

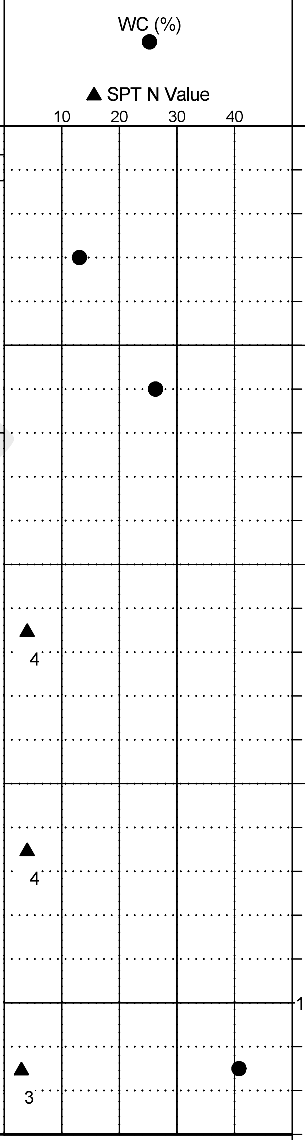
Boring Log
A-3

Figure **A-1**
Sheet **3 of 8**

Date Started: 05/20/2024
Date Completed: 05/20/2024
Logged by: T. Merlin
Checked by: M. Hintz
Location: Lat: 45.210042 Long: -123.195030 (OR State Plane N, NAD 83, intn'l ft.)
Ground Surface Elevation: 158.65 feet (NAVD 88)
Comments: Hand augered to 5 feet bgs

Drilling Contractor/Crew: Dan J. Fischer Excavating, Inc. / Tyler J.
Drilling Method: Solid Stem Auger
Rig Model/Type: Trailer-mounted Little Beaver drill rig
Hammer Type: Safety-hammer/Cathead
Hammer Weight (pounds): 140
Hammer Drop Height (inches): 30
Measured Hammer Efficiency (%): Not Available
Hole Diameter: 6 inches
Well Casing Diameter: NA
Total Depth: 11.5 feet
Depth to Groundwater: Not Identified

Elevation (feet)	Depth (feet)	Sample Data: Blow Count	Type	Recovery	Length (inches)	Number / Tests	Material Description
	0.0						Asphalt concrete (4-inches thick).
							Portland cement concrete (3.5-inches thick).
							POORLY GRADED GRAVEL (GP), (dense), moist, gray, up to 3-inch rounded gravel. [FILL]
157.5						G-1 WC	LEAN CLAY (CL), (soft), moist, dark brown. iron oxide staining
	2.5					G-2 WC	
155.0							SILT (ML), trace fine sand, soft, moist, brown to yellow-brown. [NATIVE]
	5.0	2 2 2		18in.	18	S-1 PP, TV	(PP = 1.9 tsf, TV = 1000 tsf)
152.5							
	7.5	1 2 2		18in.	18	S-2 PP, TV	(PP = 1.5 tsf, TV = 800 tsf)
150.0							
	10.0	2 1 2		18in.	18	S-3 PP, TV, WC	(PP = 1.5 tsf, TV = 700 tsf)
147.5							
	11.5						Bottom of Borehole at 11.5 feet.
	12.5						
145.0							



General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

HALEY ALDRICH
Project: McMinnville Third Street Improvements
Location: McMinnville, Oregon
Project No.: 0208183-000

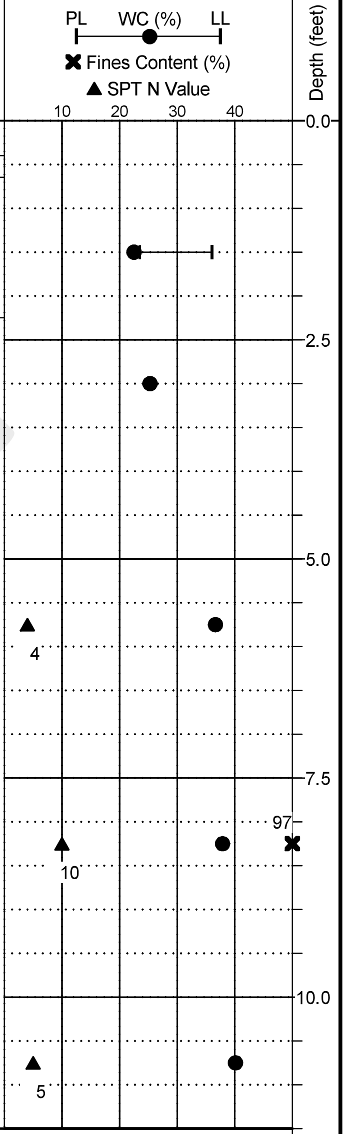
Boring Log
A-6

Figure **A-1**
Sheet **4 of 8**

Date Started: 05/20/2024
Date Completed: 05/20/2024
Logged by: T. Merlin
Checked by: M. Hintz
Location: Lat: 45.209988 Long: -123.193660 (OR State Plane N, NAD 83, intn'l ft.)
Ground Surface Elevation: 160.38 feet (NAVD 88)
Comments: Hand augered to 5 feet bgs

Drilling Contractor/Crew: Dan J. Fischer Excavating, Inc. / Tyler J.
Drilling Method: Solid Stem Auger
Rig Model/Type: Trailer-mounted Little Beaver drill rig
Hammer Type: Safety-hammer/Cathead
Hammer Weight (pounds): 140
Hammer Drop Height (inches): 30
Measured Hammer Efficiency (%): Not Available
Hole Diameter: 6 inches
Well Casing Diameter: NA
Total Depth: 11.5 feet
Depth to Groundwater: Not Identified

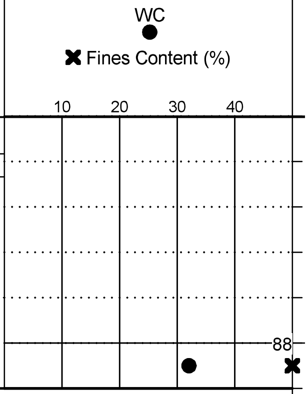
Elevation (feet)	Depth (feet)	Blow Count	Recovery	Length (inches)	Number Tests	Material Description
160.0	0.0					Asphalt concrete (4.75-inches thick).
						Portland cement concrete (3-inches thick).
						POORLY GRADED GRAVEL (GP), (dense), moist, gray, up to 3-inch rounded gravel. [FILL]
					G-1 AL, WC	LEAN CLAY (CL), (soft), moist, dark brown, trace brick fragments
						SILT (ML), trace fine sand, soft, moist, brown to yellow-brown, trace roots. [NATIVE]
157.5	2.5				G-2 WC	
155.0	5.0	2 2 2	15in.	18	S-1 PP, TV, WC	medium stiff (PP = 1.75 tsf, TV = 800 tsf)
152.5	7.5	3 5 5	17in.	18	S-2 GS, PP, TV, WC	stiff (PP = 2.8 tsf, TV = 1250 tsf)
150.0	10.0	3 2 3	15in.	18	S-3 PP, TV, WC	medium stiff, hydrocarbon-like odor (PP = 1.3 tsf, TV = 950 tsf)
						Bottom of Borehole at 11.5 feet.
147.5	12.5					



General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

Date Started: 05/20/2024
Date Completed: 05/20/2024
Contractor/Crew: Haley & Aldrich, Inc.
Logged by: T. Merlin
Checked by: M. Hintz
Rig Model/Type: Hand auger
Location: Lat: 45.209981 Long: -123.191670 (OR State Plane N, NAD 83, intn'l ft.)
Hole Diameter: 3 inches
Well Casing Diameter: NA
Ground Surface Elevation: 162.20 feet (NAVD 88)
Total Depth: 3.0 feet
Depth to Groundwater: Not Identified
Comments: Hand augered to 3 feet bgs; Infiltration test performed at this location

Elevation (feet)	Depth (feet)	Sample Data: Type	Length (inches)	Number / Tests	Material Description	Depth (feet)
162.20	0.0				Asphalt concrete (5-inches thick).	0.0
					Portland cement concrete (3-inches thick).	
					POORLY GRADED GRAVEL (GP), (dense), moist, gray, up to 3-inch rounded gravel. [FILL]	
160.0					SILT (ML), trace fine sand, (soft), moist, brown to yellow-brown. [NATIVE]	
	2.5	▨		G-1 GS, WC		2.5
	3.0				Bottom of Borehole at 3.0 feet.	



General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

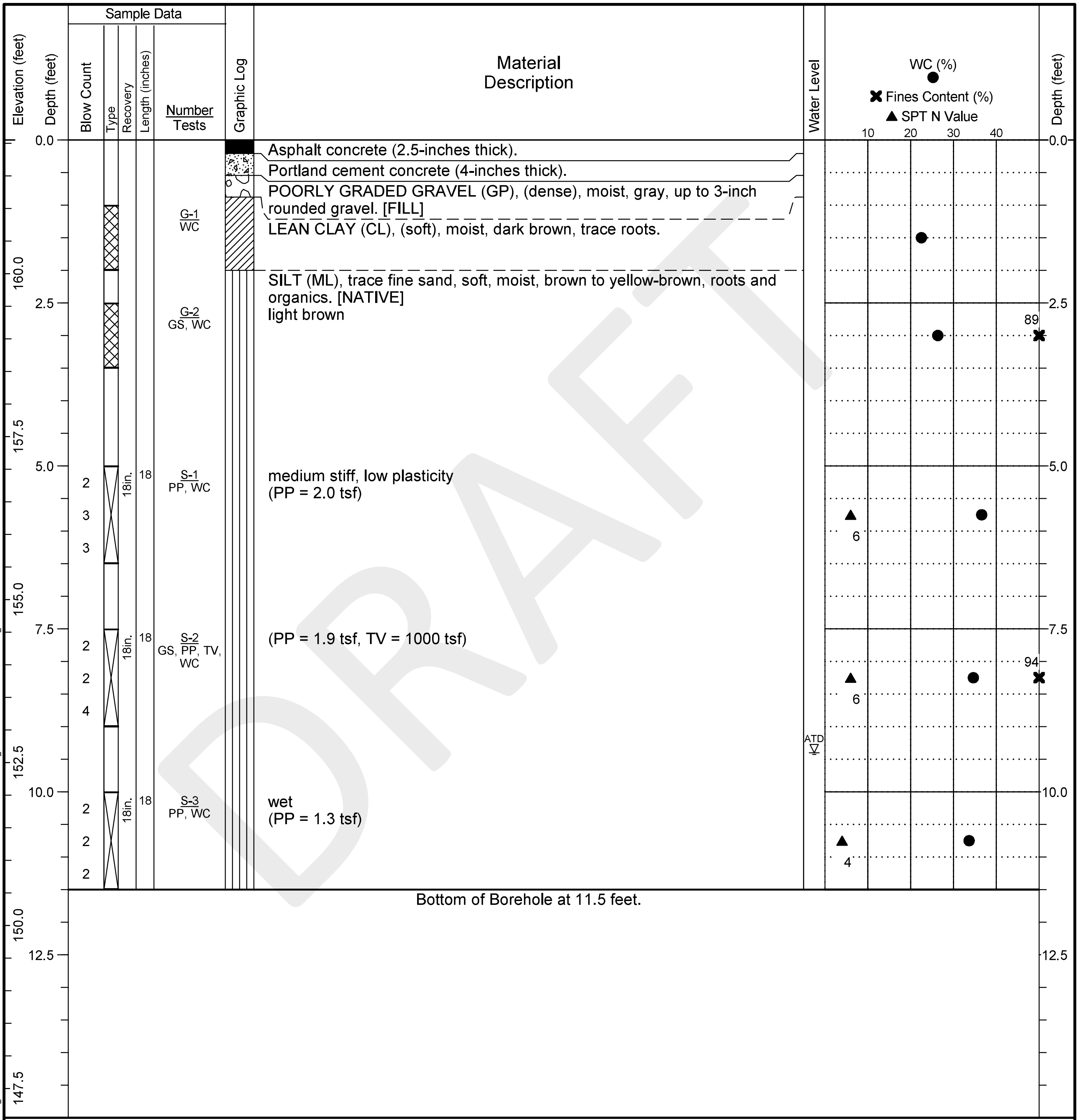
HALEY ALDRICH
Project: McMinnville Third Street Improvements
Location: McMinnville, Oregon
Project No.: 0208183-000

Hand-Auger Log
A-10/IT-1

Figure **A-1**
Sheet **6 of 8**

Date Started: 05/21/2024
Date Completed: 05/21/2024
Logged by: T. Merlin
Checked by: M. Hintz
Location: Lat: 45.209980 Long: -123.191212 (OR State Plane N, NAD 83, intn'l ft.)
Ground Surface Elevation: 162.05 feet (NAVD 88)
Comments: Hand augered to 5 feet bgs

Drilling Contractor/Crew: Dan J. Fischer Excavating, Inc. / Tyler J.
Drilling Method: Solid Stem Auger
Rig Model/Type: Trailer-mounted Little Beaver drill rig
Hammer Type: Safety-hammer/Cathead
Hammer Weight (pounds): 140
Hammer Drop Height (inches): 30
Measured Hammer Efficiency (%): Not Available
Hole Diameter: 6 inches
Well Casing Diameter: NA
Total Depth: 11.5 feet
Depth to Groundwater: 9.4 feet



Depth (feet)	Blow Count	Type	Recovery	Length (inches)	Number / Tests	Material Description	Water Level	WC (%) ● / Fines Content (%) ✖ / SPT N Value ▲
0.0						Asphalt concrete (2.5-inches thick).		
						Portland cement concrete (4-inches thick).		
						POORLY GRADED GRAVEL (GP), (dense), moist, gray, up to 3-inch rounded gravel. [FILL]		
		▨			G-1 WC	LEAN CLAY (CL), (soft), moist, dark brown, trace roots.		●
2.5		▨			G-2 GS, WC	SILT (ML), trace fine sand, soft, moist, brown to yellow-brown, roots and organics. [NATIVE] light brown		● ✖ 89
5.0	2 3 3	☒	18in.	18	S-1 PP, WC	medium stiff, low plasticity (PP = 2.0 tsf)		▲ 6 ●
7.5	2 2 4	☒	18in.	18	S-2 GS, PP, TV, WC	(PP = 1.9 tsf, TV = 1000 tsf)	ATD ▽	▲ 6 ● ✖ 94
10.0	2 2 2	☒	18in.	18	S-3 PP, WC	wet (PP = 1.3 tsf)		▲ 4 ●

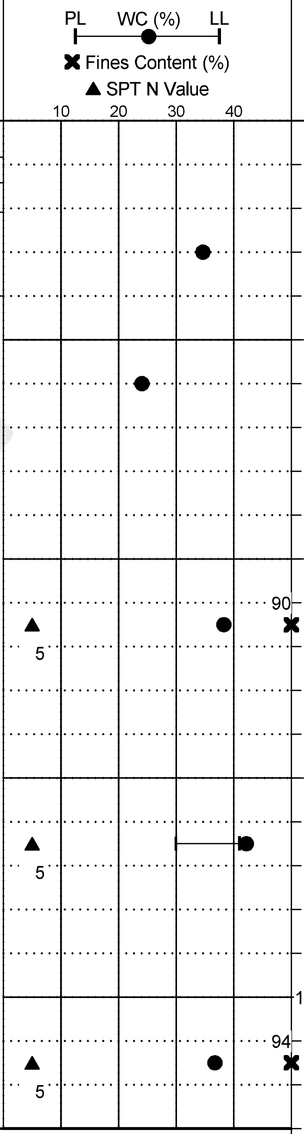
Bottom of Borehole at 11.5 feet.

General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

Date Started: 05/20/2024
Date Completed: 05/20/2024
Logged by: T. Merlin
Checked by: M. Hintz
Location: Lat: 45.209988 Long: -123.189608 (OR State Plane N, NAD 83, intn'l ft.)
Ground Surface Elevation: 161.03 feet (NAVD 88)
Comments: Hand augered to 5 feet bgs

Drilling Contractor/Crew: Dan J. Fischer Excavating, Inc. / Tyler J.
Drilling Method: Solid Stem Auger
Rig Model/Type: Trailer-mounted Little Beaver drill rig
Hammer Type: Safety-hammer/Cathead
Hammer Weight (pounds): 140
Hammer Drop Height (inches): 30
Measured Hammer Efficiency (%): Not Available
Hole Diameter: 6 inches
Well Casing Diameter: NA
Total Depth: 11.5 feet
Depth to Groundwater: Not Identified

		Sample Data							
Elevation (feet)	Depth (feet)	Blow Count	Type	Recovery	Length (inches)	Number Tests	Graphic Log	Material Description	Depth (feet)
	0.0							Asphalt concrete (5-inches thick).	0.0
								Portland cement concrete (3.5-inches thick).	
								POORLY GRADED GRAVEL (GP), (dense), moist, gray, up to 3-inch rounded gravel. [FILL]	
160.0						G-1 WC		trace fine sand, light brown	
								LEAN CLAY (CL), (soft), moist, dark brown.	
								SILT (ML), trace fine sand, soft, moist, brown to yellow-brown, roots and organics. [NATIVE]	
	2.5					G-2 WC			2.5
157.5									
	5.0	2 2 3		18in.	18	S-1 GS, PP, WC		medium stiff (PP = 1.8 tsf)	5.0
155.0									
	7.5	2 3 2		18in.	18	S-2 AL, PP, WC		(PP = 1.3 tsf)	7.5
152.5									
	10.0	2 2 3		18in.	18	S-3 GS, PP, WC		(PP = 1.4 tsf)	10.0
150.0									
								Bottom of Borehole at 11.5 feet.	
	12.5								12.5
147.5									



General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

HALEY ALDRICH

Project: McMinnville Third Street Improvements
Location: McMinnville, Oregon
Project No.: 0208183-000

Boring Log
A-13

Figure **A-1**
Sheet **8 of 8**

APPENDIX B

Laboratory Test Results

APPENDIX B

Laboratory Test Results

General

This appendix documents the laboratory testing that Haley & Aldrich, Inc. completed on select soil samples collected from the explorations. Soil samples obtained from the explorations were transported to our in-house laboratory and evaluated to confirm or modify field classifications. The specific tests conducted are outlined below. Two samples were transported to Cooper Testing Labs in Palo Alto, California, for consolidation testing. The test results are included in this appendix, and where noted, included on the exploration log in Appendix B.

Visual Classifications

Soil samples obtained from the explorations were visually classified in the field and in our geotechnical laboratory based on the Unified Soil Classification System and ASTM International (ASTM) classification methods. ASTM Test Method D 2488 “Standard Practice for the Classification of Soils (Visual-Manual Procedure)” was used to classify soils using visual and manual methods. ASTM Test Method D 2487 “Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)” was used to classify soils based on laboratory test results.

Laboratory Testing

MOISTURE CONTENT

Moisture contents of soil samples were obtained in general accordance with ASTM Test Method D 2216. The results of the moisture content tests are presented on the exploration logs included in Appendix A and on Figure B-1 in this appendix.

ATTERBERG LIMITS

We determined Atterberg limits for selected fine-grained soil samples. The liquid limit and plastic limit were determined in general accordance with ASTM D 4318-84. The results of the Atterberg limits analyses and the plasticity characteristics are summarized in the Liquid and Plastic Limits Test Report, Figures B-2 and on Figure B-1 in this appendix. This relates the plasticity index (liquid limit minus the plastic limit) to the liquid limit.

GRAIN SIZE DISTRIBUTION

Three samples were subjected to a modified grain size classification known as a No. 200-wash. The samples were "washed" through the U.S. Standard No. 200 mesh sieve to determine the relative percentages of coarse- and fine-grained material in the samples. The tests were performed in general accordance with ASTM D 1140. The results are presented on Figure B-1 in this appendix.

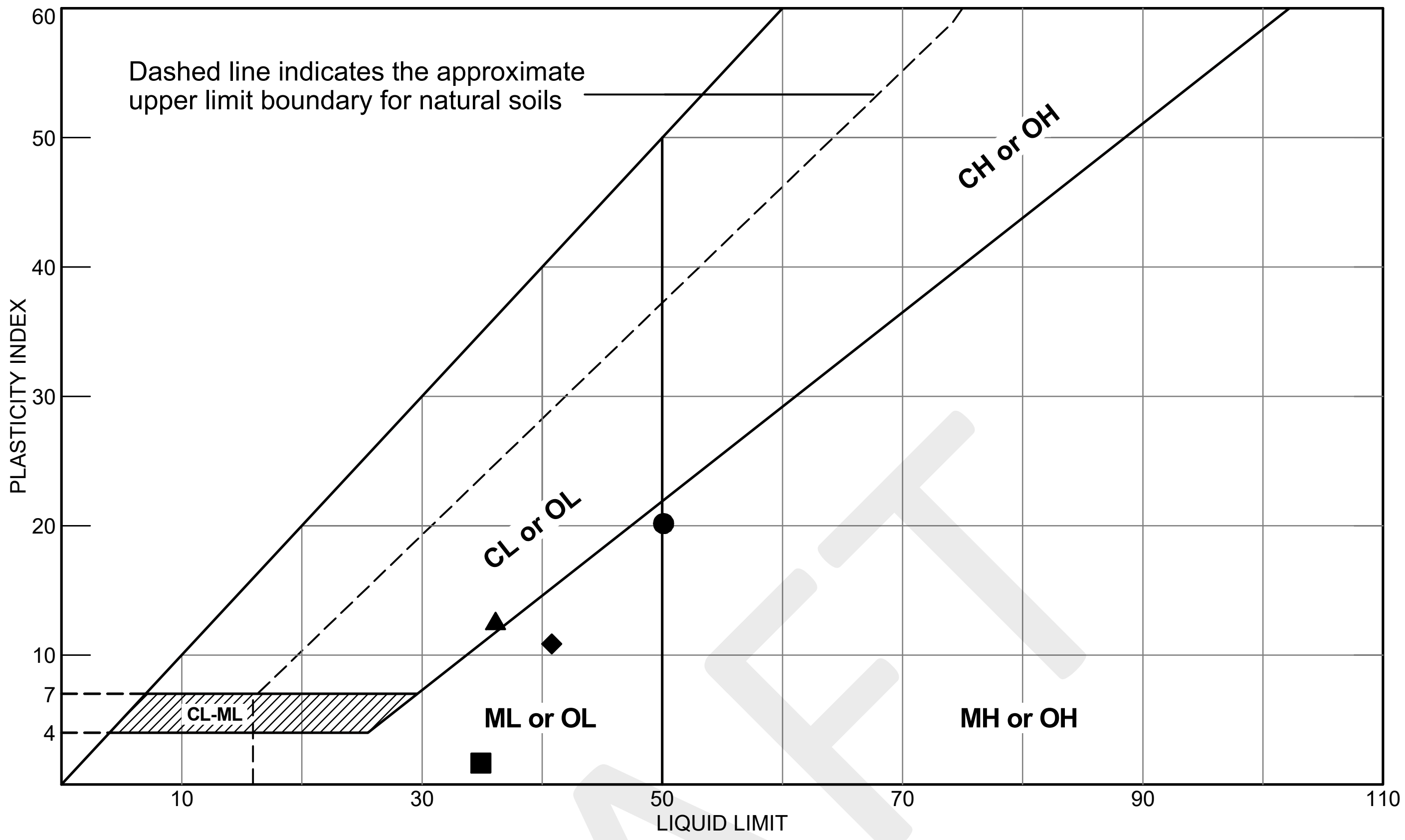
Exploration	Sample ID	Depth	Gravel (%)	Sand (%)	Fines (%)	Liquid Limit	Plastic Limit	Water Content (%)	USCS Group Symbol	Soil Description
A-1	G-1	1.0						19.0		
A-1	G-2	2.5						29.6		
A-1	S-1	5.0	0.0	3.5	96.5	50	30	37.1	MH	ELASTIC SILT
A-1	S-2	7.5								
A-1	S-3	10.0						39.8		
A-3	G-1	1.0						27.3		
A-3	G-2	2.5	0.1	7.9	91.9			21.5	CL	LEAN CLAY
A-3	S-1	5.0	0.0	8.9	91.1			37.8	ML	SILT
A-3	S-2a	7.5								
A-3	S-2b	7.9								
A-3	S-3	10.0				35	33	36.6	ML	SILT
A-6	G-1	1.0						13.1		
A-6	G-2	2.5						26.3		
A-6	S-1	5.0								
A-6	S-2	7.5								
A-6	S-3	10.0						40.8		
A-8	G-1	1.0				36	24	22.5	CL	LEAN CLAY
A-8	G-2	2.5						25.3		
A-8	S-1	5.0						36.6		
A-8	S-2	7.5	0.0	3.3	96.7			37.9	ML	SILT
A-8	S-3	10.0						40.1		
A-10/IT-1	G-1	2.5	0.0	11.8	88.2			32.0	ML	SILT
A-11	G-1	1.0						22.5		
A-11	G-2	2.5	0.0	11.4	88.6			26.3	ML	SILT
A-11	S-1	5.0						36.6		
A-11	S-2	7.5	0.0	5.8	94.2			34.6	ML	SILT
A-11	S-3	10.0						33.6		
A-13	G-1	1.0						34.6		
A-13	G-2	2.5						24.1		
A-13	S-1	5.0	0.0	9.9	90.1			38.3	ML	SILT
A-13	S-2	7.5				41	30	42.2	ML	SILT
A-13	S-3	10.0	0.0	6.1	93.9			36.7	ML	SILT

HALEY ALDRICH

Project: Third Street Improvements
Location: McMinnville, Oregon
Project No.: 0208183-000

Summary of Laboratory Results

Figure B-1
Sheet 1 of 1



Location and Description			LL	PL	PI	-#200	MC%	USCS
● **Source:** A-1 ELASTIC SILT	**Sample No.:** S-1	**Depth:** 5.0 to 6.5	50	30	20	96	37	MH
■ **Source:** A-3 SILT	**Sample No.:** S-3	**Depth:** 10.0 to 11.5	35	33	2	NT	37	ML
▲ **Source:** A-8 LEAN CLAY	**Sample No.:** G-1	**Depth:** 1.0 to 2.0	36	24	12	NT	22	CL
◆ **Source:** A-13 SILT	**Sample No.:** S-2	**Depth:** 7.5 to 9.0	41	30	11	NT	42	ML

Remarks:

●
■
▲ non-native
◆

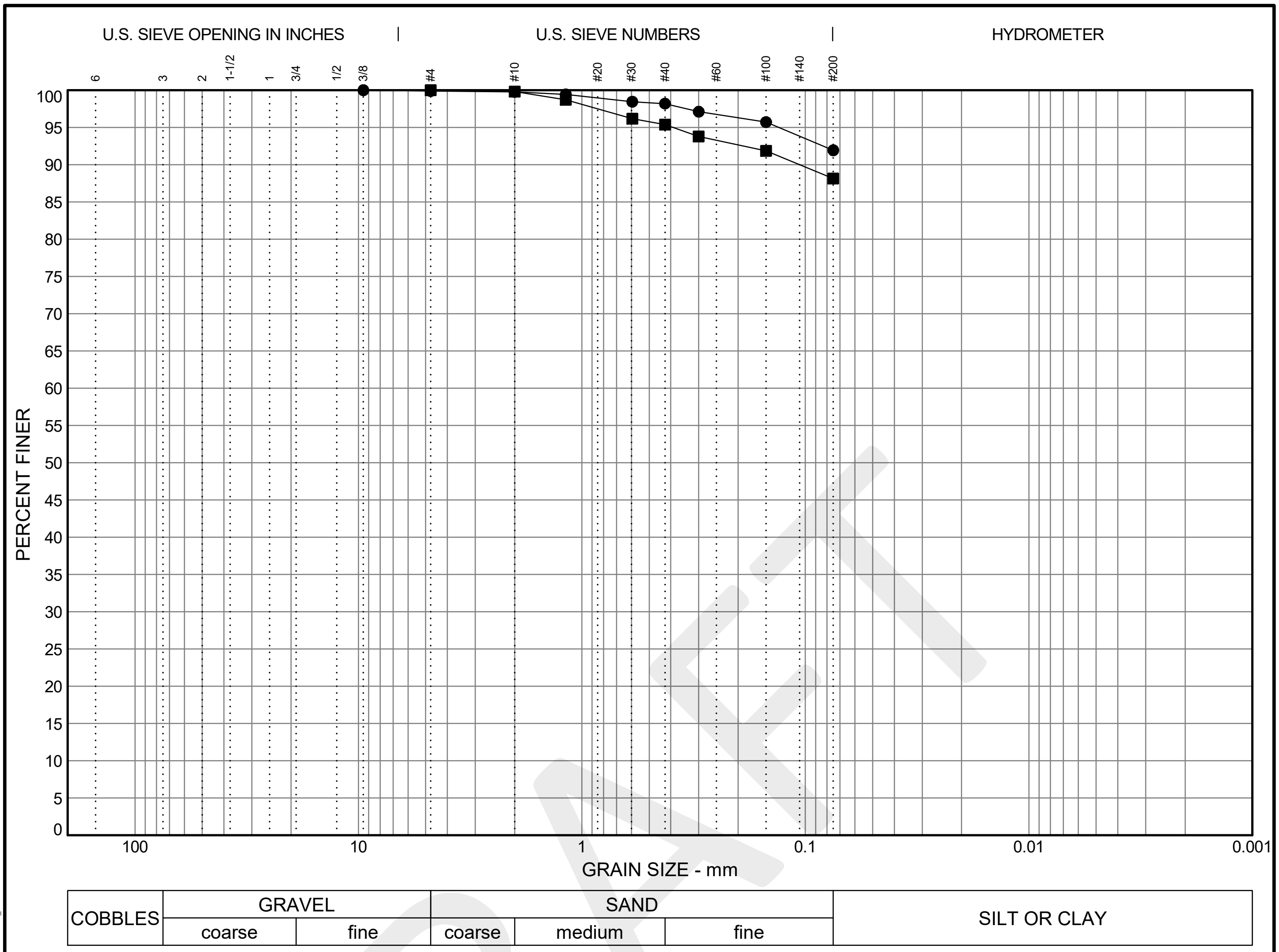
HALEY ALDRICH

Project: Third Street Improvements
Location: McMinnville, Oregon
Project No.: 0208183-000

Liquid Limit, Plastic Limit, and Plasticity Index

Figure **B-2**
Sheet **1 of 1**

HA ATTERBERG LIMITS - C:\USERS\TMERLIN\ONEDRIVE - HALEYALDRICH.COM\DESKTOP\HC_LIBRARY.GLB - 6/20/24 15:11 - \\HALEYALDRICH.COM\SHARE\CF\PROJECTS\0208183\GINT\PERM GINT FILE\0208183-000_MCMINNVILLE THIRD ST IMPROVEMENTS_GINT.GPJ - tmerlin



Location and Description			% Cobbles	% Gravel	% Sand	% Silt	% Clay	MC%	USCS
● **Source:** A-3 LEAN CLAY	**Sample No.:** G-2	**Depth:** 2.5 to 3.5	0.0	0.1	7.9	91.9		21	CL
■ **Source:** A-10/IT-1 SILT	**Sample No.:** G-1	**Depth:** 2.5 to 3.0	0.0	0.0	11.8	88.2		32	ML

☒	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●										
■										

Remarks:

●

■

HALEY ALDRICH

Project: Third Street Improvements
Location: McMinnville, Oregon
Project No.: 0208183-000

Particle-Size Analysis

Figure **B-3**
Sheet **1 of 1**

APPENDIX C

Pavement Core Photo Log and DCP Data

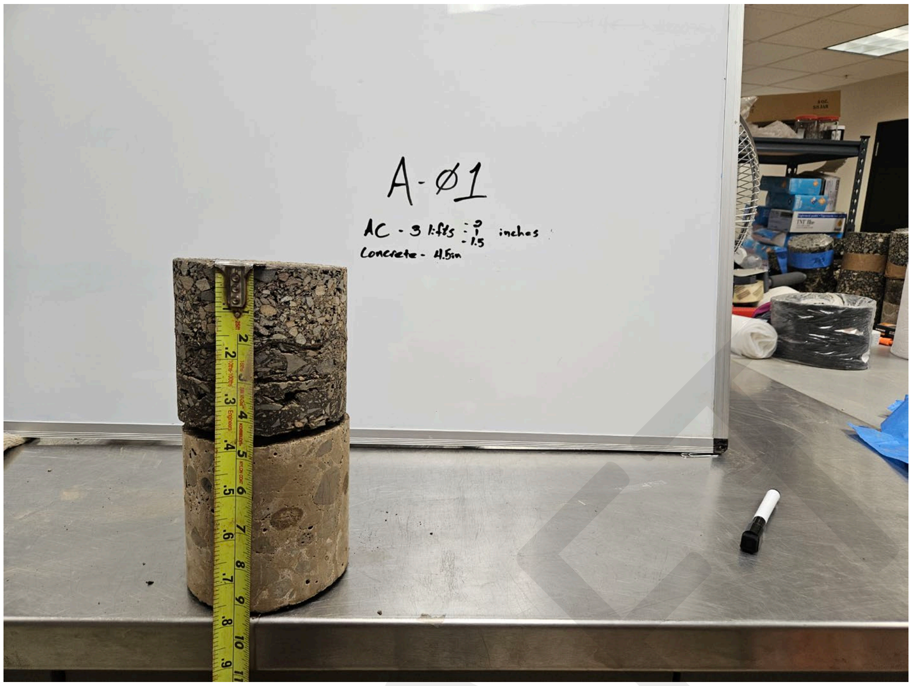


Photo 1: Pavement core from exploration location A-01.

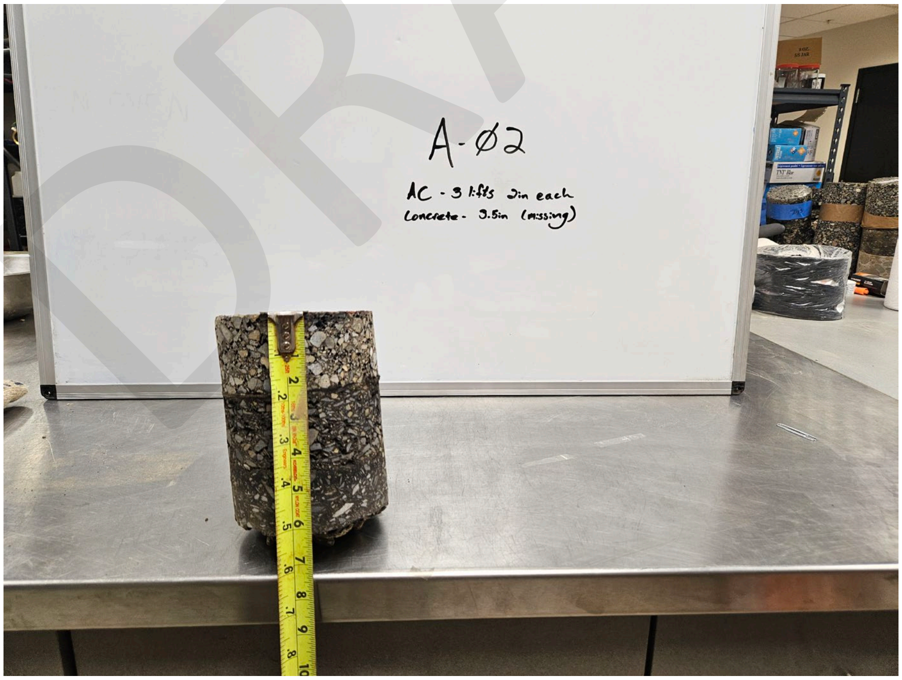


Photo 2: Pavement core from exploration location A-02.

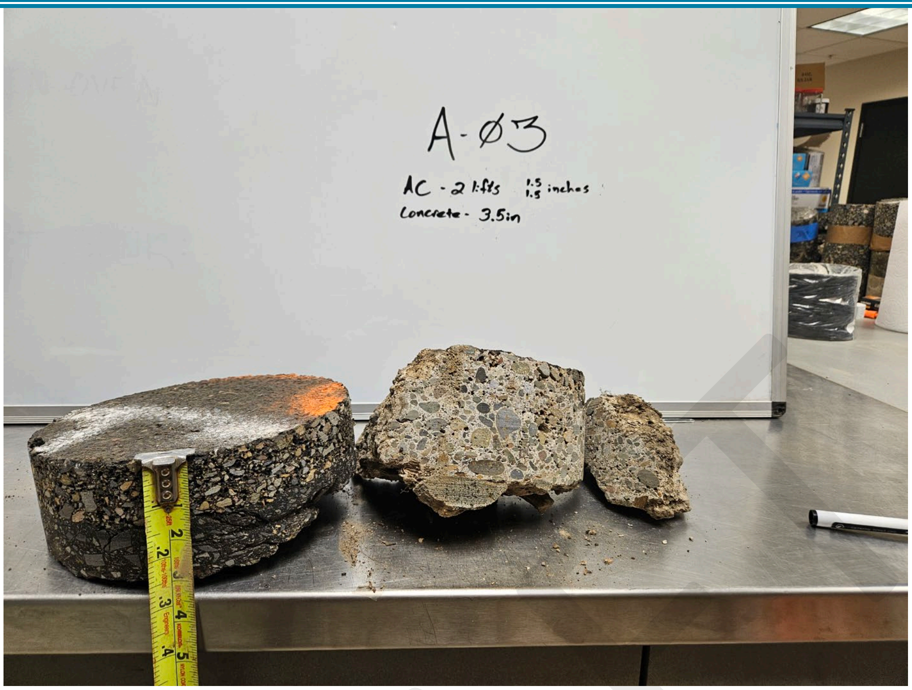


Photo 3: Pavement core from exploration location A-03.

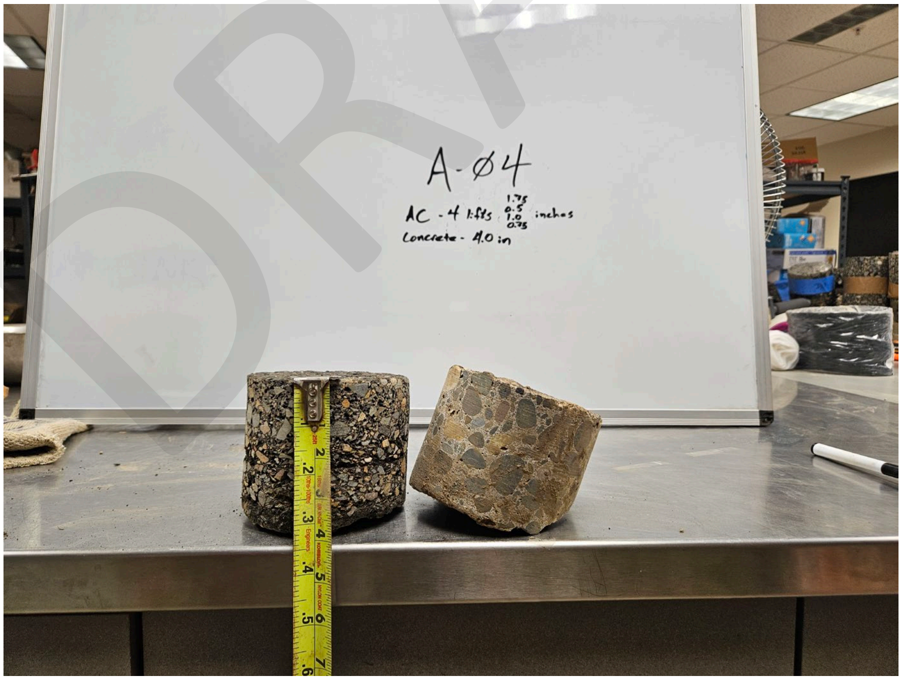


Photo 4: Pavement core from exploration location A-04.

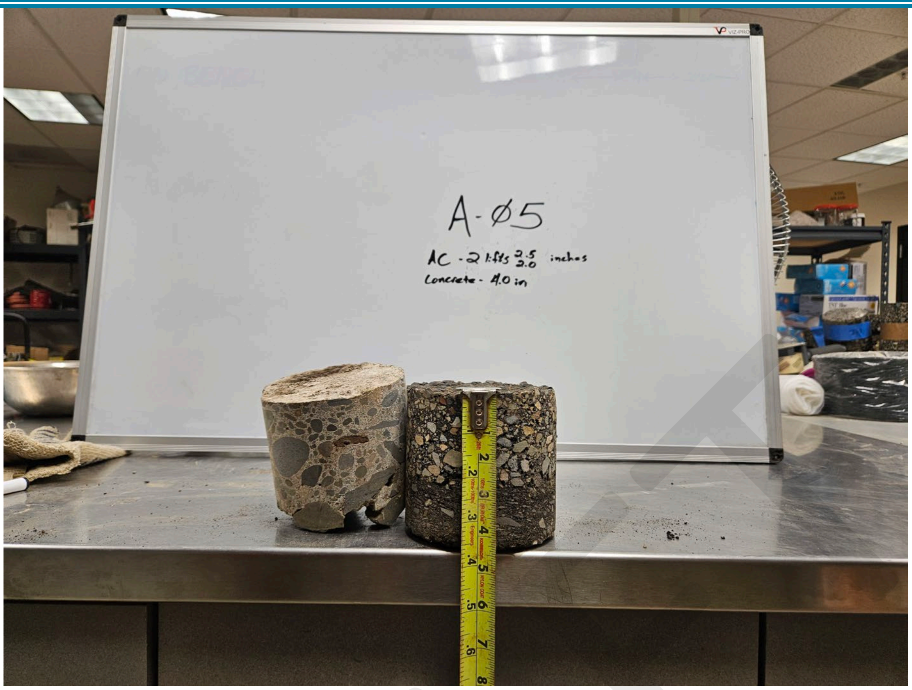


Photo 5: Pavement core from exploration location A-05.

Photo 6: Pavement core from exploration location A-06.

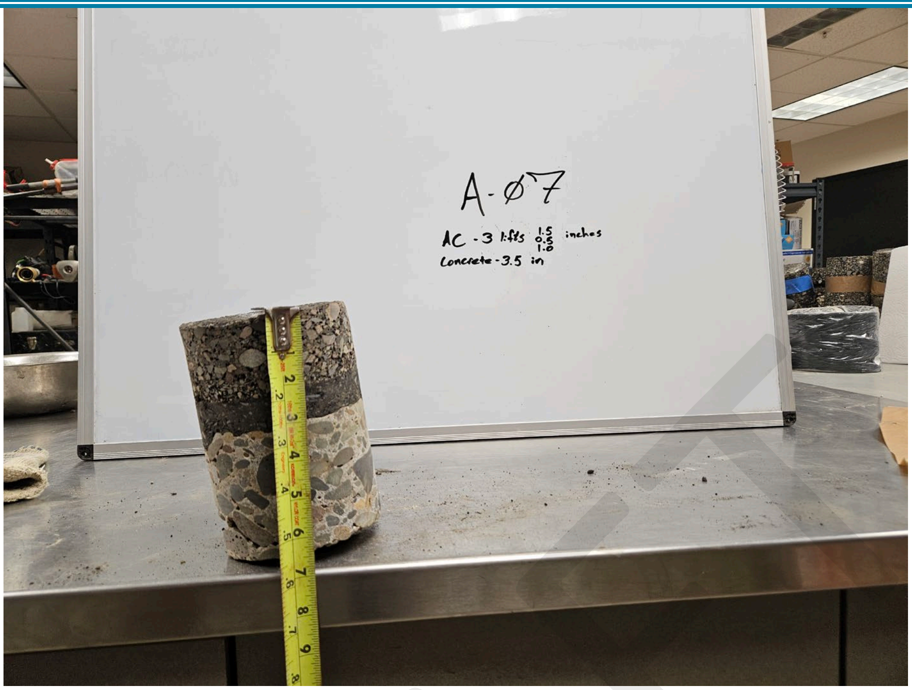


Photo 7: Pavement core from exploration location A-07.

Photo 8: Pavement core from exploration location A-08.

Photo 9: Pavement core from exploration location A-09.

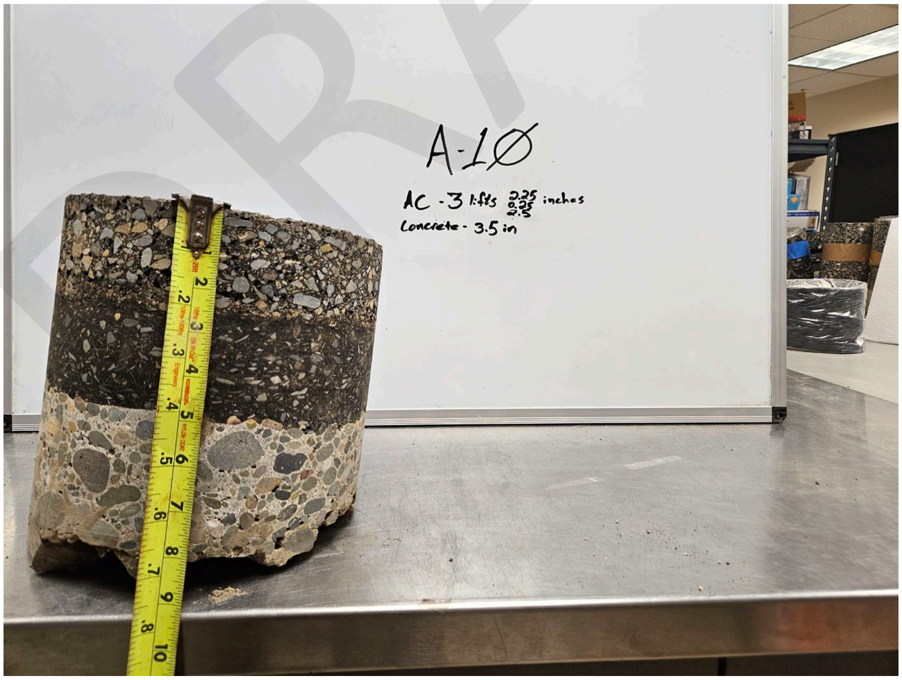


Photo 10: Pavement core from exploration location A-10.

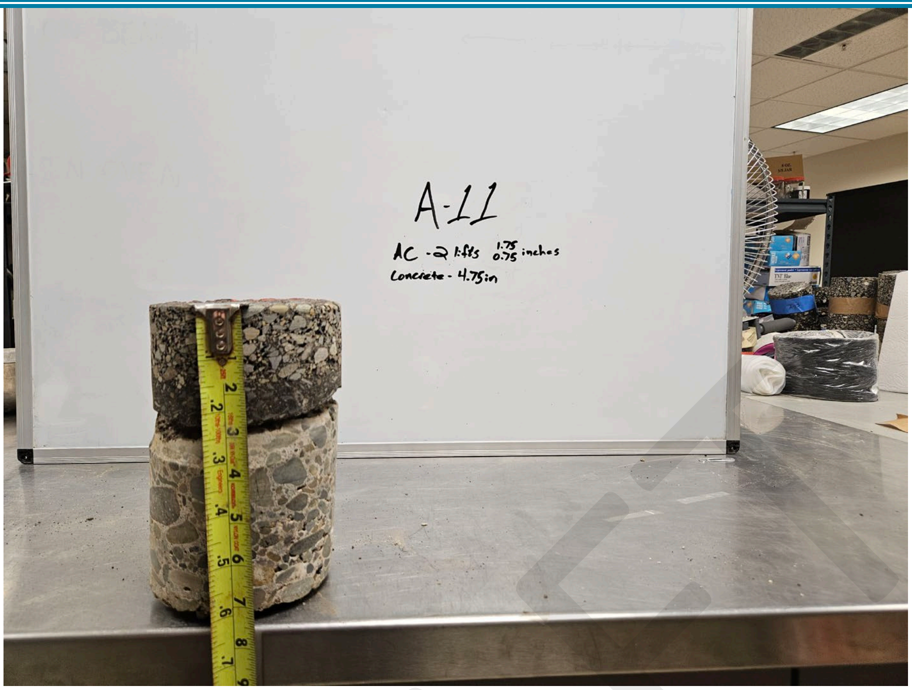


Photo 11: Pavement core from exploration location A-11.

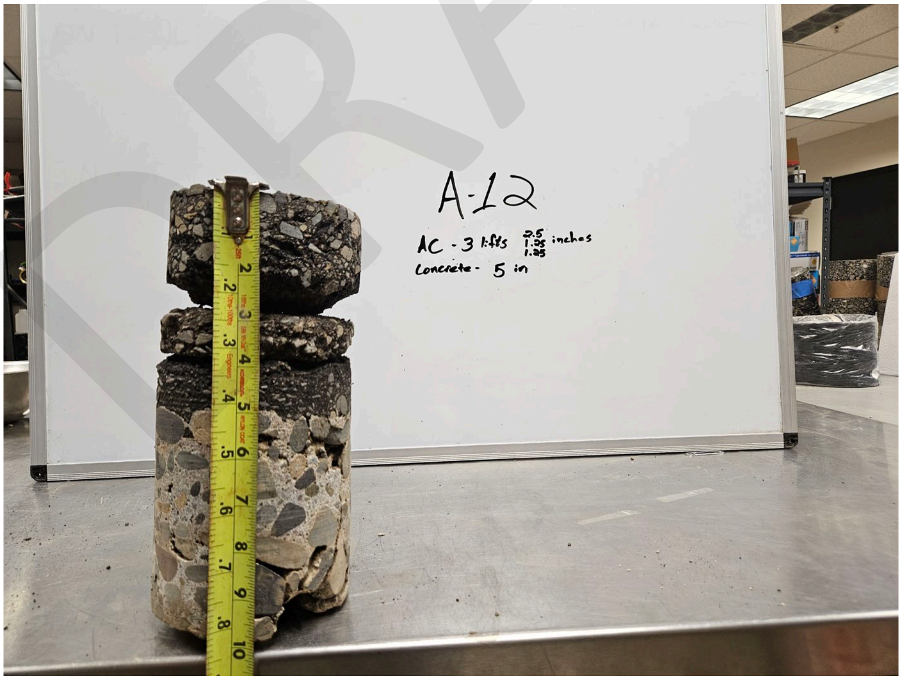


Photo 12: Pavement core from exploration location A-12.

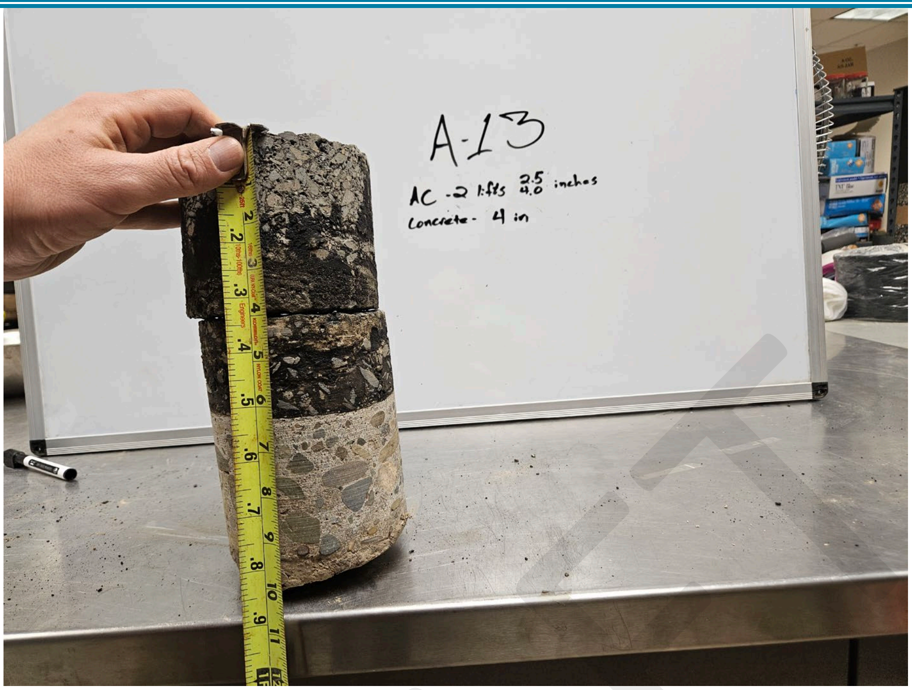


Photo 13: Pavement core from exploration location A-13.

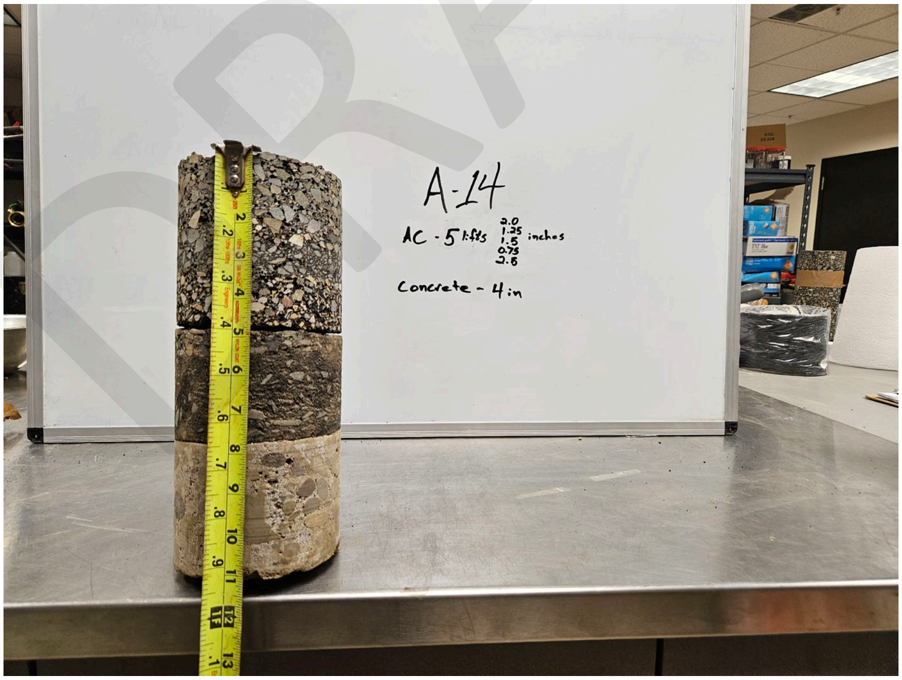


Photo 14: Pavement core from exploration location A-14.

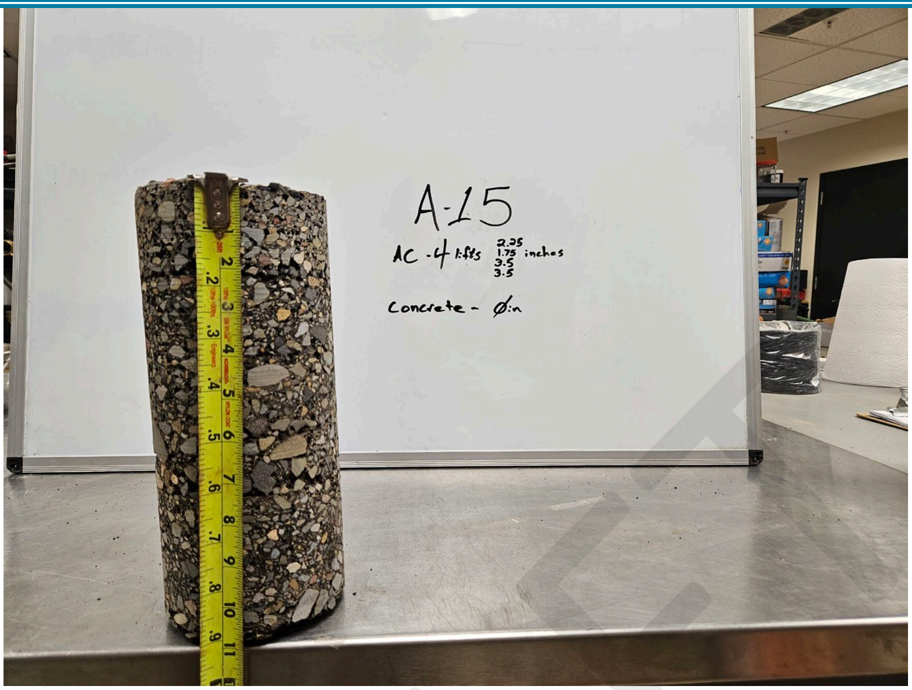


Photo 15: Pavement core from exploration location A-15.

DCP TEST DATA

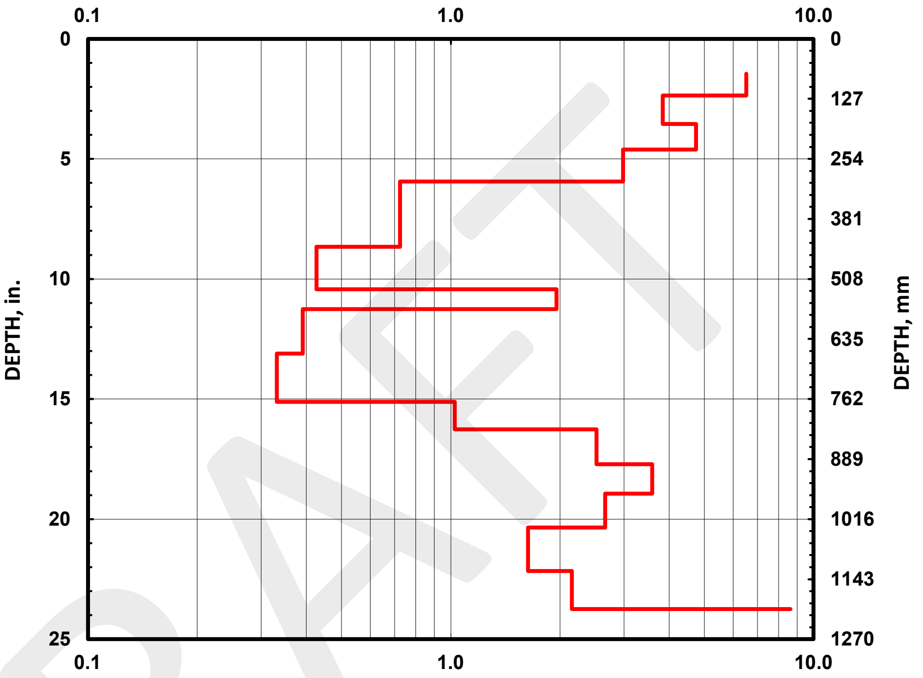
Project: *McMinnville 3rd St*
Date: 22-May-24
Location: A1
Soil Type(s): Lean Clay

Hammer
● 10.1 lbs.
○ 17.6 lbs.
○ Both hammers used

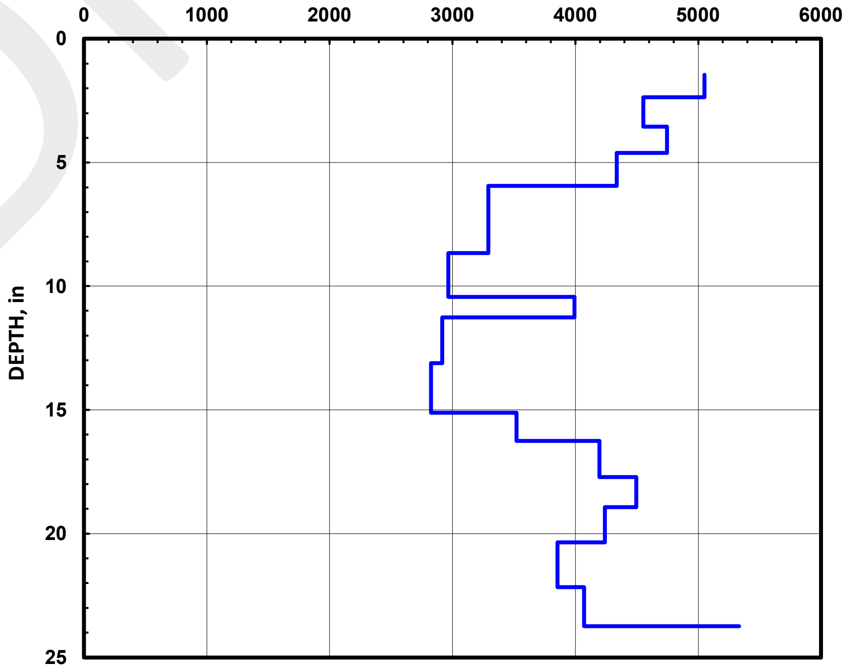
Soil Type
○ CH
● CL
○ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	37	2
2	60	2
2	90	2
2	117	2
2	151	2
2	220	2
1	265	2
1	286	2
1	333	2
1	384	2
1	413	2
2	450	2
2	481	2
2	517	2
2	563	2
2	603	2

CALIFORNIA BEARING RATIO, CBR



RESILIENT MODULUS, Mr (psi)



DCP TEST DATA

Project: *McMinnville 3rd St*
Location: A2

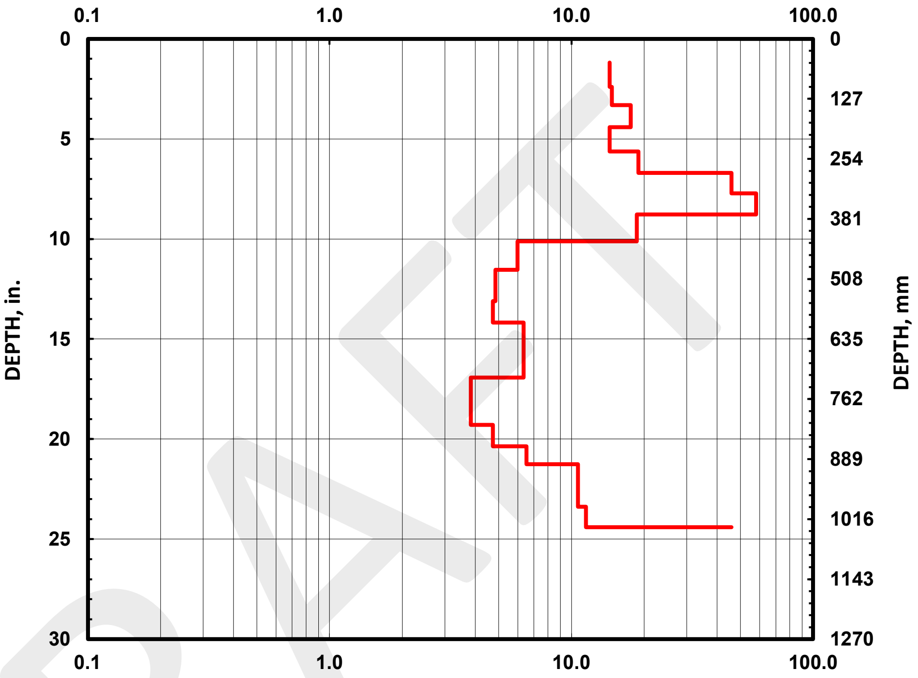
Date: 22-May-24
Soil Type(s): Lean Clay

Hammer
● 10.1 lbs.
○ 17.6 lbs.
○ Both hammers used

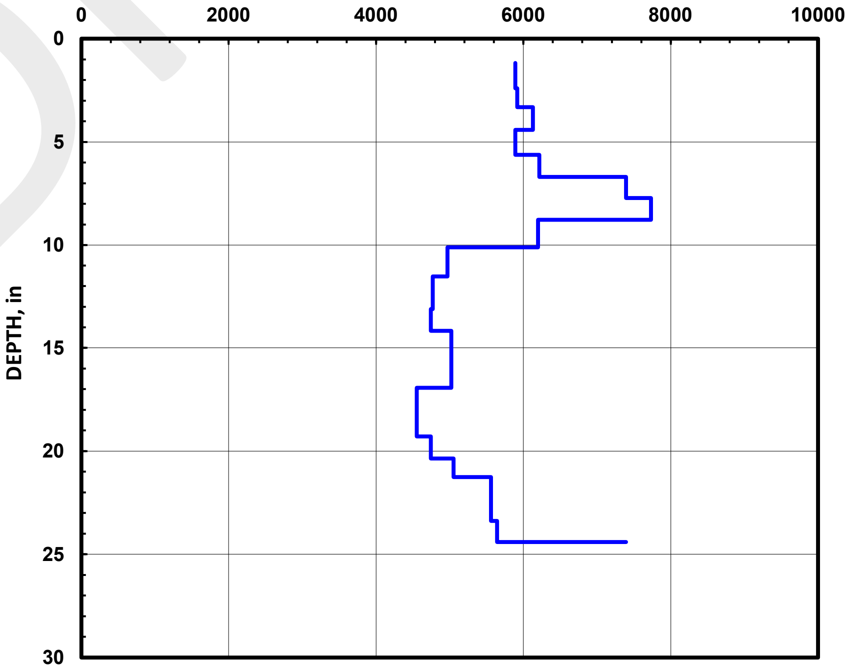
Soil Type
○ CH
● CL
○ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	30	2
4	61	2
3	84	2
4	112	2
4	143	2
4	170	2
6	196	2
7	223	2
5	257	2
3	293	2
3	333	2
2	360	2
3	395	2
3	430	2
2	460	2
2	490	2
2	517	2
2	540	2
3	567	2
3	594	2
3	620	2

CALIFORNIA BEARING RATIO, CBR



RESILIENT MODULUS, Mr (psi)



DCP TEST DATA

Project: *McMinnville 3rd St*

Location: A3

Date: 22-May-24

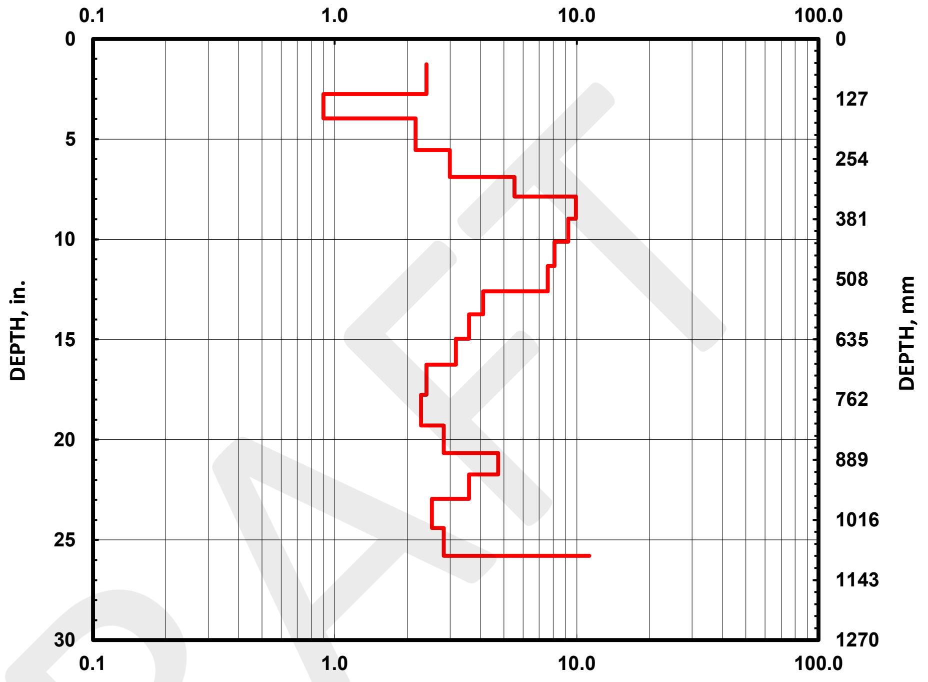
Soil Type(s): Lean Clay

Hammer
● 10.1 lbs.
○ 17.6 lbs.
○ Both hammers used

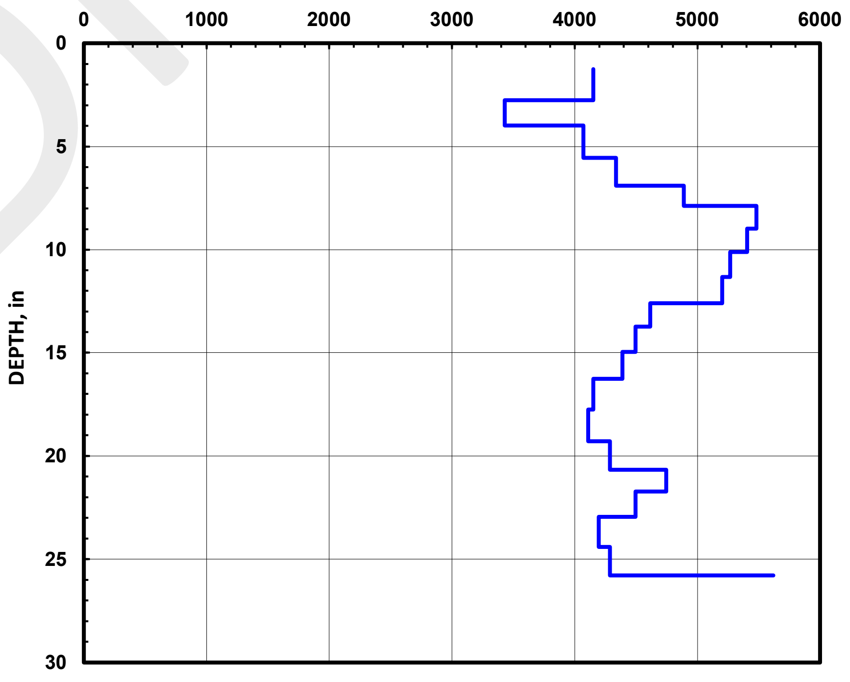
Soil Type
○ CH
● CL
○ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	32	2
2	70	2
1	101	2
2	141	2
2	175	2
2	200	2
3	228	2
3	257	2
3	288	2
3	320	2
2	349	2
2	380	2
2	413	2
2	451	2
2	490	2
2	525	2
2	552	2
2	583	2
2	620	2
2	655	2

CALIFORNIA BEARING RATIO, CBR



RESILIENT MODULUS, Mr (psi)



DCP TEST DATA

Project: *McMinnville 3rd St*
Date: 21-May-24
Location: A5
Soil Type(s): Lean Clay

Hammer
● 10.1 lbs.
○ 17.6 lbs.
○ Both hammers used

Soil Type
○ CH
● CL
○ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	25	2
10	50	2
8	75	2
6	105	2
4	134	2
4	159	2
4	190	2
3	220	2
3	245	2
5	271	2
5	296	2
3	326	2
2	364	2
2	405	2
2	445	2
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2	495	2
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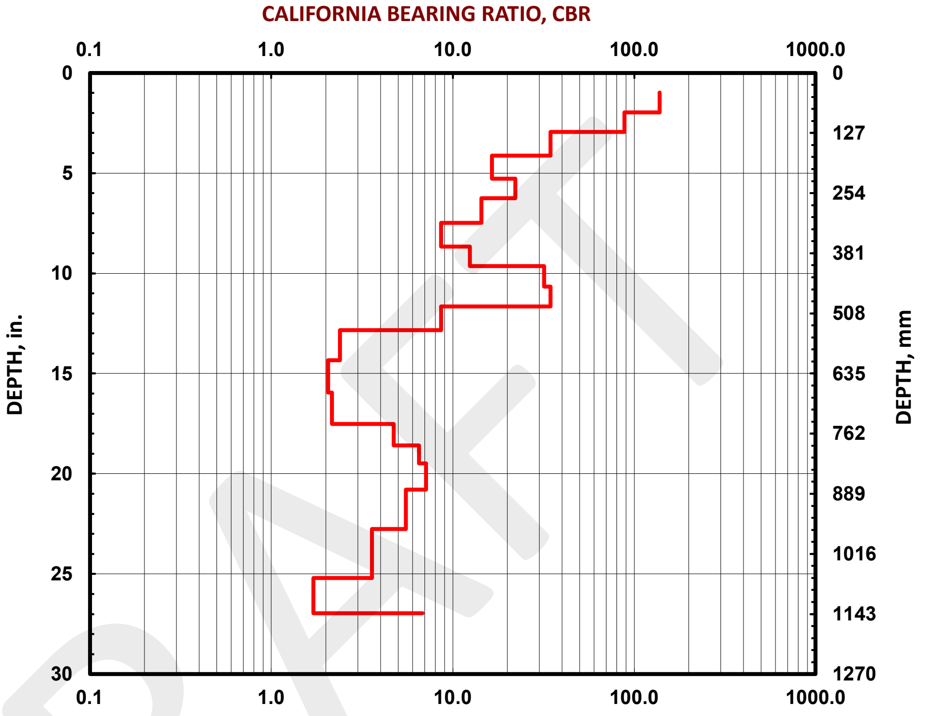
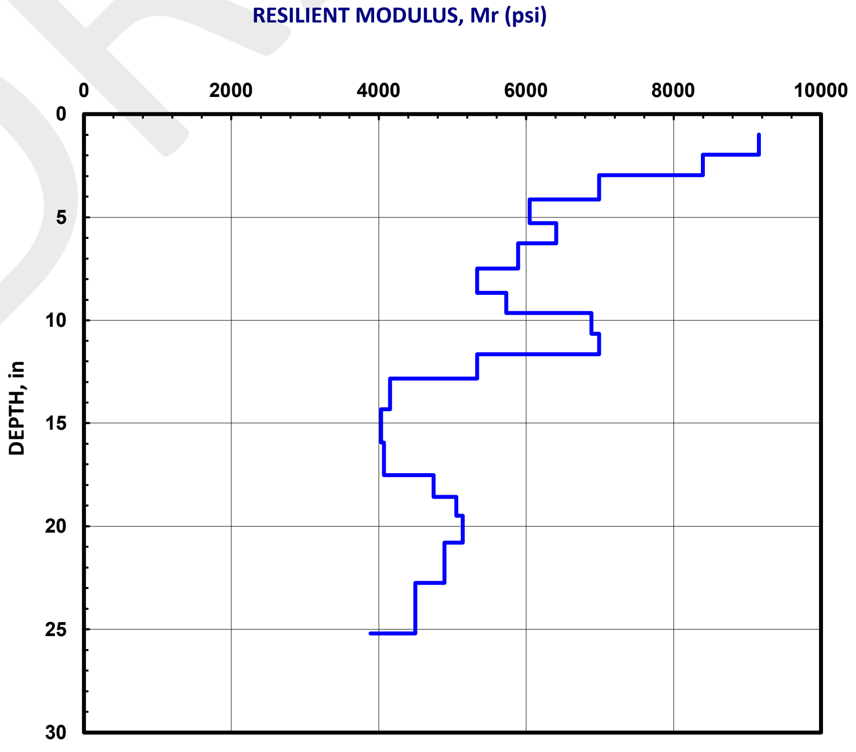
2	553	2
2	578	2
2	609	2
2	640	2
2	685	2





DCP TEST DATA

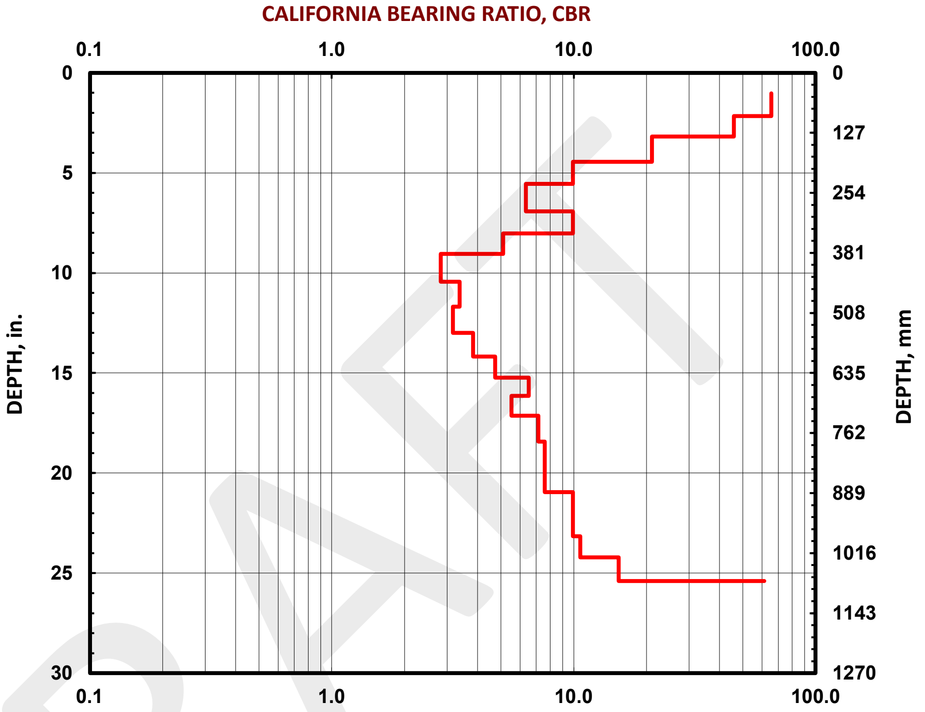
Project: *McMinnville 3rd St*
Location: A6

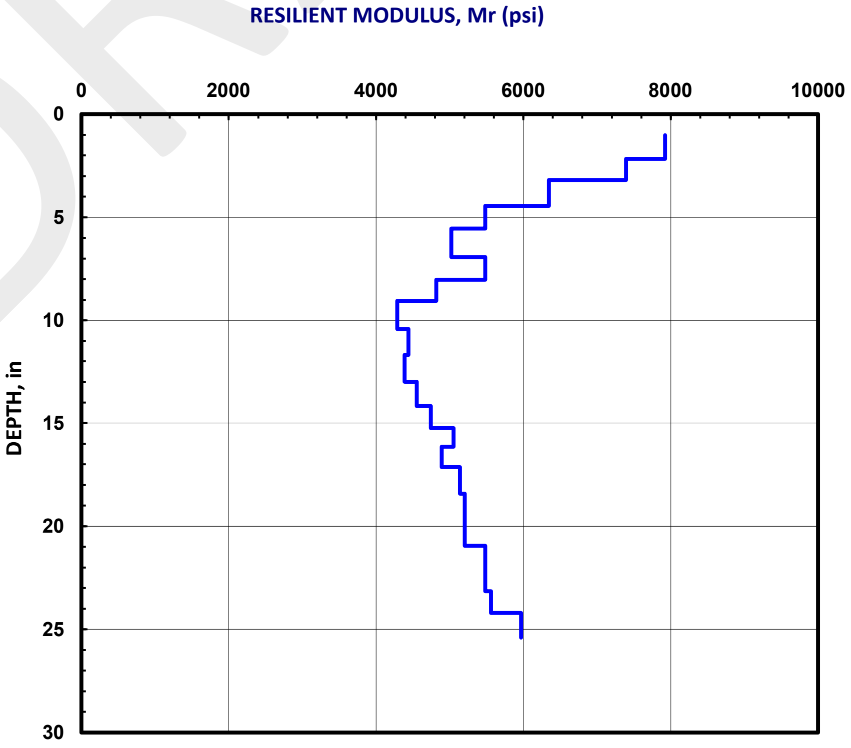
Date: 20-May-24
Soil Type(s): Lean Clay

Hammer
● 10.1 lbs.
○ 17.6 lbs.
○ Both hammers used

Soil Type
○ CH
● CL
○ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	26	2
8	55	2
6	81	2
5	113	2
3	141	2
3	176	2
3	204	2
2	230	2
2	265	2
2	297	2
2	330	2
2	360	2
2	387	2
2	410	2
2	435	2
3	468	2
3	500	2
3	532	2
3	560	2
3	588	2
3	615	2
4	645	2





DCP TEST DATA

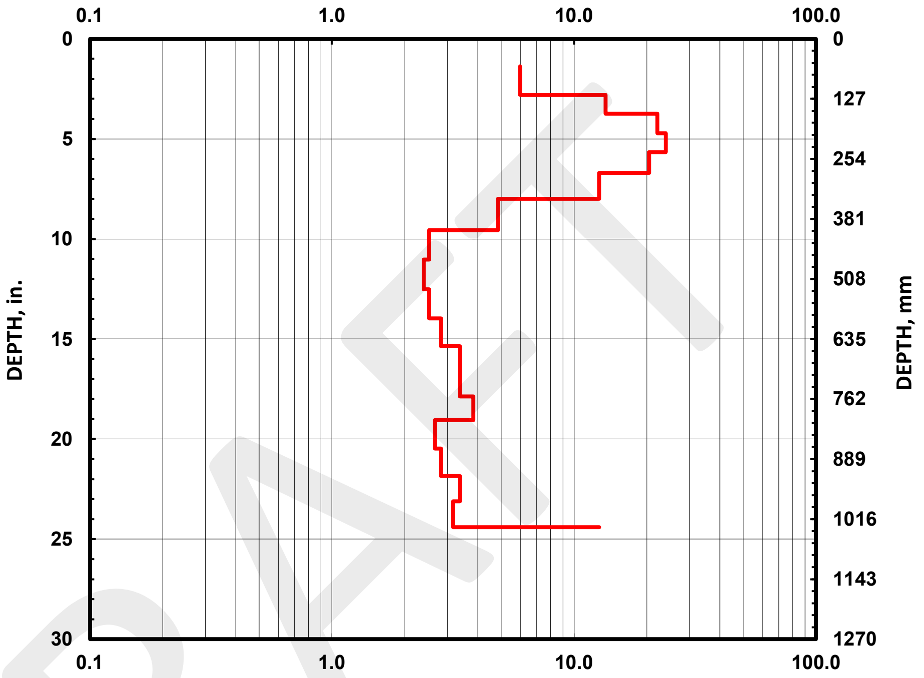
Project: *McMinnville 3rd St*
Location: A8
Date: 22-May-24
Soil Type(s): Lean Clay

Hammer
● 10.1 lbs.
○ 17.6 lbs.
○ Both hammers used

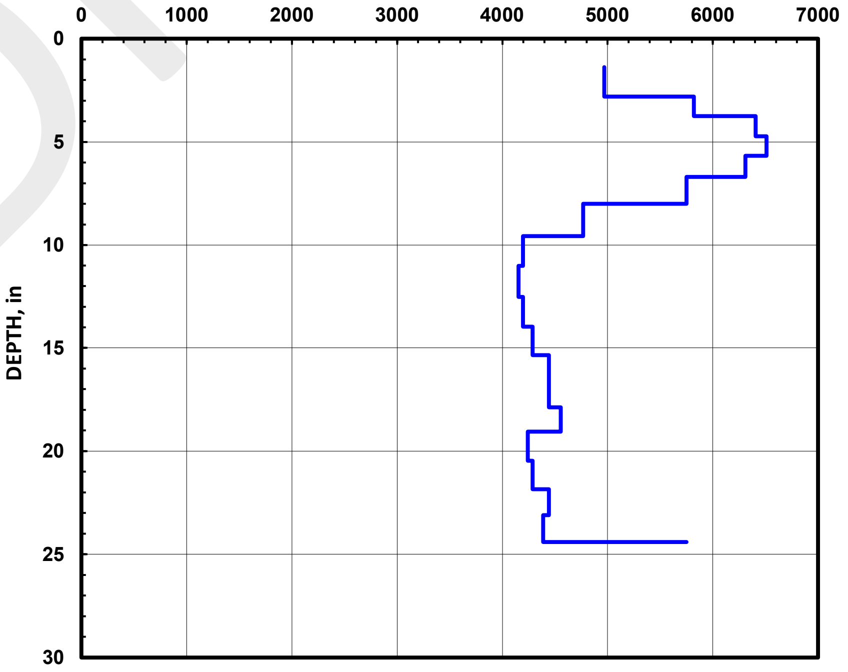
Soil Type
○ CH
● CL
○ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	35	2
3	71	2
3	95	2
4	120	2
4	144	2
4	170	2
4	203	2
3	243	2
2	280	2
2	318	2
2	355	2
2	390	2
2	422	2
2	454	2
2	484	2
2	520	2
2	555	2
2	587	2
2	620	2

CALIFORNIA BEARING RATIO, CBR



RESILIENT MODULUS, Mr (psi)



DCP TEST DATA

Project: *McMinnville 3rd St*

Location: A11

Date: 21-May-24

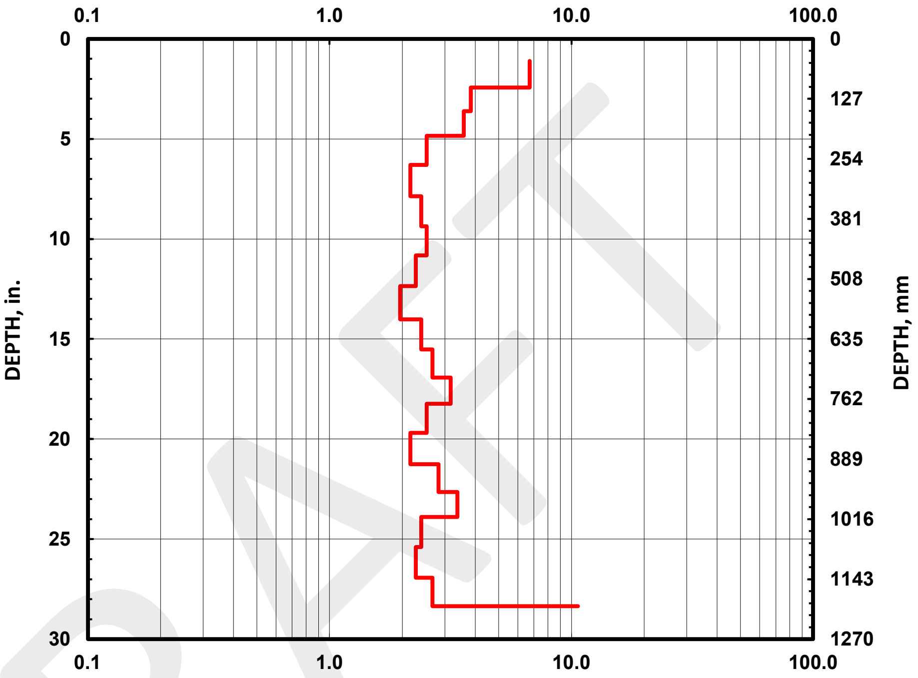
Soil Type(s): Lean Clay

Hammer
● 10.1 lbs.
○ 17.6 lbs.
○ Both hammers used

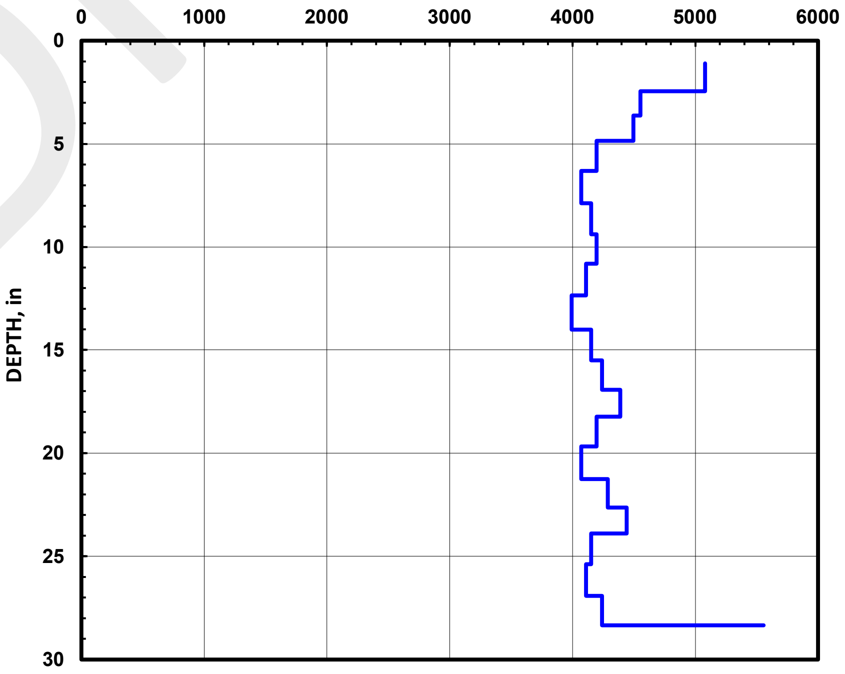
Soil Type
○ CH
● CL
○ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	28	2
3	62	2
2	92	2
2	123	2
2	160	2
2	200	2
2	238	2
2	275	2
2	314	2
2	356	2
2	394	2
2	430	2
2	463	2
2	500	2
2	540	2
2	575	2
2	607	2
2	645	2
2	684	2
2	720	2

CALIFORNIA BEARING RATIO, CBR



RESILIENT MODULUS, Mr (psi)



DCP TEST DATA

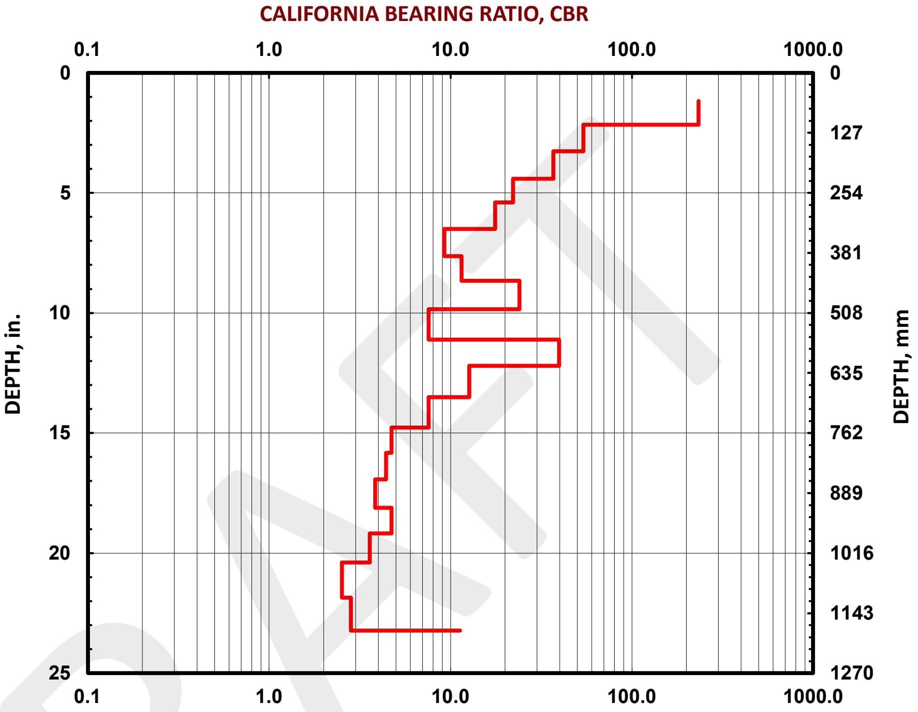
Project: *McMinnville 3rd St*
Location: A12

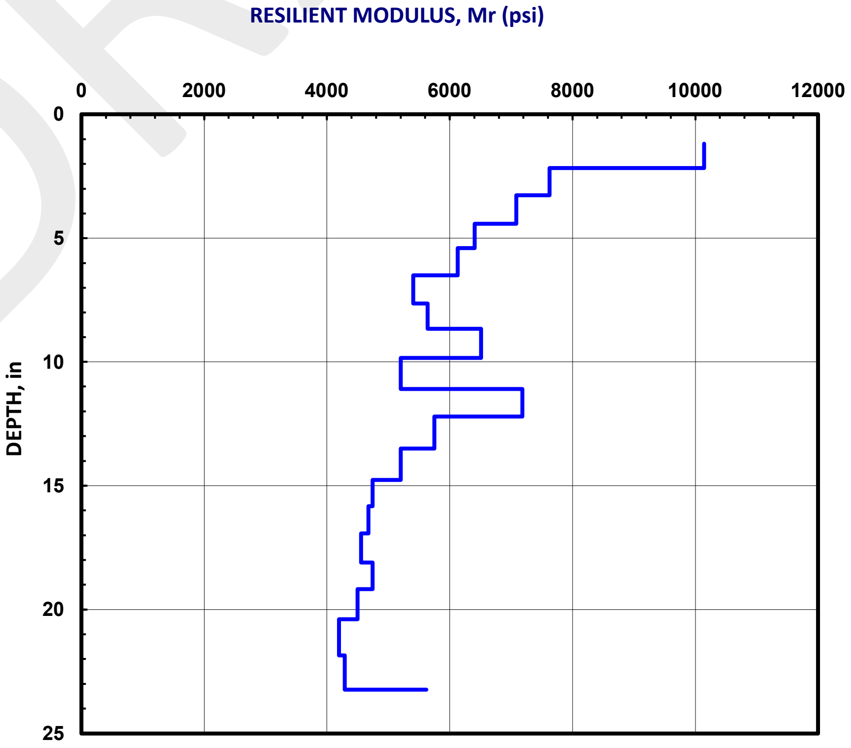
Date: 20-May-24
Soil Type(s): Lean Clay

Hammer
● 10.1 lbs.
○ 17.6 lbs.
○ Both hammers used

Soil Type
○ CH
● CL
○ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	30	2
13	55	2
7	83	2
6	112	2
4	137	2
4	165	2
3	194	2
3	220	2
5	250	2
3	282	2
6	310	2
4	343	2
3	375	2
2	402	2
2	430	2
2	460	2
2	487	2
2	518	2
2	555	2
2	590	2





DCP TEST DATA

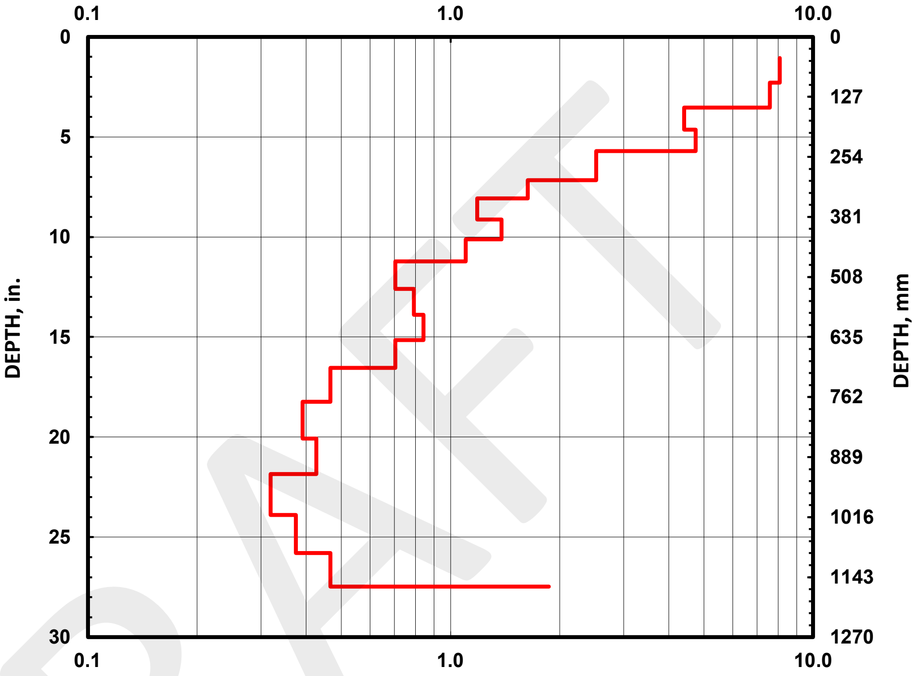
Project: *McMinnville 3rd St*
Location: A13
Date: 20-May-24
Soil Type(s): Lean Clay

Hammer
● 10.1 lbs.
○ 17.6 lbs.
○ Both hammers used

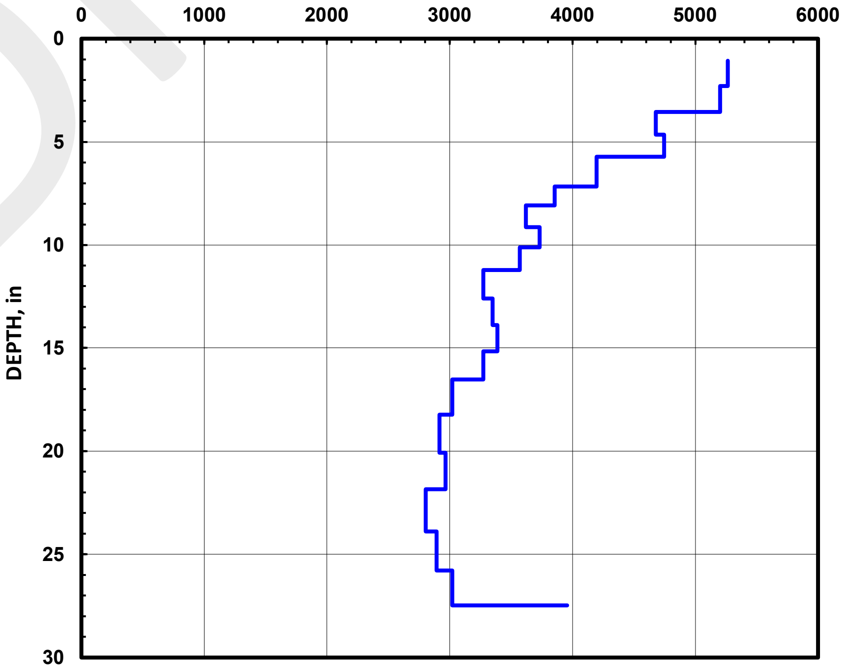
Soil Type
○ CH
● CL
○ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	27	2
3	58	2
3	90	2
2	118	2
2	145	2
2	182	2
1	205	2
1	232	2
1	257	2
1	285	2
1	320	2
1	353	2
1	385	2
1	420	2
1	463	2
1	510	2
1	555	2
1	607	2
1	655	2
1	698	2

CALIFORNIA BEARING RATIO, CBR



RESILIENT MODULUS, Mr (psi)



DCP TEST DATA

Project: *McMinnville 3rd St*

Location: A14

Date: 20-May-24

Soil Type(s): Lean Clay

Hammer
● 10.1 lbs.
○ 17.6 lbs.
○ Both hammers used

Soil Type
○ CH
● CL
○ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	60	2
1	90	2
1	105	2
4	135	2
3	160	2
3	189	2
2	220	2
2	246	2
2	270	2
2	297	2
2	328	2
2	358	2
2	391	2
2	420	2
2	448	2
2	448	2
2	480	2
2	505	2
2	532	2
2	560	2
2	595	2
2	625	2
3	655	2
3	684	2
3	715	2