

Utility Conformance Letter

Merrit Taylor
Town of Erie | Development Engineer
645 Holbrook St.
Erie, CO 80516

Date: October 14th 2025

RE: Utility Conformance Letter - Erie Police Station, 1000 Telleen Avenue, Erie, CO 80516

The Erie Police Department Expansion project involves increasing the footprint of the existing Town of Erie Police Department building by adding an approximately 17,229 SF two story addition for a total of 48,349 SF to accommodate the need for a growing department to serve the increasing population within the Town of Erie, Colorado. The proposed 48,349 square foot building is located on an approximately 5.56 acre parcel at 1000 Telleen Avenue, Erie, Colorado 80516 situated in the SE ¼ of the SE ¼ of Section 13, Township 1N, Range 69W of the 6th Principal Meridian zoned Community Mixed-Use (CMU).

This Utility Conformance Letter for the design and analysis of water and sanitary sewer infrastructure to serve the Erie Police Department Expansion project was prepared by me or under my direct supervision in accordance with the *Town of Erie Engineering Standards, Sections 600 – Water Facilities and 700 – Sanitary Sewer Facilities*. We acknowledge that the City's review is for general conformance with submittal requirements, current design criteria, and standard engineering principles and practices.

The existing police department is served by an 8" water main looped through the site connected to a 12" main in Telleen Avenue to the north as well as a 12" main in County Line Road to the east. An existing 3" meter and service line connects to the building to provide domestic water supply. Wastewater exits the building via a 6" PVC service line which transitions at existing manholes to an 8" PVC line conveyed north to an 18" PVC collector line within Telleen Avenue.

The proposed expansion will demolish existing service lines back to connection points on the north end of the property. A proposed 8" PVC watermain will be constructed as a loop around the expansion with a 6" tap for fire service and a 3" tap for domestic service. The existing 3" water meter will be salvaged and re-installed below grade within a concrete vault for measurement of domestic water use. Sanitary waste will be conveyed by 8" PVC exiting the proposed building on the west side and connecting to an existing manhole on the north side of the property.

Under proposed conditions the following values have been calculated & provided to Professional Engineering Consultants (PEC) by ME Engineers the Mechanical Engineering Consultants on the project for use in this demand and capacity analysis.

- Maximum Domestic Water Demand = 110 gallons per minute (gpm) = 0.25 cfs
- Peak Fire Flow Required = 2375 gallons per minute (gpm) = 5.30 cfs

Sanitary Sewer Analysis

The Erie Police Department is located within the Orchard Glen Sanitary Sewer Basin and the following values were provided to PEC by the Town of Erie for use in evaluating the existing 18" sanitary sewer main where discharge is conveyed to within Telleen Avenue north of the site.

- Master Plan Peak Flow @ (d/D of 0.244) = 1.11 MGD (1.71 cfs)
- Capacity @ (d/D of 0.7) = 7.08 MGD (10.95 cfs)

The proposed peak flow rate of wastewater from the project (110 gpm) or (0.25 cfs) will increase the master plan peak flow from 1.71 cfs to 1.96 cfs, well below the capacity of 10.95 cfs in the basin.

For on-site conveyance of wastewater, the proposed 8" PVC sanitary sewer service has a consistent design slope of 1.00% which provides a capacity of 1.326 cfs under full flow conditions. Exhibit A attached to this Utility Conformance Letter demonstrates sanitary sewer flow is adequately conveyed under peak and max. capacity scenarios to the interceptor line by an 8" PVC service line.

Water Distribution Analysis

The proposed Erie Police Department Expansion is located in Town of Erie potable water pressure zone 2 and Professional Engineering Consultants (PEC) was provided with the following excerpt (**Figure 1**) from the Town of Erie's water distribution model that illustrates residual pressures at select nodes & under the maximum daily demand.



Figure 1 – Town of Erie Water Model Excerpt

In addition to this information provided by the Town the project team collected site specific Hydrant testing data which has been included in Exhibit B following this letter. Hydrant flow testing was performed at two existing hydrants located on site. Hydrant one on the northwest corner of the building and hydrant two on the southeast corner of the building. Residual and static pressures measured at existing hydrants closely match the residual pressure at select nodes adjacent to the site sent over in the Town of Erie Water Model Excerpt.

Hydrant testing provided by Arapahoe Fire Protection did not list data for the final hydrant flow test of flow available at 20 psi residual pressure as typically required by Town of Erie standards and specifications. It is to be noted that the building will be sprinklered and site hydrant flow will supplement the interior fire protection system within the proposed building.

To determine the velocity and head loss within the proposed 8" waterline loop as well as the adjacent 12" watermain three scenarios were evaluated. Firstly, a base scenario of 20 gpm at all nodes to calibrate the model, resulting in expected pressures at select nodes matching site specific hydrant data and the towns model excerpt sent over. Secondly, the peak domestic demand of 110 gpm (0.25 cfs) and lastly the impact on the system when the peak fire flow required of 2,375 gpm (5.30 cfs) is conveyed. A fire flow of 2,375 gpm was determined from Table B105.1(2) within Appendix B of the 2021 International Fire Code for Type IIB construction and proposed building square footage. Exhibit C, attached to this Utility Conformance Letter provides a hydraulic modeling output from the Bentley software product WaterGEMS illustrating these three scenarios.

Modeling outputs indicate that the proposed 8" watermain looped through the site meets Town of Erie criteria for maximum head loss of less than 2' per 1000' during conveyance of the peak domestic demand. Results also show that during peak fire flow demand pressure drop within the system does not exceed the town standards of 30 psi.

The Erie Police Department Expansion Project does not create or pose the potential for adverse impacts on existing town infrastructure that serves the project. The existing sanitary sewer trunk main for the orchard glen basin has significant capacity to capture the peak wastewater effluent from the project. The proposed 8" waterline loop through the site connected to the existing 12" watermain within Telleen Avenue and County Line Road does not experience significant pressure drop under peak domestic demand or required fire flow for the project.

Sincerely,

Nicholas Raley, PE
Professional Engineering Consultants, PA

Exhibit A – Sanitary Sewer Calculations

Channel Report

Erie PD | 8 in. PVC Sanitary Sewer Service

Circular

Diameter (ft) = 0.67

Invert Elev (ft) = 5048.50

Slope (%) = 1.00

N-Value = 0.012

Calculations

Compute by: Known Q

Known Q (cfs) = 0.25

Highlighted

Depth (ft) = 0.20

Q (cfs) = 0.250

Area (sqft) = 0.09

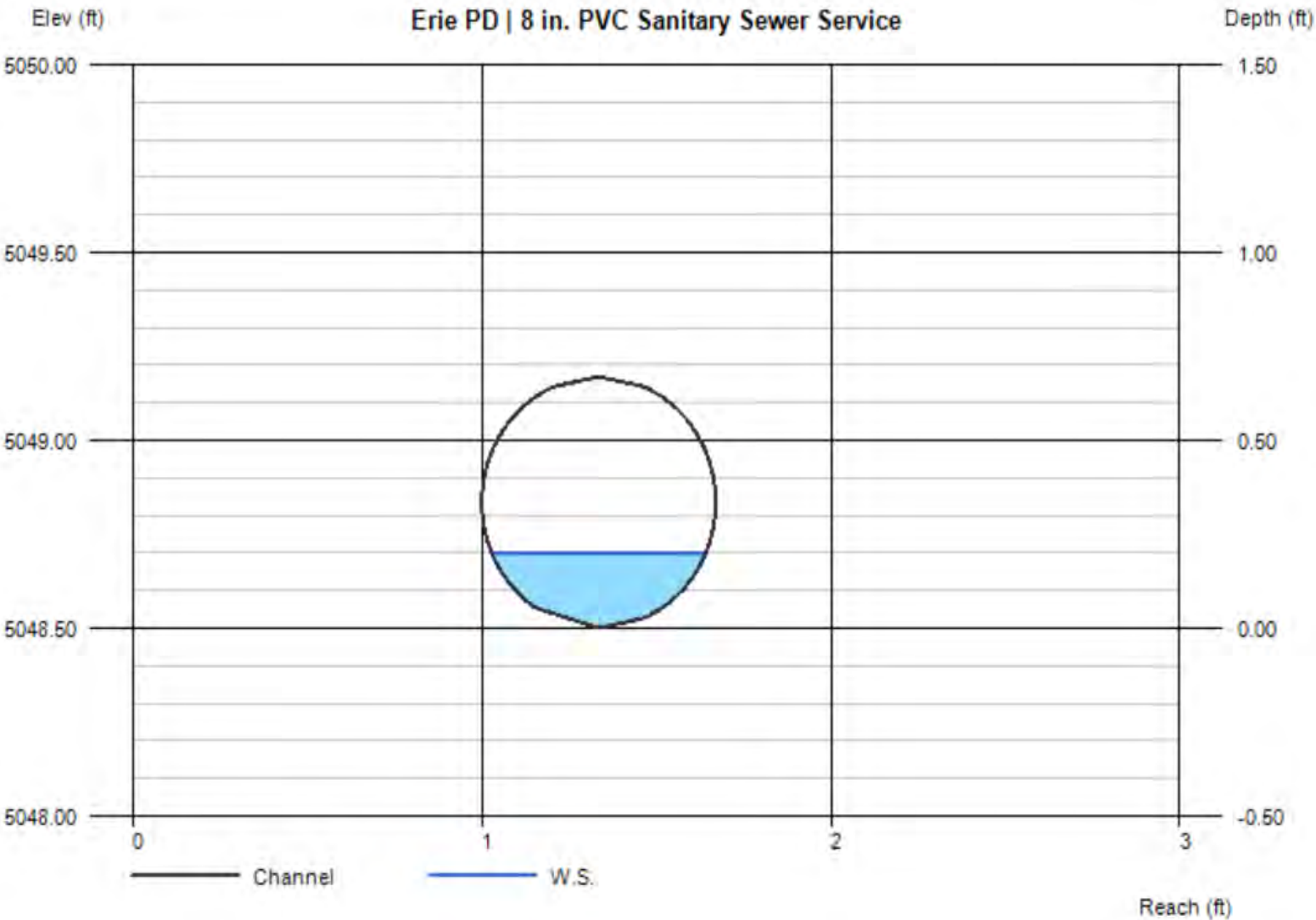
Velocity (ft/s) = 2.81

Wetted Perim (ft) = 0.78

Crit Depth, Yc (ft) = 0.23

Top Width (ft) = 0.61

EGL (ft) = 0.32



Depth	Q	Area	Veloc	Wp	Yc
(ft)	(cfs)	(sqft)	(ft/s)	(ft)	(ft)
0.07	0.028	0.018	1.51	0.43	0.08
0.13	0.117	0.051	2.32	0.62	0.16
0.20	0.260	0.089	2.92	0.78	0.24
0.27	0.447	0.132	3.39	0.92	0.32
0.34	0.668	0.177	3.77	1.06	0.39
0.40	0.894	0.221	4.04	1.19	0.45
0.47	1.113	0.264	4.21	1.33	0.50
0.54	1.297	0.302	4.29	1.48	0.54
0.60	1.414	0.334	4.23	1.68	0.56
0.67	1.326	0.353	3.76	2.10	0.55

Peak Flow

Full Flow Capacity

Exhibit B – Fire Hydrant Test Results

Report of Inspection / Test

Annual NFPA 25

2025-02-24

Property

Erie Police Department
5114-3

1000 Telleen Ave
Erie CO 80516-8418

Chris Holland

Print Date: 2025-02-25

Conducted by: Brent Disher

Arapahoe Fire Protection
Arapahoe Fire Protection
11901 E. 14th
Ave CO 80010
303-366-4905



Report of Inspection / Test for Asset - 1 Hydrant

Fire Hydrant Information

Location

North west front of building

QUESTIONS

Is there a hydrant wrench that is available and accessible?	☐ Yes ☐ No ☒ NA	Is the hydrant free from cracks or leaks at outlets and on the top?	☒ Yes ☐ No ☐ NA
Are pumper and nozzle caps tight?	☒ Yes ☐ No ☐ NA	Is the hydrant properly painted and is the paint in good condition?	☒ Yes ☐ No ☐ NA
Does the operating nut turn with no difficulty?	☒ Yes ☐ No ☐ NA	Did the hydrant flow until clear (minimum of 1 minute)?	☒ Yes ☐ No ☐ NA
Are all dry barrels which require pumping identified?	☐ Yes ☐ No ☒ NA	Does the hydrant completely shut off?	☒ Yes ☐ No ☐ NA
Have the strainers been cleaned (if possible)?	☐ Yes ☐ No ☒ NA	Did monitor nozzle flowed acceptable water?	☐ Yes ☐ No ☒ NA
Have backflow devices, if installed, passed full flow test?	☐ Yes ☐ No ☒ NA	Is there no ice or water in the barrel?	☒ Yes ☐ No ☐ NA
Are hydrant caps, stems, outlets, and threads lubricated and in good condition?	☒ Yes ☐ No ☐ NA	Is Exposed piping properly secured and free from leaks or physical damage?	☒ Yes ☐ No ☐ NA
Is the Operating nut not worn, twisted or broken?	☒ Yes ☐ No ☐ NA	Is the Road box and shutoff valve visible and accessible?	☐ Yes ☐ No ☒ NA
Have dry barrels drained in at least 1 hour?	☒ Yes ☐ No ☐ NA	Have control valves been operated through complete range??	☐ Yes ☐ No ☒ NA
Are strainers, if installed, free from corrosion and not blocked?	☐ Yes ☐ No ☒ NA	Did monitor nozzles move through complete range?	☐ Yes ☐ No ☒ NA
Are monitor nozzles lubricated?	☐ Yes ☐ No ☒ NA		

Hydrant Flow Test

Static Pressure	60	Residual Pressure	50
Pitot Pressure	58	Orifice Size	0.90

Report of Inspection / Test

Annual NFPA 25

2025-02-24

Property

Erie Police Department
5114-3
1000 Telleen Ave
Erie CO 80516-8418
Chris Holland
Print Date: 2025-02-25

Conducted by: Brent Dishar

Arapahoe Fire Protection
Arapahoe Fire Protection
11901 E. 14th
Ave CO 80010
303-366-4905



Orifice Coefficient	1420	Flow	1420
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Static Hydrant

Static Pressure	63	Residual Pressure	55
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Final Hydrant Flow Test

Flow at 20 psi residual pressure:	N/A	Total Flow	N/A
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Report of Inspection / Test for Asset - 2 Hydrant

Fire Hydrant Information

Location	North west corner behind building
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QUESTIONS

Is there a hydrant wrench that is available and accessible?	☐ Yes ☐ No ☒ NA	Is the hydrant free from cracks or leaks at outlets and on the top?	☒ Yes ☐ No ☐ NA
Are pumper and nozzle caps tight?	☐ Yes ☐ No ☒ NA	Is the hydrant properly painted and is the paint in good condition?	☒ Yes ☐ No ☐ NA
Does the operating nut turn with no difficulty?	☒ Yes ☐ No ☐ NA	Did the hydrant flow until clear (minimum of 1 minute)?	☒ Yes ☐ No ☐ NA
Are all dry barrels which require pumping identified?	☐ Yes ☐ No ☒ NA	Does the hydrant completely shut off?	☒ Yes ☐ No ☐ NA
Have the strainers been cleaned (if possible)?	☐ Yes ☐ No ☒ NA	Did monitor nozzle flowed acceptable water?	☒ Yes ☐ No ☐ NA
Have backflow devices, if installed, passed full flow test?	☐ Yes ☐ No ☒ NA	Is there no ice or water in the barrel?	☒ Yes ☐ No ☐ NA
Are hydrant caps, stems, outlets, and threads lubricated and in good condition?	☒ Yes ☐ No ☐ NA	Is Exposed piping properly secured and free from leaks or physical damage?	☒ Yes ☐ No ☐ NA
Is the Operating nut not worn, twisted or broken?	☒ Yes ☐ No ☐ NA	Is the Road box and shutoff valve visible and accessible?	☒ Yes ☐ No ☐ NA
Have dry barrels drained in at least 1 hour?	☒ Yes ☐ No ☐ NA	Have control valves been operated through complete range??	☒ Yes ☐ No ☐ NA
Are strainers, if installed, free from corrosion and not blocked?	☐ Yes ☐ No ☒ NA	Did monitor nozzles move through complete range?	☐ Yes ☐ No ☒ NA

Report of Inspection / Test

Annual NFPA 25

2025-02-24

Property

Erie Police Department
5114-3
1000 Telleen Ave
Erie CO 80516-8418
Chris Holland
Print Date: 2025-02-25

Conducted by: Brent Disher

Arapahoe Fire Protection
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Ave CO 80010
303-366-4905



Are monitor nozzles lubricated?

☐ Yes
☐ No
☒ NA

Hydrant Flow Test

Static Pressure	55	Residual Pressure	50
Pitot Pressure	56	Orifice Size	2.5
Orifice Coefficient	0.90	Flow	1396

Static Hydrant

Static Pressure	58	Residual Pressure	50
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Final Hydrant Flow Test

Flow at 20 psi residual pressure:	N/A	Total Flow	N/A
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Report of Inspection / Test

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2025-02-24

Property

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Questions with Photos and Notes

2 Hydrant - Is the hydrant properly painted and is the paint in good condition?

Yes

Notes:

Paint is faded due to sun



1 Hydrant - Is the hydrant properly painted and is the paint in good condition?

Yes

Notes:

Paint is starting to get faded due to the sun

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Deficiencies - 1 Hydrant

None

Deficiencies - 2 Hydrant

None

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Ave CO 80010
303-366-4905



Inspector Signature

I state that the information on this form is correct at the time and place of my inspection, and all equipment tested at this time was left in operational condition upon completion of this inspection except as noted.

Inspector Name

Brent Disher

Signature

A handwritten signature in black ink, appearing to read "Brent Disher", written over a light gray background.

Date Completed

2025-02-24

Client Signature

I state that the information on this form is correct at the time and place of my inspection, and all equipment tested at this time was left in operational condition upon completion of this inspection except as noted.

Client Name

Chris holland

Signature

A handwritten signature in black ink, appearing to read "Chris Holland", written over a light gray background.

Date Completed

2025-02-24

Exhibit C – WaterGEMS Modeling Outputs

FlexTable: Pipe Table

Label	Length (Scaled) (ft)	Start Node	Stop Node	Diameter (in)	Hazen-Williams C	Flow (gpm)	Velocity (ft/s)	Headloss Gradient (ft/ft)
Ex. Lateral 1	38	HYD 2	Ex. 12" x 6" Tee (N)	6.0	120.0	53	0.60	0.000
Ex. Lateral 2	24	HYD 1	Ex. 12" x 6" Tee (S)	6.0	120.0	67	0.76	0.001
Ex. Pipe 1	190	Ex. 12" x 6" Tee (N)	Ex. 12" x 8" Tee (N)	12.0	120.0	35	0.10	0.000
Ex. Pipe 2	423	Ex. 12" x 6" Tee (S)	Ex. 12" x 6" Tee (N)	12.0	120.0	2	0.01	0.000
Ex. Pipe 3	23	Ex. 12" x 6" Tee (S)	Ex. 12" x 8" Tee (S)	12.0	120.0	45	0.13	0.000
Prop. Pipe 1	282	Ex. 12" x 8" Tee (S)	Dom. Water Tap	8.0	120.0	25	0.16	0.000
Prop. Pipe 2	21	Dom. Water Tap	Fire Flow Tap	8.0	120.0	5	0.03	0.000
Prop. Pipe 3	656	Ex. 12" x 8" Tee (N)	Fire Flow Tap	8.0	120.0	15	0.09	0.000

FlexTable: Junction Table

Label	Elevation (ft)	Zone	Demand Collection	Demand (gpm)	Hydraulic Grade (ft)	Pressure (psi)
Ex. 12" x 6" Tee (N)	5,050.00	Zone - 1	<Collection: 1 items>	20	5,176.99	55
Ex. 12" x 6" Tee (S)	5,053.00	Zone - 1	<Collection: 1 items>	20	5,176.99	54
Ex. 12" x 8" Tee (S)	5,054.00	Zone - 1	<Collection: 1 items>	20	5,176.99	53
Ex. 12" x 8" Tee (N)	5,055.00	Zone - 1	<Collection: 1 items>	20	5,176.98	53
Dom. Water Tap	5,057.00	Zone - 1	<Collection: 1 items>	20	5,176.98	52
Fire Flow Tap	5,057.00	Zone - 1	<Collection: 1 items>	20	5,176.98	52



Base Scenario - Hydrant Calibration

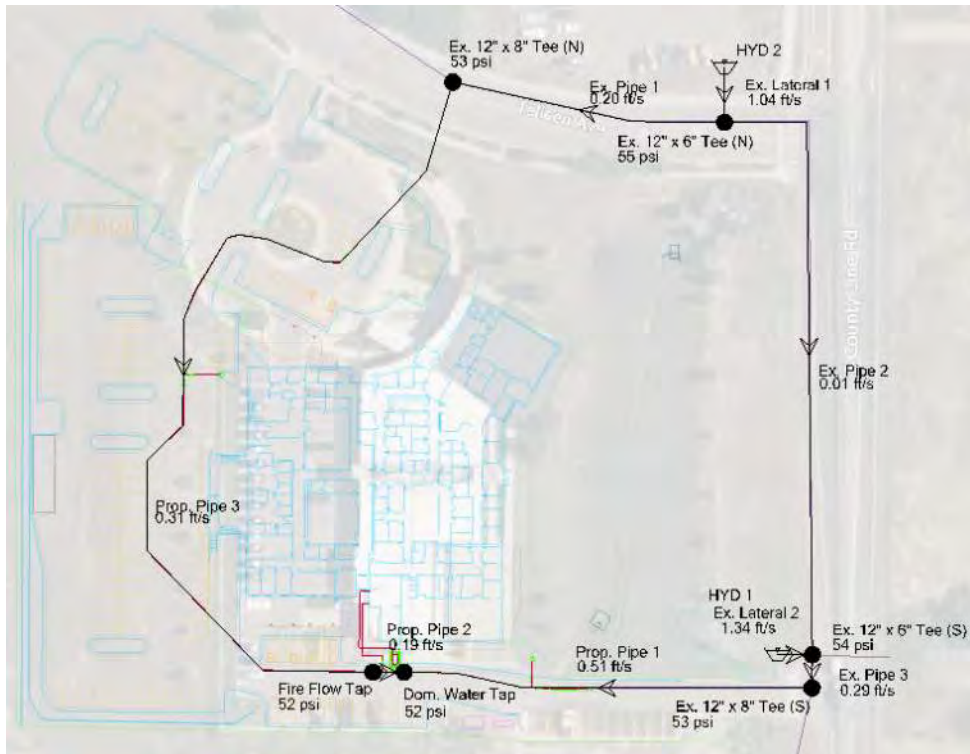
20 GPM Demand at all nodes shown

FlexTable: Pipe Table

Label	Length (Scaled) (ft)	Start Node	Stop Node	Diameter (in)	Hazen-Williams C	Flow (gpm)	Velocity (ft/s)	Headloss Gradient (ft/ft)
Ex. Lateral 1	38	HYD 2	Ex. 12" x 6" Tee (N)	6.0	120.0	92	1.04	0.001
Ex. Lateral 2	24	HYD 1	Ex. 12" x 6" Tee (S)	6.0	120.0	118	1.34	0.002
Ex. Pipe 1	190	Ex. 12" x 6" Tee (N)	Ex. 12" x 8" Tee (N)	12.0	120.0	69	0.20	0.000
Ex. Pipe 2	423	Ex. 12" x 6" Tee (N)	Ex. 12" x 6" Tee (S)	12.0	120.0	3	0.01	0.000
Ex. Pipe 3	23	Ex. 12" x 6" Tee (S)	Ex. 12" x 8" Tee (S)	12.0	120.0	101	0.29	0.000
Prop. Pipe 1	282	Ex. 12" x 8" Tee (S)	Dom. Water Tap	8.0	120.0	81	0.51	0.000
Prop. Pipe 2	21	Fire Flow Tap	Dom. Water Tap	8.0	120.0	29	0.19	0.000
Prop. Pipe 3	656	Ex. 12" x 8" Tee (N)	Fire Flow Tap	8.0	120.0	49	0.31	0.000

FlexTable: Junction Table

Label	Elevation (ft)	Zone	Demand Collection	Demand (gpm)	Hydraulic Grade (ft)	Pressure (psi)
Ex. 12" x 6" Tee (N)	5,050.00	Zone - 1	<Collection: 1 items>	20	5,176.96	55
Ex. 12" x 6" Tee (S)	5,053.00	Zone - 1	<Collection: 1 items>	20	5,176.96	54
Ex. 12" x 8" Tee (S)	5,054.00	Zone - 1	<Collection: 1 items>	20	5,176.96	53
Ex. 12" x 8" Tee (N)	5,055.00	Zone - 1	<Collection: 1 items>	20	5,176.96	53
Dom. Water Tap	5,057.00	Zone - 1	<Collection: 1 items>	110	5,176.90	52
Fire Flow Tap	5,057.00	Zone - 1	<Collection: 1 items>	20	5,176.90	52



Second Scenario - Peak Domestic Demand

110 GPM at domestic water tap node, 20 gpm at all other nodes

Note: Laterals used in model to calibrate site specific pressures within existing water system. Hydrant laterals do not actually experience significant velocity during a fire flow demand event.

FlexTable: Pipe Table

Label	Length (Scaled) (ft)	Start Node	Stop Node	Diameter (in)	Hazen-Williams C	Flow (gpm)	Velocity (ft/s)	Headloss Gradient (ft/ft)
Ex. Lateral 1	38	HYD 2	Ex. 12" x 6" Tee (N)	6.0	120.0	1,083	12.29	0.100
Ex. Lateral 2	24	HYD 1	Ex. 12" x 6" Tee (S)	6.0	120.0	1,392	15.79	0.159
Ex. Pipe 1	190	Ex. 12" x 6" Tee (N)	Ex. 12" x 8" Tee (N)	12.0	120.0	962	2.73	0.003
Ex. Pipe 2	423	Ex. 12" x 6" Tee (N)	Ex. 12" x 6" Tee (S)	12.0	120.0	101	0.29	0.000
Ex. Pipe 3	23	Ex. 12" x 6" Tee (S)	Ex. 12" x 8" Tee (S)	12.0	120.0	1,473	4.18	0.006
Prop. Pipe 1	282	Ex. 12" x 8" Tee (S)	Dom. Water Tap	8.0	120.0	1,453	9.27	0.042
Prop. Pipe 2	21	Dom. Water Tap	Fire Flow Tap	8.0	120.0	1,433	9.14	0.041
Prop. Pipe 3	656	Ex. 12" x 8" Tee (N)	Fire Flow Tap	8.0	120.0	942	6.01	0.019

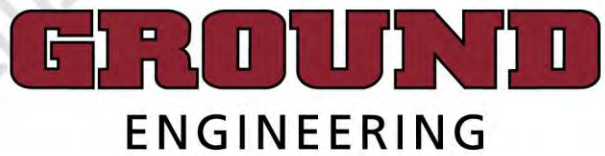
FlexTable: Junction Table

Label	Elevation (ft)	Zone	Demand Collection	Demand (gpm)	Hydraulic Grade (ft)	Pressure (psi)
Ex. 12" x 6" Tee (N)	5,050.00	Zone - 1	<Collection: 1 items>	20	5,173.21	53
Ex. 12" x 6" Tee (S)	5,053.00	Zone - 1	<Collection: 1 items>	20	5,173.20	52
Ex. 12" x 8" Tee (S)	5,054.00	Zone - 1	<Collection: 1 items>	20	5,173.06	52
Ex. 12" x 8" Tee (N)	5,055.00	Zone - 1	<Collection: 1 items>	20	5,172.69	51
Dom. Water Tap	5,057.00	Zone - 1	<Collection: 1 items>	20	5,161.13	45
Fire Flow Tap	5,057.00	Zone - 1	<Collection: 1 items>	2,375	5,160.24	45

Third Scenario - Fire Flow Demand

Peak fire flow demand of 2,375 gpm reflected at Fire Flow Tap node. All other nodes remain at base flow condition of 20 gpm





**Geotechnical Subsurface Exploration Program
Erie Police Station Addition
Erie, Colorado**

**Prepared For:
Town of Erie Public Works
150 Bonnell / Lambert Avenue
Erie, Colorado 80516**

Attention: Chad Alexander

Job Number: 25-0003

February 21, 2025

Revised March 13, 2025

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PURPOSE AND SCOPE OF STUDY

This report presents the results of a geotechnical evaluation performed by GROUND Engineering Consultants, Inc. (GROUND) for the Town of Erie Public Works in support of the proposed addition to the Erie Police Station facility at 1000 Talleen Avenue in Erie Colorado. Our study was conducted in general accordance with GROUND's Proposal No. 2412-2442.rev1, dated January 10, 2025

A field exploration program was conducted to obtain information on the subsurface conditions. Material samples obtained during the subsurface exploration were tested in the laboratory to provide data on the engineering characteristics of the on-site soils. The results of the field exploration and laboratory testing are presented herein.

This report has been prepared to summarize the data obtained and to present our findings and conclusions based on the proposed development/improvements and the subsurface conditions encountered. Design parameters and a discussion of engineering considerations related to the proposed improvements are included herein. This report should be understood and utilized in its entirety; specific sections of the text, drawings, graphs, tables, and other information contained within this report are intended to be understood in the context of the entire report. This includes the *Closure* section of the report which outlines important limitations on the information contained herein.

This report was prepared for the design purposes of the Town of Erie Public Works department based on our understanding of the proposed project at the time of preparation of this report. The data, conclusions, opinions, and geotechnical parameters provided herein should not be construed to be sufficient for other purposes, including the use by contractors, or any other parties for any reason not specifically related to the design of the project. Furthermore, the information provided in this report was based on the exploration and testing methods described below. Deviations between what was reported herein and the actual surface and/or subsurface conditions may exist, and in some cases those deviations may be significant.

PROPOSED CONSTRUCTION

Based on a provided conceptual site plan, we understand that proposed construction will consist additions to the west and southeast side of the existing police station. The west

**Erie Police Station Addition
Erie, Colorado**

addition is planned to consist of a two-story and be approximately 22,000 square feet in footprint area. The southeast addition is proposed to be one-story and be approximately 2,300 square feet in area. Additionally, a retaining wall is proposed to be construction on the west portion of the site. The height of this wall is unknown at the time of this report.

Specific loading was not available at the time of this study; however, we anticipate moderate to heavy loading consistent with structures of this size and configuration. The project area and test hole locations are shown on Figure 1. Proposed grading plans were not available at the time of this study, however based our understanding of the project site and a client provided existing site survey we anticipate cuts and fills on the order of 3 feet may be necessary to facilitate construction near the existing building with greater cuts anticipated on the west side of the existing parking lot up to approximately 12 feet.

Additionally, it is our understanding that the existing Police Station was constructed on a shallow foundation with a slab-on-grade floor system constructed on a system of aggregate piers utilized to improve the native site subgrade materials.

If our described understanding/interpretation of the proposed project is incorrect or project elements differ in any way from that expressed above, including changes to improvement locations, dimensions, orientations, loading conditions, elevations/grades, etc., and/or additional buildings/structures/site improvements are incorporated into this project, either after the original information was provided to us or after the date of this report, GROUND or another geotechnical engineer must be retained to re-evaluate the conclusions and parameters presented herein.

SITE CONDITIONS

At the time of our subsurface exploration program, the site supported the existing Erie Police Station. The east portion of the lot contains a detention pond, the existing building sits in the central portion of the site with parking situated to the south and west of the building. The west portion of the site is undeveloped with a local hill west of the parking lot. The hill is approximately 7 feet tall and based on grading plans from the 2014 construction as well as current grading plans provided by the owner was placed following the construction of the original police station. The site is bordered by Telleen Avenue to the north, East County Line Road to the east, a gravel drive to the south, and residential housing to the west.

The developed portion of the site is generally flat with the undeveloped western portion of the site rising on the order of 3 to 4 percent.

Man-made fill was observed in the test holes to depths ranging from approximately 3 to 8 feet below existing grades. Some of these fills were placed during the initial construction of the Erie Police Station. The exact extents, limits, and composition of any man-made fill was not determined as part of the scope of work addressed by this study and should be expected to exist at varying locations and depth throughout the site associated with previous uses.

SUBSURFACE EXPLORATION

The subsurface exploration for the project was conducted on January 27th and February 4th, 2025. A total of nine (9) test holes were drilled with a truck-mounted and a buggy-mounted, continuous flight power auger rig to evaluate the subsurface conditions as well as to retrieve soil samples for laboratory testing and analysis. Seven (7) foundation test holes were drilled near the proposed location for the new building to depths of approximately 30 to 36 feet below existing grades. Two (2) test holes were drilled for the proposed parking areas and drive lanes to depths of approximately 5 to 10 feet below existing grades. A representative of GROUND directed the subsurface exploration, logged the test holes in the field, and prepared the soil and bedrock samples for transport to our laboratory.

Samples of the subsurface materials were retrieved with a 2-inch I.D. 'California' liner sampler or a split spoon sampler. The sampler was driven into the substrata with blows from a 140-pound hammer falling 30 inches, a procedure similar to the Standard Penetration Test described by ASTM Method D1586. Penetration resistance values, when properly evaluated, indicate the relative density or consistency of soils. Depth and elevations at which the samples were obtained and associated penetration resistance values are shown on the test hole logs.

The approximate locations of the test holes are shown in Figure 1. Logs of the exploratory test holes are presented in Figure 2 and Appendix A. Explanatory notes and a legend are provided in Figure 3.

LABORATORY TESTING

Samples retrieved from our test holes were examined and visually classified in the laboratory by the project engineer. Laboratory testing of soil samples obtained from the subject site included standard property tests, such as natural moisture contents, grain size analyses, liquid and plastic limits, swell/consolidation testing, and soil corrosivity. Laboratory tests were performed in general accordance with applicable ASTM protocols. Results of the laboratory testing program are summarized on Tables 1 and 2.

GEOLOGIC SETTING

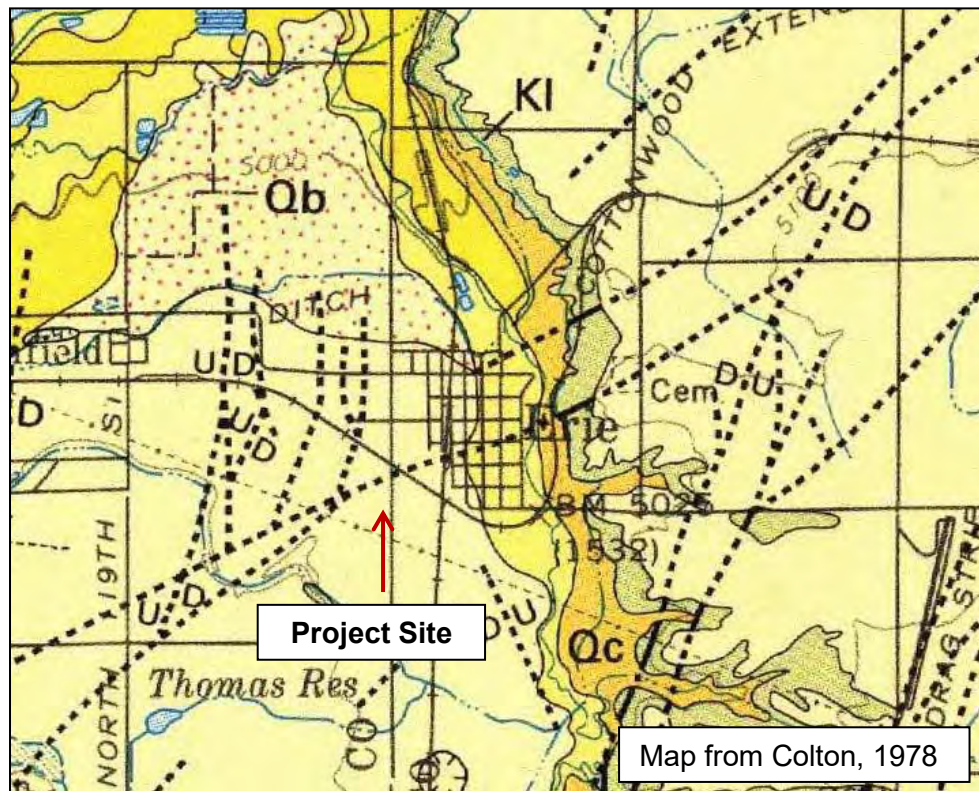
The subject parcel lies within the Denver Basin geologic province that consists largely of a sequence of sedimentary rock formations deposited and preserved in a structural depression in north-central Colorado. In the general project area, these sedimentary rocks dip eastward at low angles (less than 10 degrees, typically) and are overlain by a variety of surficial deposits including alluvial (stream-laid) sediments, eolian (wind-blown) materials, and colluvial (slope-wash) deposits.

Published maps depict the site as underlain by Upper Pleistocene to Holocene Loess (e.g., Trimble and Machette, 1979¹ and Colton 1978²). The surficial materials are mapped as underlain by deposits of the Upper Cretaceous, coal-bearing Laramie Formation and the Upper Cretaceous Fox Hills Sandstone (Trimble and Machette, 1979).

¹ Trimble, D.E. and M.N. Machette, 1979, *Geologic Map of the Greater Denver Area, Front Range Urban Corridor, Colorado*, U.S. Geological Survey, Miscellaneous Investigations Series, Map I-856-H.

² Colton, R.B., 1978, *Geologic map of the Boulder-Fort Collins-Greeley area, Front Range Urban Corridor, Colorado: U.S. Geological Survey, Miscellaneous Investigations Series Map I-855-G, scale 1:100000.*

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Loess (wind-blown silt) consists of eolian deposits of silts with sands and clays. Feldspars and micas in the original silts and clays commonly have weathered to clays, as well. Because of their mode of origin, loess deposits can be vulnerable to hydro-compaction. The native sands and clays encountered in the test holes are interpreted to be severely weathered loess.

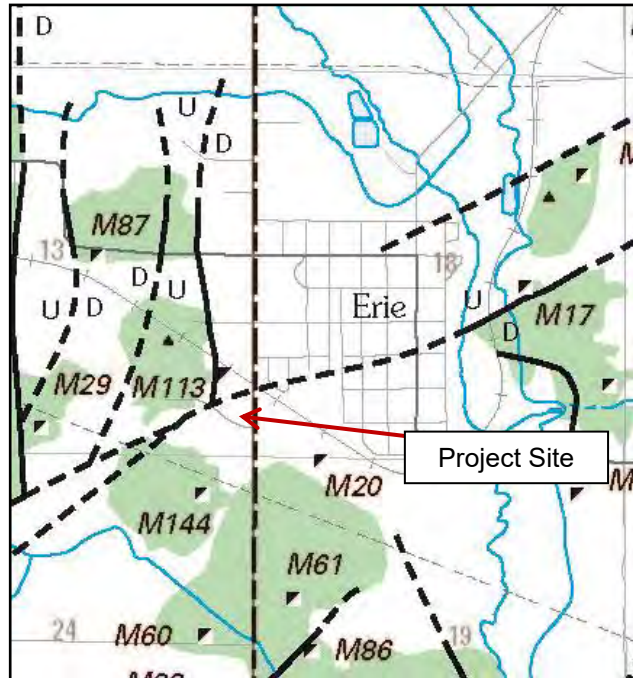
In the Front Range area, the Laramie Formation consists primarily of clay shales and claystones, with subordinate deposits of siltstones and sandstones. Coal beds of up to 6 or more feet in thickness are common, as well. The clay shales and claystones are moderately to highly expansive. Coal has been mined from the Laramie Formation in the Broomfield, Louisville, Lafayette, Erie and Firestone areas, and subsidence or collapse has taken place and may continue to occur over some of these mined areas.

The Fox Hills Sandstone consists largely of sandstones, interbedded locally with claystones. Well-cemented, resistant sandstones are encountered, as well as over-consolidated, but friable sandstones. The claystones typically are moderately to highly expansive. The bedrock encountered in the test holes are interpreted to be Fox Hills Formation Sandstone materials.

Several faults are depicted on geologic maps of the project vicinity. Those are considered inactive faults, identified during the course of mining operations.

GEOLOGIC HAZARDS

Mining Activity and Subsidence Review of U.S. Geological Survey topographic maps covering the site (e.g., U.S.G.S. 1967, revised 1979³) and other available, published maps depicting areas of coal extraction by Roberts and others (2001⁴), Amuedo and Ivey (1975⁵), and Jones and others (1978⁶), did not indicate past mining activities on or adjacent (closer than ~500 feet) to the subject parcel. The two closest documented coal mine workings were the Northrup and Stewart mines and were located approximately 500 feet to the northwest and southwest, respectively (Roberts, et al., 1975). No indications of mining activities were apparent on the site during the site reconnaissance. Therefore, there appears to be little potential for surface subsidence associated with consolidation of former mine workings at depth based on our review of these documents.



Hydro-compactive or 'Collapsible' Soils Certain surficial deposits in the Denver metropolitan area, typically eolian (wind-blown) materials including loess, are known to be susceptible to local hydro-compaction or "collapse." Hydro-compaction consists of a significant volume loss due to

³ U.S. Geological Survey, 1967 (revised 1979) 7.5 Minute Series (Topographic), Erie Quadrangle, Colorado.

⁴ Roberts, S.B., J.L. Hynes, and C.L. Woodward, 2001, *Maps Showing The Extent of Mining, Locations of Mine Shafts, Adits, Air Shafts, and Bedrock Faults, and Thickness of Overburden Above Abandoned Coal Mines in the Boulder-Weld Coal Field, Boulder, Weld, and Adams Counties, Colorado*, United States Geological Survey, Geologic Investigations Series I-2735.

⁵ Amuedo and Ivey, 1975, *Coal Mine Subsidence and Land Use in the Boulder-Weld Coal Field, Boulder and Weld Counties, Colorado*, Colorado Geological Survey, Environmental Geology No. 9, 88 pages.

⁶ Jones, D.C., J.E. Schultz and D.K. Murray, 1978, *Coal Resources and Development Map of Colorado*, Colorado Geological Survey, Map Series 9.

re-structuring of the constituent grains of the soil to a more compact arrangement upon wetting under a surcharge load.

The soils extending to depths of about 24 to 26 feet on the site are interpreted to be loess deposits and a relatively severe potential for hydro-compaction was indicated by laboratory testing of those soils (see Table 1). As discussed in the *Geotechnical Considerations for Design* section of this report, structures supported directly on these soils would be vulnerable to relatively severe post-construction movements. We recommend supporting the building and other improvements not tolerant of significant post-construction movements on deep foundations or aggregate pier systems. More than typical, on-going maintenance should be anticipated for pavements and other improvements supported at shallow depths.

Expansive Soils Heave Swelling clayey soils and bedrock change volume in response to changes in moisture content that can occur seasonally, or in response to changes in land use, including development. Expansion potentials vary with moisture contents, density and details of the clay chemistry and mineralogy. The swell potential in any particular area can vary markedly both laterally and vertically due to the complex interbedding of the site soil and bedrock materials. Moisture changes also occur erratically, resulting in conditions that cannot always be predicted.

SUBSURFACE CONDITIONS

In general, the test holes penetrated a thin layer of surficial materials consisting of surficial materials consisting of topsoil⁷ or asphalt, approximately 3 to 7 inches in thickness. Below the surficial materials we observed man-made fill consisting of silty/clayey sand that continued to depths ranging from 3 to 8 feet below existing grades. The fill materials were generally underlain by native silty/clayey sands materials that continued to depths ranging from 24 to 30 feet below existing grades. Sandstone bedrock materials were generally encountered below the overburden sands and continued to the test hole termination depths of 30 to 36 feet below existing grades. Weathered sandstone and claystone layers were noted above the sandstone bedrock locally in test holes 3 and 5.

Please see the attached drill logs in Figure 2 and Appendix A for individual soil profiles.

⁷ 'Topsoil' as used herein is defined geotechnically. The materials so described may or may not be suitable for landscaping or as a growth medium for such plantings as may be proposed for the project.

Fill materials consisted of silty / clayey sand and were low to non-plastic, fine to medium grained, dry to slightly moist, and brown in color with localized caliche.

Sand ranged from silty to clayey, were fine to medium grained, low to non- plastic, dry to slightly moist, loose to dense, and brown in color.

Sandy Clay was fine to medium grained, low to medium plastic, moist to very moist, stiff to very stiff, and brown with local iron staining.

Weathered Claystone / Sandstone was interbedded, fine grained, non to highly plastic, moist, and brown in color.

Weathered Sandstone materials were non-plastic, fine grained, slightly moist, and brown in color with iron staining.

Sandstone Bedrock materials were non-plastic, fine grained, slightly moist, hard to very hard, and orange-brown to gray-brown in color with iron staining.

Groundwater was encountered locally in test hole 2 at a depth of 32 feet below existing grades. Test hole 5 was completed as a groundwater monitoring hole and was checked approximately 14 days later and was dry to a depth of approximately 30 feet below existing grade. Groundwater levels can be expected to fluctuate, however, in response to annual and longer-term cycles of precipitation, irrigation, surface drainage, land use, and the development of transient, perched water conditions.

Swell-Consolidation Testing yielded results ranging from 1.6 percent swell to 3.8 percent consolidation upon wetting under surcharge loads approximately equal to the anticipated overburden pressures (Table 1).

SEISMIC CLASSIFICATION

Based on extrapolation of available data to depth and our experience in the project area, we consider the area of the proposed building likely to meet the criteria for a Seismic Site Classification of **D** according to the ASCE 7-16 (Table 20.3-1). (Exploration and/or shear wave velocity testing to a depth of 100 feet or more was not part of our present scope of services.) If, however, a quantitative assessment of the site seismic properties is desired, to obtain a Seismic Site Classification of C, for example, then shear wave velocity testing

should be performed. GROUND can provide a fee estimate for shear wave velocity testing upon request. We consider the likelihood of obtaining a Site Classification of C to be low.

Using longitude and latitude coordinates obtained from Google Earth and the ASCE Hazard Tool (ascehazardtool.org) the project area is indicated to possess an S_{DS} value of **0.217** and an S_{D1} value of **0.092** for the site latitude and longitude and a Site Class of D.

GEOTECHNICAL CONSIDERATIONS FOR DESIGN

The conclusions and parameters provided in this report were based on the data presented herein, our experience in the general project area with similar structures, and our engineering judgment with regard to the applicability of the data and methods of forecasting future performance. A variety of engineering parameters were considered as indicators of potential future soil movements.

Our parameters and conclusions were based on our judgment of “likely movement potentials,” (i.e., the amount of movement likely to be realized if site drainage is generally effective, estimated to a reasonable degree of engineering certainty) as well as our assumptions about the owner’s willingness to accept geotechnical risk. “Maximum possible” movement estimates necessarily will be larger than those presented herein. They also have a significantly lower likelihood of being realized, in our opinion, and generally require more expensive measures to address.

We encourage the Town of Erie Public Works department upon receipt of this report, to discuss these risks and the geotechnical alternatives with us.

The native overburden materials encountered at the project site extending to depths ranging from 24 to 30 feet below existing grades consisted primarily of silty / clayey sands. Based on laboratory data, these materials exhibited a variable potential for hydro-collapse up to approximately 3.8 percent.

Undocumented fills were also noted locally within test holes 3, 6, and 7 ranging up to approximately 8 feet in thickness. It is unknown if these materials were placed in a properly moisture-density treated manner, and in the absence of testing and/or fill placement documentation, it should be assumed that they were not until documentation stating otherwise is produced. In undocumented fill soils, there exists a largely unquantifiable risk of volume change of the fill (primarily from consolidation of materials) associated with the

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presence of unknown materials and voids in the fill. The fill materials noted in the remainder of the test holes were tested and documented by GROUND Engineering during the 2014 construction of the Erie Police Station facility.

Based on the presence of the collapsible subgrade materials GROUND estimates that shallow foundations placed directly on the on-site materials could experience 2 to 4 inches, or more, of potential post-construction settlement locally with similar differential movements over spans of about 40 feet.

To help reduce the potential of post-construction movements several remedial options are presented below.

Building Foundation and Floor Types In GROUND's opinion, supporting the proposed buildings on drilled pier foundation system will provide the lowest estimates of likely post-construction foundation movement (about ½ inch, with similar differential movements over spans of about 40 feet) and will provide the least risk of excessive foundation movements. However, deep foundation systems may not be necessary to carry the structural loads for the proposed facility and bedrock was not encountered until depths on the order of 24 to 30 feet below existing grades. Drilled piers on the order of 40 feet or more in length would likely be necessary to support the facility. We anticipate that piers of this length may not be the most cost-effective foundation option. However, geotechnical parameters for drilled piers foundations can be provided upon request, however.

Constructing the building floors as structural floors, also supported on drilled piers, will yield similarly low post-construction floor movement estimates. Exterior flatwork adjacent to the building, particularly at and near building entrances could be constructed as structural floors. Geotechnical parameters for structural floors can be provided upon request.

The existing facility was previously constructed on a system of aggregate piers in the presence of collapsible overburden materials and undocumented fills. Therefore, as an economically feasible alternative, shallow foundations and slab-on-grade floor systems appear to be geotechnically feasible at this site, provided that they bare on soils improved by aggregate piers or similar ground improvement techniques. Aggregate piers, "geopiers," "vibro-piers," etc., are proprietary foundation systems and the design parameters will be provided by the specialty designer/installer. Such systems will provide

sufficient bearing capacity without the risk of excessive settlement. We suggest contacting either of the following designer/installers, although others may be available:

- Ground Improvement Engineering 816 / 421 – 4334
- Keller (formerly Hayward Baker) 303 / 469 – 1136

The data in this report should be sufficient for the designer/installed to provide their design, and additional parameters for shallow foundations can be provided upon request.

In GROUND's experience plumbing trenches commonly can impact aggregate piers. The location of planned utility trenches should be coordinated with the aggregate pier designer to limit the impact on aggregate piers.

SHALLOW FOUNDATIONS

The geotechnical parameters below may be used for design of foundations for the proposed building.

Geotechnical Parameters for Shallow Foundation Design w/ rammed aggregate piers

- 1) Footings should bear on a section of soils improved by rammed aggregate piers as discussed in the *Geotechnical Considerations for Design* section of this report.
- 2) Footings bearing on soils improved by rammed aggregate piers are used, the allowable bearing pressure provided by the rammed aggregate pier designer/installer should be used.

To reduce differential settlements between footings or along continuous footings, footing loads should be as uniform as possible. Differentially loaded footings will settle differentially.

- 3) Spread footings should have a minimum lateral dimension of **16 or more inches** for linear strip footings and **24 or more inches** for isolated pad footings. Actual footing dimensions should be determined by the structural engineer.
- 4) Footings should bear at an elevation **3 or more feet** below the lowest adjacent exterior finish grades to have adequate soil cover for frost protection.

- 5) Continuous foundation walls should be reinforced as designed by a structural engineer to span an unsupported length of at least **10 feet**.
- 6) Geotechnical parameters for lateral resistance to foundation loads are provided in the *Lateral Loads* section of this report.
- 7) Connections of all types must be flexible and/or adjustable to accommodate the anticipated, post-construction movements of the structure.

Shallow Foundation Construction

- 8) The contractor should take adequate care when making excavations not to compromise the bearing or lateral support of installed rammed aggregate piers.
- 9) Care should be taken when excavating the foundations to avoid disturbing the supporting aggregate pier materials particularly in excavating the last few inches.
- 10) Foundation-supporting soils may be disturbed or deform excessively under the wheel loads of heavy construction vehicles as the excavations approach footing bearing levels. Construction equipment should be as light as possible to limit development of this condition. The movement of vehicles over proposed foundation areas should be restricted.
- 11) If aggregate piers are disturbed during excavation, the aggregate pier designer/installer should be contacted to determine how to address.
- 12) Fill placed against the sides of the footings should be properly compacted in accordance with the *Project Earthwork* section of this report.

SLAB-ON-GRADE FLOORS

The geotechnical parameters below may be used for design of slab-on-grade floors for the proposed buildings. ACI Sections 301/302/360 provide guidance regarding concrete slab-on-grade design and construction.

Geotechnical Parameters for Design of Slab-on-Grade Floors

- 1) A slab-on-grade floor system should bear on at a section of properly moisture-density treated and compacted CDOT Class 5 or 6 aggregate base course materials placed over a layer of Mirafi® BXG 110 or equivalent. This section should extend from the bottom of the slab to the top of the rammed aggregate piers.
- 2) A minimum of 12 inches of base course, noted above, should be placed over the BXG 110 and rammed aggregate piers as discussed in the *Geotechnical Considerations for Design* section of this report and designed by others.
- 3) Floor slabs should be adequately reinforced. Floor slab design, including slab thickness, concrete strength, jointing, and slab reinforcement should be developed by a structural engineer.
- 4) An allowable vertical modulus of subgrade reaction (**Kv**) of **250 pci** may be used for design of a concrete, slab-on-grade floor bearing on a remedial fill section consisting of site CDOT Class 5/6 Aggregate Base fill materials.

These values are for a 1-foot x 1-foot plate; they should be adjusted for slab dimension.

- 5) Floor slabs should be separated from all bearing walls and columns with slip joints, which allow unrestrained vertical movement.

Slip joints should be observed periodically, particularly during the first several years after construction. Slab movement can cause previously free-slipping joints to bind. Measures should be taken to assure that slab isolation is maintained in order to reduce the likelihood of damage to walls and other interior improvements.

However, where estimates of post-construction movement is sufficiently low and the associated risks are understood and accepted by the owner and the design team, slip joint may be omitted to allow for the use a thickened slab foundation elements.

- 6) Concrete slabs-on-grade should be provided with properly designed control joints.

ACI, AASHTO, and other industry groups provide guidelines for proper design and construction concrete slabs-on-grade and associated jointing. The design and construction of such joints should account for cracking as a result of shrinkage, curling, tension, loading, and curing, as well as proposed slab use. Joint layout based on the slab design may require more frequent, additional, or deeper joints, and should reflect the configuration and proposed use of the slab.

Particular attention in slab joint layout should be paid to areas where slabs consist of interior corners or curves (e.g., at column blockouts or reentrant corners) or where slabs have high length to width ratios, significant slopes, thickness transitions, high traffic loads, or other unique features. Improper placement or construction will increase the potential for slab cracking.

- 7) Interior partitions resting on floor slabs should be provided with slip joints so that if the slabs move, the movement cannot be transmitted to the upper structure. This detail is also important for wallboards and doorframes. Slip joints should allow **1½ inches or more** of vertical, differential movement. Accommodation for differential movement also should be made where partitions meet bearing walls.
- 8) Post-construction movements may not displace slab-on-grade floors and utility lines in the soils beneath them to the same extent. Design of floor penetrations, connections, and fixtures should accommodate **up to 2 inches** of differential movement.
- 9) A vapor barrier beneath a building floor slab can be beneficial with regard to reducing exterior moisture moving into the building, through the slab, but can retard downward drainage of construction moisture. Uneven moisture release can result in slab curling. Elevated vapor fluxes can be detrimental to the adhesion and performance of many floor coverings and may exceed various flooring manufacturers' usage criteria.

Per the 2006 ACI *Location Guideline*, a vapor barrier is required under concrete floors when that floor is to receive moisture-sensitive floor covering and/or adhesives, or the room above that floor has humidity control.

Therefore, in light of the several, potentially conflicting effects of the use vapor-barriers, the owner and the architect and/or contractor should weigh the performance of the slab and appropriate flooring products in light of the intended building use, etc., during the floor system design process and the selection of flooring materials. Use of a plastic vapor-barrier membrane may be appropriate for some building areas and not for others.

In the event a vapor barrier is utilized, it should consist of a minimum 15 mil thickness, extruded polyolefin plastic (no recycled content or woven materials), maintain a permeance less than 0.01 perms per ASTM E-96 or ASTM F-1249, and comply with ASTM E-1745 (Class "A"). Vapor barriers should be installed in accordance with ASTM E-1643.

Polyethylene ("poly") sheeting (even if 15 mils in thickness which polyethylene sheeting commonly is not) does not meet the ASTM E-1745 criteria and should not be used as vapor barrier material. It can be easily torn and/or punctured, does not possess necessary tensile strength, gets brittle, tends to decompose over time, and has a relatively high permeance.

Construction Considerations for Slab-on-Grade Floors

- 10) Loose, soft, or otherwise unsuitable materials exposed on the prepared surface on which the floor slab will be cast should be excavated and replaced with properly compacted fill. Care should be taken not to disturb the rammed aggregate piers or subgrade improvements.
- 11) Concrete floor slabs should be constructed and cured in accordance with applicable industry standards and slab design specifications.
- 12) All plumbing lines should be carefully tested before operation. Where plumbing lines enter through the floor, a positive bond break should be provided.

RETAINING WALLS

Wall Foundation Undocumented fill materials were noted in the proposed location of the retaining wall that extended to 5 and 7 feet below existing grades. Additionally, our site investigation indicated that the existing subgrade materials have a collapse potential up

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to approximately 3.8 percent. Based on the presence of the collapsable subgrade materials GROUND estimates that shallow foundations placed directly on the on-site materials could experience 2 to 4 inches, or more, of potential post-construction settlement locally with similar differential movements over spans of about 40 feet. Wall settlement/deformation can result in significant cracking and other damage. Therefore, due to the presence of the undocumented fills as well as the collapsable site soils, it is GROUND's opinion that the retaining wall should be construction on subgrade improved with an aggregate pier system similar to the addition to the Police Station addition.

Lateral Earth Pressures Lateral earth pressures for the onsite overburden sands and/or imported CDOT Class 1 Structure fill, that can be utilized in design of the proposed retaining wall, are provided in the *Lateral Earth Pressures* section of this report below.

Imported CDOT Class 1 Structure Fill (if used) Select, granular materials imported for use as CDOT Class 1 Structure Backfill should meet the criteria as tabulated below. All imported soils should be tested and approved by the Geotechnical Engineer prior to transport to the site.

CDOT CLASS 1 STRUCTURE BACKFILL

Sieve Size or Parameter	Acceptable Range
2-inch Sieve	100% passing
No. 4 Sieve	30% to 100% passing
No. 50 Sieve	10% to 60% passing
No. 200 Sieve	5% to 20% passing
Liquid Limit	$\leq 35 \%$
Plasticity Index	$\leq 6 \%$
Internal Friction Angle	≥ 34 degrees

GROUND recommends the use of structure backfill behind the walls to achieve lower lateral earth pressures. To realize the lower equivalent fluid unit weight, structure backfill should be placed behind the wall to a minimum distance equal or greater than half of the wall height. Where structure backfill is used, the upper 1 foot of the wall backfill should be

a relatively impermeable soil or otherwise protected to reduce surface water infiltration into the wall backfill.

The lateral resistance of retaining wall foundations placed on undisturbed natural soils and/or properly compacted fill material at the site will be combination of the sliding resistance of the footing on the foundation materials and the passive pressure against the sides of the footings. The upper foot of embedment should not be relied upon for passive resistance, however.

Wall Drainage Retaining walls should be provided with drains at the heels of the walls, or with weep holes, or both, to help reduce the development of hydrostatic loads. General parameters for these drains are provided in the *Subsurface Drainage* section of this report.

In addition to surrounding the drain pipes with at least 6 inches of free-draining gravel, the gravel should extend upward to within 12 inches of the backfill surface behind the wall or the wall should be backed with a layer of geocomposite drainage medium, e.g., an appropriate MiraDrain® product or equivalent. The gravel or drainage product backing the wall should be in hydraulic connection with the wall heel drain. If gravel is selected, it should be separated from the enclosing soils by a layer of filter fabric to reduce the migration of fines into the drainage system. Damp-proofing should be applied to the back side of rigid types of retaining walls.

Wall Construction Considerations Backfill soils should be thoroughly mixed to achieve uniform moisture content, placed in uniform lifts not exceeding 8 inches in loose thickness, and properly compacted to **95 or more percent** of the maximum Standard Proctor density at moisture contents within **2 percent** of the optimum moisture content as determined by ASTM D698. The Contractor should take care not to over-compact the backfills which could result in excessive lateral pressures on the walls. Backfill for

The Geotechnical Engineer should be retained to observe the exposed excavation prior to placement of backfill, observe earthwork operations and drain installation, and test the soils.

Some settlements of wall backfills will occur even where the material is placed correctly. This settlement will likely be differential, increasing with depth of fill. Shallowly founded structures and pavements should not be located on or adjacent to the backfilled zones.

Where improvements must be placed in the backfilled zones, structural design, pipe connections, etc., should take into account backfill settlements, including differential movement and the associated risks are understood by the Owner. The Geotechnical Engineer should be retained to provide additional parameters for founding of improvements in such areas.

LATERAL EARTH PRESSURES

The at-rest, active, and passive conditions for the on-site backfill are summarized on the table below. Base friction may be combined with passive earth pressure if the foundation is in a drained condition. The values for the on-site material in the upper 10 feet provided in the table below were approximated utilizing a unit weight of 118 pcf and an angle of internal friction (ϕ) of 28 degrees. Values for imported materials meeting the requirements of CDOT Class 1 structure fill are also approximated utilizing a unit weight of 125 and an angle of internal friction (ϕ) of 34 degrees.

Lateral Earth Pressures (Equivalent Fluid Unit Weights)

Material Type	Water Condition	At-Rest (pcf)	Active (pcf)	Passive(pcf)	Friction Coefficient
On-Site Backfill	Drained	63	43	285 (max. 2,850 psf)	0.35
CDOT Class 1	Drained	36	36	400 (max 4,000 psf)	0.45

The upper 1 foot of embedment should be neglected for passive resistance, however. Where this passive soil pressure is used to resist lateral loads, it should be understood that significant lateral strains will be required to mobilize the full value indicated above, likely 1 inch or more. A reduced passive pressure can be used for reduced anticipated strains, however.

The lateral earth pressures indicated above are for a horizontal upper backfill slope. The additional loading of an upward sloping backfill as well as loads from traffic, stockpiled materials, etc., should be included in the wall/shoring design. GROUND can provide the adjusted lateral earth pressures when the additional loading conditions and site grading are clearly defined.

WATER-SOLUBLE SULFATES

The concentrations of water-soluble sulfates measured in selected earth material samples retrieved from the test holes ranged up to 0.02 percent by weight (See Table 2). Such concentrations of soluble sulfates represent a *negligible* environment for sulfate attack on concrete exposed to these materials. Degrees of attack are based on the scale of 'negligible,' 'moderate,' 'severe' and 'very severe' as described in the "Design and Control of Concrete Mixtures," published by the Portland Cement Association (PCA). The Colorado Department of Transportation (CDOT) utilizes a corresponding scale with 4 classes of severity of sulfate exposure (Class 0 to Class 3) as described in the published table below.

**REQUIREMENTS TO PROTECT AGAINST DAMAGE TO
CONCRETE BY SULFATE ATTACK FROM EXTERNAL SOURCES OF SULFATE**

Severity of Sulfate Exposure	Water-Soluble Sulfate (SO₄) In Dry Soil (%)	Sulfate (SO₄) In Water (ppm)	Water Cementitious Ratio (maximum)	Cementitious Material Requirements
Class 0	0.00 to 0.10	0 to 150	0.45	Class 0
Class 1	0.11 to 0.20	151 to 1500	0.45	Class 1
Class 2	0.21 to 2.00	1501 to 10,000	0.45	Class 2
Class 3	2.01 or greater	10,001 or greater	0.40	Class 3

Based on our test results and PCA and CDOT guidelines, GROUND recommends the use of the appropriate cement in all concrete exposed to site soils, conforming to one of the following requirements:

Class 0 (Negligible)

- (1) ASTM C150 Type I, II, III, or V.
- (2) ASTM C595 Type IL, IP, IP(MS), IP(HS), or IT.

The contractor should be aware that certain concrete mix components affecting sulfate resistance including, but not limited to, the cement, entrained air, and fly ash, can affect workability, set time, and other characteristics during placement, finishing and curing. The contractor should develop mix(es) for use in project concrete which are suitable with regard to these construction factors, as well as sulfate resistance. A reduced, but still

significant, sulfate resistance may be acceptable to the owner, in exchange for desired construction characteristics.

SOIL CORROSIVITY

The degree of risk for corrosion of metals in soils commonly is considered to be in two categories: corrosion in undisturbed soils and corrosion in disturbed soils. The potential for corrosion in undisturbed soil is generally low, regardless of soil types and conditions, because it is limited by the amount of oxygen that is available to create an electrolytic cell. In disturbed soils, the potential for corrosion typically is higher, but is strongly affected by soil chemistry and other factors.

A preliminary corrosivity analysis was performed to provide a general assessment of the potential for corrosion of ferrous metals installed in contact with earth materials at the site, based on the conditions existing at the time of GROUND's evaluation. Soil chemistry and physical property data including pH, and sulfides content were obtained. Test results are summarized on Table 2.

Soil Resistivity In order to assess the "worst case" for mitigation planning, samples of materials retrieved from the test holes were tested for resistivity in the laboratory, after being saturated with water, rather than in the field. Resistivity also varies inversely with temperature. Therefore, the laboratory measurements were made at a controlled temperature.

Measurements of electrical resistivity indicated a value of approximately 4,000 ohm-centimeters in a sample of the site earth materials.

pH Where pH is less than 4.0, soil serves as an electrolyte; the pH range of about 6.5 to 7.5 indicates soil conditions that are optimum for sulfate reduction. In the pH range above 8.5, soils are generally high in dissolved salts, yielding a low soil resistivity (AWWA, 2010). Testing indicated a pH value of approximately 8.2.

Reduction-Oxidation testing indicated a negative potential: approximately -68 mV. Lower potentials typically create a more corrosive environment.

Sulfide Reactivity testing for the presence of sulfides indicated 'Trace' results. The presence of sulfides in the site soils also suggests a more corrosive environment.

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Corrosivity Assessment The American Water Works Association (AWWA, 2010⁸) has developed a point system scale used to predict corrosivity. The scale is intended for protection of ductile iron pipe but is valuable for project steel selection. When the scale equals 10 points or higher, protective measures for ductile iron pipe are suggested. The AWWA scale (Table A.1 Soil-test Evaluation) is presented below. The soil characteristics refer to the conditions at and above pipe installation depth.

Table A.1 Soil-test Evaluation

<u>Soil Characteristic / Value</u>	<u>Points</u>
Resistivity	
<1,500 ohm-cm	10
1,500 to 1,800 ohm-cm	8
1,800 to 2,100 ohm-cm	5
2,100 to 2,500 ohm-cm	2
2,500 to 3,000 ohm-cm	1
>3,000 ohm-cm	0
pH	
0 to 2.0	5
2.0 to 4.0	3
4.0 to 6.5	0
6.5 to 7.5	0 *
7.5 to 8.5	0
>8.5	3
Redox Potential	
< 0 (negative values)	5
0 to +50 mV	4
+50 to +100 mV	3½
> +100 mV	0
Sulfide Reactivity	
Positive	3½
Trace	2
Negative	0
Moisture	
Poor drainage, continuously wet	2
Fair drainage, generally moist	1
Good drainage, generally dry	0

* If sulfides are present and low or negative redox-potential results (< 50 mV) are obtained, add three points for this range.

⁸ American Water Works Association ANSI/AWWA C105/A21.5-05 Standard.

We anticipate that drainage at the site after construction will be effective. Therefore, based on the values obtained for the soil parameters, the overburden soils do not appear to comprise a highly corrosive environment for metals (7 points).

If additional information is needed regarding soil corrosivity, the American Water Works Association or a Corrosion Engineer should be contacted. It should be noted, however, that changes to the site conditions during construction, such as the import of other soils, or the intended or unintended introduction of off-site water, may significantly alter corrosion potential.

PROJECT EARTHWORK

The earthwork criteria below are based on our interpretation of the geotechnical conditions encountered in the test holes. Where these criteria differ from applicable municipal specifications, e.g., for trench backfill compaction along a public utility line, the latter should be considered to take precedence.

General Considerations Site grading should be performed as early as possible in the construction sequence to allow settlement of fills and surcharged ground to be realized to the greatest extent prior to subsequent construction.

Prior to earthwork construction, existing concrete, asphalt, vegetation, and other deleterious materials should be removed and disposed of off-site. Relic underground utilities should be abandoned in accordance with applicable regulations, removed as necessary, and properly capped.

Topsoil if present on-site should not be incorporated into ordinary fills. Instead, topsoil should be stockpiled during initial grading operations for placement in areas to be landscaped or for other approved uses.

Use of Existing Native Soils Overburden soils that are free of trash, organic material (including all firewood, wood chips, etc.), construction debris, and other deleterious materials are suitable, in general, for placement as compacted fill. Organic materials should not be incorporated into project fills.

Fragments of rock, cobbles, and inert construction debris (e.g., concrete or asphalt) larger than 3 inches in maximum dimension will require special handling and/or placement to be

incorporated into project fills. In general, such materials should be placed as deeply as possible in the project fills. A Geotechnical Engineer should be consulted regarding appropriate direction for usage of such materials on a case-by-case basis when such materials have been identified during earthwork. Standard parameters that likely will be generally applicable can be found in Section 203 of the current CDOT Standard Specifications for Road and Bridge Construction.

Imported Fill Materials If it is necessary to import material to the site the imported soils should be free of organic material, and other deleterious materials. **Imported material should consist of materials that have less than 35 percent passing the No. 200 Sieve and should have a plasticity index less than 15.** Representative samples of the materials proposed for import should be tested and approved prior to transport to the site.

Fill Platform Preparation Prior to filling, the top 12 inches of in-place materials on which fill soils will be placed should be scarified, moisture conditioned and properly compacted in accordance with the parameters below to provide a uniform base for fill placement. *If over-excavation is to be performed, then these parameters for subgrade preparation are for the subgrade **below the bottom** of the specified over-excavation depth.*

If surfaces to receive fill expose loose, wet, soft or otherwise deleterious material, additional material should be excavated, or other measures taken to establish a firm platform for filling. The surfaces to receive fill must be effectively stable prior to placement of fill.

GROUND's experience within the project area suggests the frost depth to be approximately 3 feet below ground surface.

Fill Placement Fill materials should be thoroughly mixed to achieve a uniform moisture content, placed in uniform lifts not exceeding 8 inches in loose thickness, and properly compacted.

Soils that classify as GP, GW, GM, GC, SP, SW, SM, or SC in accordance with the USCS classification system (granular materials) should be compacted to **95 or more percent** of the maximum Modified Proctor dry density at moisture contents within **2 percent** of optimum moisture content as determined by ASTM D1557.

Soils that classify as ML or CL should be compacted to **95 or more percent** of the maximum Standard Proctor density at moisture contents within **2 percent** of the optimum moisture content as determined by ASTM D698.

Soils that classify as CH (claystone) fine grained soils should be avoided for use under structures, flatwork, or pavements. GROUND Engineering should be contacted prior to using these materials for these potential uses.

It may be necessary to rework the fill materials more than once by adjusting moisture and replacing the materials, in order to achieve the recommended compaction and moisture criteria.

No fill materials should be placed, worked, or rolled while they are frozen, thawing, or during poor/inclement weather conditions.

Care should be taken with regard to achieving and maintaining proper moisture contents during placement and compaction. Materials that are not properly moisture conditioned may exhibit significant pumping, rutting, and deflection at moisture contents near optimum and above. The contractor should be prepared to handle soils of this type, including the use of chemical stabilization, if necessary.

Compaction areas should be kept separate, and no lift should be covered by another until relative compaction and moisture content within the suggested ranges are obtained.

Use of Squeegee Relatively uniformly graded fine gravel or coarse sand, i.e., “squeegee,” or similar materials commonly are proposed for backfilling foundation excavations, utility trenches (excluding approved pipe bedding), and other areas where employing compaction equipment is difficult. In general, GROUND does not suggest this procedure for the following reasons:

Although commonly considered “self-compacting,” uniformly graded granular materials require densification after placement, typically by vibration. The equipment to densify these materials is not available on many job-sites.

Even when properly densified, granular materials are permeable and allow water to reach and collect in the lower portions of the excavations backfilled with those materials. This

leads to wetting of the underlying soils and resultant potential loss of bearing support as well as increased local heave or settlement.

It is GROUND's opinion that wherever possible, excavations be backfilled with approved, on-site soils placed as properly compacted fill. Where this is not feasible, use of "Controlled Low Strength Material" (CLSM), i.e., a lean, sand-cement slurry ("flowable fill") or a similar material for backfilling should be considered.

Where "squeegee" or similar materials are proposed for use by the contractor, the design team should be notified by means of a Request for Information (RFI), so that the proposed use can be considered on a case-by-case basis. "Squeegee" can be used where deemed acceptable by the project documents.

Settlements Settlements will occur in filled ground, typically on the order of 1 to 2 percent of the fill depth. For a 6-foot fill, for example, this corresponds to a settlement of about 1 inch. If fill placement is performed properly and is tightly controlled, in GROUND's experience the majority (on the order of 60 to 80 percent) of that settlement will typically take place during earthwork construction, provided the contractor achieves the compaction levels herein. The remaining potential settlements likely will take several months or longer to be realized, and may be exacerbated if these fills are subjected to changes in moisture content.

Cut and Filled Slopes Permanent site slopes supported by on-site soils up to 5 feet in height may be constructed no steeper than 3:1 (horizontal : vertical). Minor raveling or surficial sloughing should be anticipated on slopes cut at this angle until vegetation is well re-established. Surface drainage should be designed to direct water away from slope faces.

EXCAVATION CONSIDERATIONS

Excavation Difficulty Test holes for the subsurface exploration were advanced to the depths indicated on the test hole logs by means of conventional, truck/track-mounted, geotechnical drilling equipment. We anticipate no significant excavation difficulties in the majority of the site with conventional heavy-duty excavation equipment in good working condition. Local cobbles may be encountered within project excavations and the contractor should be prepared to handle materials of this nature.

Any excavations in which personnel will be working must comply with all OSHA Standards and Regulations (CFR 29 Part 1926). The contractor's "responsible person" should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. GROUND has provided the information above solely as a service to the client, and is not assuming responsibility for construction site safety or the contractor's activities.

Surface Water and Groundwater: Good surface drainage should be provided around temporary excavation slopes to direct surface runoff away from the slope faces. A properly designed swale should be provided at the top of the excavations. In no case should water be allowed to pond at the site. Slopes should be protected against erosion. Erosion along the slopes will result in sloughing and could lead to a slope failure.

Groundwater was observed locally in test hole 2 at a depth of 32 feet. Therefore, groundwater is not anticipated to be a significant factor during construction of this project. However, if seepage or groundwater is encountered in project excavations, the Geotechnical Engineer should evaluate the conditions and provide additional recommendations, as appropriate.

UTILITY PIPE INSTALLATION AND BACKFILLING

The measures and criteria below are based on GROUND's evaluation of the local, geotechnical conditions. Where the parameters herein differ from applicable municipal requirements, the latter should be considered to govern.

Pipe Support The bearing capacity of the site soils appeared adequate, in general, for support of buried utilities. The pipes + contents, typically, are less dense than the soils which will be displaced for installation. Therefore, GROUND anticipates no significant pipe settlements in these materials where properly bedded.

Excavation bottoms may expose soft, loose or otherwise deleterious materials, including debris. Firm materials may be disturbed by the excavation process. All such unsuitable materials should be excavated and replaced with properly compacted fill. Areas allowed to pond water will require excavation and replacement with properly compacted fill. The contractor should take particular care to ensure adequate support near pipe joints which are less tolerant of extensional strains.

Where thrust blocks are needed, they may be designed utilizing the parameters set forth in the *Later Earth Pressures* section of this report.

Trench Backfilling Some settlement of compacted soil trench backfill materials should be anticipated, even where all the backfill is placed and compacted correctly. Typical settlements are on the order of 1 to 2 percent of fill thickness. However, the need to compact to the lowest portion of the backfill must be balanced against the need to protect the pipe from damage from the compaction process. Some thickness of backfill may need to be placed at compaction levels lower than recommended or specified (or smaller compaction equipment used together with thinner lifts) to avoid damaging the pipe. Protecting the pipe in this manner can result in somewhat greater surface settlements. Therefore, although other alternatives may be available, the following options are presented for consideration:

Controlled Low Strength Material: Because of these limitations, we recommend backfilling the entire depth of the trench (both bedding and common backfill zones) with “controlled low strength material” (CLSM), i.e., a lean, sand-cement slurry, “flowable fill,” or similar material along all trench alignment reaches with low tolerances for surface settlements.

We recommend that CLSM used as pipe bedding and trench backfill exhibit a 28-day unconfined compressive strength between 50 to 150 psi so that re-excavation is not unusually difficult.

Placement of the CLSM in several lifts or other measures likely will be necessary to avoid ‘floating’ the pipe. Measures also should be taken to maintain pipe alignment during CLSM placement.

Compacted Soil Backfilling: Where compacted soil backfilling is employed, using the site soils or similar materials as backfill, the risk of backfill settlements entailed in the selection of this higher risk alternative must be anticipated and accepted by the Client/Owner.

We anticipate that the on-site soils excavated from trenches will be suitable, in general, for use as common trench backfill within the above-described limitations. Backfill soils should be free of vegetation, organic debris and other deleterious materials. Fragments of rock, cobbles, and inert construction debris (e.g., concrete or asphalt) coarser than 3 inches in maximum dimension should not be incorporated into trench backfills.

If it is necessary to import material for use as backfill, the imported soils should meet the requirements set for in the *Project Earthwork* section of this report. Representative samples of the materials proposed for import should be tested and approved prior to transport to the site.

Soils placed for compaction as trench backfill should be conditioned to a relatively uniform moisture content, placed and compacted in accordance with the parameters in the *Project Earthwork* section of this report.

Pipe Bedding Pipe bedding materials, placement and compaction should meet the specifications of the pipe manufacturer and applicable municipal standards. Bedding should be brought up uniformly on both sides of the pipe to reduce differential loadings.

As discussed above, we recommend the use of CLSM or similar material in lieu of granular bedding and compacted soil backfill where the tolerance for surface settlement is low. (Placement of CLSM as bedding to at least 12 inches above the pipe can protect the pipe and assist construction of a well-compacted conventional backfill, although possibly at an increased cost relative to the use of conventional bedding.)

If a granular bedding material is specified, GROUND recommends that with regard to potential migration of fines into the pipe bedding, design and installation follow ASTM D2321. If the granular bedding does not meet filter criteria for the enclosing soils, then non-woven filter fabric (e.g., Mirafi® 140N, or the equivalent) should be placed around the bedding to reduce migration of fines into the bedding which can result in severe, local surface settlements. Where this protection is not provided, settlements can develop/continue several months or years after completion of the project. In addition, clay or concrete cut-off walls should be installed to interrupt the granular bedding section to reduce the rates and volumes of water transmitted along the utility alignment which can contribute to migration of fines.

If granular bedding is specified, the site derived materials should not be anticipated to be suitable for that use. Materials proposed for use as pipe bedding should be tested for suitability prior to use. Imported materials should be tested and approved prior to transport to the site.

SURFACE DRAINAGE

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The site soils are relatively stable with regard to moisture content – volume relationships at their existing moisture contents. Other than the anticipated, post-placement settlement of fills, post-construction soil movements will result primarily from the introduction of water into the soils underlying the proposed structure, hardscaping and pavements. Assuming that suggested increases in site elevation are performed, we do not anticipate a rise in the local water table sufficient to approach grade beam or floor elevations. Therefore, wetting of the soils likely will result from infiltrating surface waters (precipitation, irrigation, etc.), and water flowing along constructed pathways such as bedding in utility pipe trenches.

The following drainage measures should be followed both for during construction and as part of project design. The facility should be observed periodically to evaluate the surface drainage and identify areas where drainage is ineffective. Routine maintenance of site drainage should undertaken throughout the design life of the proposed facility. If these measures are not implemented and maintained effectively, the movement estimates provided in this report could be exceeded.

- 1) Wetting or drying of the underslab areas should be avoided during and after construction. Permitting increases/variations in moisture to the adjacent or supporting soils may result in increased total and/or differential movements.
- 2) Positive surface drainage measures should be provided and maintained to reduce water infiltration into foundation soils.

The ground surface surrounding the exterior of each building should be sloped to drain away from the foundation in all directions. A minimum slope of 12 inches in the first 10 feet should be constructed in the areas not covered with pavement or concrete slabs, or a minimum of 3 percent in the first 10 feet in the areas covered with pavement or concrete slabs. Reducing the slopes to comply with ADA requirements or other reasons may be necessary but may result in an increased potential for moisture infiltration and subsequent volume change of the underlying soils.

In no case should water be allowed to pond near or adjacent to foundation elements, hardscaping, etc.

- 3) Drainage also should be established to direct water away from sidewalks and other hardscaping as well as utility trench alignments which are not tolerant of moisture-volume changes in the underlying soils or flow of infiltrating water.

The ground surface near foundation elements should be able to convey water away readily. Cobbles or other materials that tend to act as baffles and restrict surface flow should not be used to cover the ground surface near the foundations.

Where the ground surface does not convey water away readily, additional post-construction movements and distress should be anticipated.

- 4) In GROUND's experience, it is common during construction that in areas of partially completed paving or hardscaping, bare soil behind curbs and gutters, and utility trenches, water is allowed to pond after rain or snow-melt events. Wetting of the subgrade can result in loss of subgrade support and increased settlements / increased heave. By the time final grading has been completed, significant volumes of water can already have entered the subgrade, leading to subsequent distress and failures. The contractor should maintain effective site drainage throughout construction so that water is directed into appropriate drainage structures.

In no case should water be permitted to pond adjacent to or on sidewalks, hardscaping, or other improvements as well as utility trench alignments, which are likely to be adversely affected by moisture-volume changes in the underlying soils or flow of infiltrating water.

- 5) Roof downspouts and drains, if used, should discharge well beyond the perimeter of the structure foundation, or be provided with positive conveyance off-site for collected waters.

If roof downspouts and drains are not used, then surface drainage design should anticipate concentrated volumes of water adjacent to the buildings.

- 6) Irrigation water – both that applied to landscaped areas and over-spray – commonly is a significant cause of distress to improvements. Where (near-) saturated soil conditions are sustained, distress to nearby improvements should be anticipated.

To reduce the potential for such distress, vegetation requiring watering should be located 10 or more feet from the building perimeter, flatwork, or other improvements. Irrigation sprinkler heads should be deployed so that applied water is not introduced near or into foundation/subgrade soils. Landscape irrigation should be limited to the minimum quantities necessary to sustain healthy plant growth.

Use of drip irrigation systems can be beneficial for reducing over-spray beyond planters. Drip irrigation also can be beneficial for reducing the amounts of water introduced to building foundation soils, but only if the total volumes of applied water are controlled with regard to limiting that introduction. Controlling rates of moisture increase beneath the foundations, floors and other improvements should take higher priority than minimizing landscape plant losses.

Where plantings are desired within 10 feet of the building, plants should be placed in water-tight planters, constructed either in-ground or above-grade, to reduce moisture infiltration in the surrounding subgrade soils. Planters should be provided with positive drainage and landscape underdrains.

As an alternative involving only a limited increase in risk, the use of water-tight planters may be replaced by local shallow underdrains beneath the planter beds.

- 7) Plastic membranes should not be used to cover the ground surface near the building without careful consideration of other components of project drainage. Plastic membranes can be beneficial to directing surface waters away from the building and toward drainage structures. However, they effectively preclude evaporation and transpiration of shallow soil moisture. Therefore, soil moisture tends to increase beneath a continuous membrane. Where plastic membranes are used, additional shallow, subsurface drains should be installed.

Perforated “weed barrier” membranes that allow ready evaporation from the underlying soils may be used.

SUBSURFACE DRAINAGE

As a component of project civil design, properly functioning, subsurface drain systems (underdrains) can be beneficial for collecting and discharging saturated subsurface

waters. Underdrains will not collect water infiltrating under unsaturated (vadose) conditions, or moving via capillarity, however. In addition, if not properly constructed and maintained, underdrains can transfer water into foundation soils, rather than remove it. This will tend to induce heave or settlement of the subsurface soils, and may result in distress. Underdrains can, however, provide an added level of protection against relatively severe post-construction movements by draining saturated conditions near individual structures should they arise, and limiting the volume of wetted soil.

Professional opinion varies regarding the potential benefits relative to the cost of an underdrain system. Therefore, the owner and the design team and contractor should assess the net benefit of an underdrain system as a component of overall project drainage. (GROUND does not specifically recommend underdrains at this site with the exception of below grade levels if proper surface drainage measures are implemented.)

If, however, below-grade or partially below-grade level(s) are incorporated into project design, then an underdrain system should be included to protect those portions of the building. Damp-proofing should be applied to the exteriors of below-grade elements. The provision of Tencate MiraFi® G-Series backing (or comparable wall drain provisions) on the exteriors of (some) below-grade elements may be appropriate, depending on the intended use. If a (partially) below-grade level is limited in extent, the underdrain system, etc., may be local to that area.

Geotechnical Parameters for Underdrain Design Where an underdrain system is included in project drainage design, it should be designed in accordance with the parameters below. The actual underdrain layout, outlets, and locations should be developed by a civil engineer.

An underdrain system should be tested by the contractor after installation and after placement and compaction of the overlying backfill to verify that the system functions properly.

- 1) An underdrain system for a building should consist of perforated, rigid, PVC collection pipe at least 4 inches in diameter, non-perforated, rigid, PVC discharge pipe at least 4 inches in diameter, free-draining gravel, and filter fabric, as well as a waterproof membrane.

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- 2) The free-draining gravel should contain less than 5 percent passing the No. 200 Sieve and more than 50 percent retained on the No. 4 Sieve, and have a maximum particle size of 2 inches. Each collection pipe should be surrounded on the sides and top (only) with 6 or more inches of free-draining gravel.
- 3) The gravel surrounding the collection pipe(s) should be wrapped with filter fabric (MiraFi 140N® or the equivalent) to reduce the migration of fines into the drain system.
- 4) The waterproof membrane should underlie the gravel and pipe, and be attached to the foundation grade beam or stem wall.
- 5) The underdrain system should be designed to discharge at least 10 gallons per minute of collected water.
- 6) The high point(s) for the collection pipe flow lines should be below the lowest foundation bearing elevation. Multiple high points can be beneficial to reducing the depths to which the system would be installed.

The collection and discharge pipe for the underdrain system should be laid on a slope sufficient for effective drainage, but a minimum of 1 percent. (Flatter gradients may be used but will convey water less efficiently and entail an increased risk of local post-construction movements.)

Pipe gradients also should be designed to accommodate at least 1 inch of differential movement after installation along a 50-foot run.

- 7) Underdrain 'clean-outs' should be provided at intervals of no more than 100 feet to facilitate maintenance of the underdrains. Clean-outs also should be provided at collection and discharge pipe elbows of 60 degrees or more.
- 8) The underdrain discharge pipes should be connected to one or more sumps from which water can be removed by pumping, or to outlet(s) for gravity discharge. We suggest that collected waters be discharged directly into the storm sewer system, if possible.

INFILTRATION TESTING

Infiltration rate testing was performed within the existing detention pond to the east of existing Erie Police Station on March 3, 2025 using Modified Philip- Dunne Infiltrometer (MPD) testing equipment. Testing was performed at three (3) locations. All of the tests were performed at depths ranging from 3 to 7 inches below rough surface grade. Materials encountered in the top foot below rough surface grade generally consisted of clayey sand. A summary of the data obtained during MPD testing is provided in *Appendix B*. The locations presented on the map in Appendix B should be considered only as a general reference.

A brief summary of the resulting **Ksat** (saturated hydraulic conductivity) values is provided below. "Infiltration rate" values are also provided below. The "infiltration rates" are based on the **Ksat** values and are considered to represent infiltration in the stabilized condition after the soil has become saturated locally and the rate of infiltration has become effectively constant. They are presented in the 'minutes per inch' units of a traditional percolation test result.

MODIFIED PHILLIP-DUNNE INFILTROMETER TEST RESULTS SUMMARY

Infiltration Test	Approximate Test Depth (in)	Ksat (in/hr)	Infiltration Rate (minutes/in)
1	7	1.15	52.2
2	5	1.21	53.6
3	3	3.53	17.0

It should be noted infiltration rates can vary greatly over short distances (both vertical and horizontal) and field testing may not effectively indicate local areas of either significantly higher or lower hydraulic conductivity (particularly where earth materials are variable in composition and consistency). Similarly, in-situ soil structure, moisture contents, and consistency have significant impacts on the infiltration rates of soils. Man-placed fills, even when derived from site soils, should be anticipated to possess somewhat different infiltration properties. A qualified engineer should evaluate the results of these tests before

they are used in design. Similarly, it may be beneficial to perform additional infiltration rate testing, to verify design infiltration rates following construction.

PAVEMENT SECTIONS

A pavement section is a layered system designed to distribute concentrated traffic loads to the subgrade. Performance of the pavement structure is directly related to the physical properties of the subgrade soils and traffic loadings. The standard care of practice in pavement design describes the flexible pavement section as a “20-year” design pavement; however, most flexible pavements will not remain in satisfactory condition without routine maintenance and rehabilitation procedures performed throughout the life of the pavement. Pavement designs for the private pavements were developed in general accordance with the design guidelines and procedures of the American Association of State Highway and Transportation Officials (AASHTO).

Subgrade Materials Based on our review of the above referenced documentation and laboratory testing, the subgrade materials below the project pavements consisted largely of sand and clay subgrade materials.

Based on our experience with the site soils and similar materials, an R-value of 20 was estimated for the on-site materials. This value converts to a resilient modulus value of 4,940 psi based on CDOT correlation tables. It is important to note that significant decreases in soil support as quantified by the resilient modulus have been observed as the moisture content increases above the optimum. Therefore, pavements that are not properly drained may experience a loss of the soil support and subsequent reduction in pavement life.

Anticipated Traffic Traffic data for the proposed facility was unavailable at the time of our report preparation. Based on our experience with similar projects equivalent an 18-kip daily load application (EDLA) values of 10 and 30 were assumed for the Light Duty and Heavy Duty traffic areas, respectively. The EDLA values of 10 and 30 were converted to equivalent 18-kip single axle load (ESAL) values of 73,000 and 219,000 for a 20-year design life. If anticipated traffic loadings differ significantly from these assumed values, GROUND should be notified to re-evaluate the pavement sections below.

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If anticipated traffic loadings differ significantly from these assumed values, GROUND should be notified to re-evaluate the pavement thickness provided below.

Pavement Sections The soil resilient modulus and the ESAL values were used to determine the required design structural number for the project pavements. The required structural number was then used to develop the pavement sections. Pavement designs were based on the DARWin™ computer program that solves the 1993 AASHTO pavement design equations. A Reliability Level of 80 percent was utilized to develop the pavement sections, together with a Serviceability index loss of 2.5. An overall standard of deviation of 0.44 also was used. Structural coefficients of 0.44 and 0.11 were used for hot bituminous asphalt and aggregate base course, respectively. The resultant minimum pavement sections that should be used at the facility are tabulated below.

****Minimum Pavement Sections***

<i>Location</i>	<i>Full Depth Asphalt (inches Asphalt)</i>	<i>Composite Section (inches Asphalt / inches ABC)</i>	<i>Rigid Section (inches PCCP / inches ABC)</i>
Light Duty Pavement	5.5	4 / 6	6 / 6
Heavy Duty Pavement	-	5 / 6	7 / 6

ABC = Aggregate Base Course, PCCP= Portland Cement Concrete Pavement

Pavement areas subjected to high turning stresses or frequent starting or stopping of heavy vehicles such as busses, trucks, trash collection areas, should be provided with rigid pavements consisting of **7 or more inches** of portland cement concrete underlain by **6 inches** of properly compacted CDOT Class 6 Aggregate Base Course. In our experience, asphalt pavements will not perform as well as rigid pavement in areas of high turning stresses, prolonged static loading, or frequent starting and stopping of heavy vehicles, and additional maintenance costs should be anticipated if the asphalt sections are utilized in these areas.

Pavement Materials Asphalt pavement should consist of a bituminous plant mix composed of a mixture of aggregate and bituminous material. Asphalt mixture(s) should

meet the requirements of a job-mix formula established by a qualified engineer as well as applicable local municipality design requirements.

Based on our experience with similar projects, aggregate gradation **S** (nominal ¾-inch) and binder type **PG58-28 or better** should be used for the lower lift(s), and gradation **SX** (nominal ½-inch) and binder type **PG64-22 or better** for the top lift. Other binder types may be appropriate based on the Town of Erie Public Works department's performance expectations, experience, and project budget.

For the lower (S) lift(s), lift thicknesses generally should be between **2¼ and 3½ inches**. The top (SX) lift generally should be between **2 and 3 inches** in thickness.

Aggregate base material should meet the criteria of CDOT Class 5 or 6 Aggregate Base Course. Base course should be placed in and compacted in accordance with the standards in the *Project Earthwork* section of this report.

Pavement concrete should consist of a plant mix composed of a mixture of aggregate, portland cement and appropriate admixtures meeting the requirements of a job-mix formula established by a qualified engineer as well as applicable municipal design requirements design requirements. Concrete should have a minimum modulus of rupture of third point loading of **650 psi**. Normally, concrete with a 28-day compressive strength of **4,500 psi** should develop this modulus of rupture value. The concrete should be air-entrained with approximately 6 percent air and should have a minimum cement content of **6 sacks per cubic yard**. Maximum allowable slump should be **4 inches**.

These concrete mix design criteria should be coordinated with other project requirements including any criteria for sulfate resistance presented in the *Water-Soluble Sulfates* section of this report. To reduce surficial spalling resulting from freeze-thaw cycling, we suggest that pavement concrete meet the requirements of CDOT Class P concrete. In addition, the use of de-icing salts on concrete pavements during the first winter after construction will increase the likelihood of the development of scaling. Placement of flatwork concrete during cold weather so that it is exposed to freeze-thaw cycling before it is fully cured also increases its vulnerability to scaling. Concrete placing during cold weather conditions should be blanketed or tented to allow full curing. Depending on the weather conditions, this may result in 3 to 4 weeks of curing, and possibly more.

Concrete pavements should contain sawed or formed joints. CDOT and various industry groups provide guidelines for proper design and concrete construction and associated jointing. In areas of repeated turning stresses, such as truck loading and unloading areas, the concrete pavement joints should be fully tied and doweled. Example layouts for joints, as well as ties and dowels, which may be applicable, can be found in CDOT's M standards, found at the CDOT website: <http://www.dot.state.co.us/DesignSupport/>. PCA, ACI, and ACPA publications also provide useful guidance in these regards. Joint spacings less than the 15-foot maximum indicated in CDOT's M standards, e.g., 10 feet or 12 feet, may be beneficial to reduce concrete cracking.

Subgrade Preparation Due to the potential for hydro-collapse noted in the overburden sandy soils, it is GROUND's opinion that, the pavement subgrade should be over-excavated to a minimum depth of **36 inches**, moisture-conditioned and properly re-compacted. Although subgrade preparation to this depth will not eliminate the hydro-collapse potential within the overburden soils, it work will tend to make the movements more uniform and less damaging. Additionally, rework to this depth was previously performed during the original construction of the Erie Police station, and the flatwork and parking at the existing facility appears to be performing acceptably. More than typical flatwork maintenance should be anticipated by the facility owner even with this depth of rework below project pavements.

Subgrade preparation should extend the full width of the pavement from back-of-curb to back-of-curb. The subgrade for any sidewalks and other project hardscaping also should be prepared in the same manner.

Geotechnical criteria for fill placement and compaction are provided in the *Project Earthwork* section of this report. The contractor should be prepared to either dry the subgrade materials or moisten them, as needed, prior to compaction.

Proof Rolling Immediately prior to paving, the subgrade should be proof rolled with a heavily loaded, pneumatic tired vehicle. Areas that show excessive deflection during proof rolling should be excavated and replaced and/or stabilized. Areas allowed to pond prior to paving will require significant re-working prior to proof-rolling. Establishment of a firm paving platform (as indicated by proof rolling) is an additional requirement beyond proper fill placement and compaction. It is possible for soils to be compacted within the limits

indicated in the *Project Earthwork* section of this report and fail proof rolling, particularly in the upper range of moisture content.

Additional Observations The collection and diversion of surface drainage away from paved areas is extremely important to the satisfactory performance of the pavements. The subsurface and surface drainage systems should be carefully designed to ensure removal of the water from paved areas and subgrade soils. Allowing surface waters to pond on pavements will cause premature pavement deterioration. Where topography, site constraints, or other factors limit or preclude adequate surface drainage, pavements should be provided with edge drains to reduce loss of subgrade support. The long-term performance of the pavement also can be improved greatly by proper backfilling and compaction behind curbs, gutters, and sidewalks so that ponding is not permitted and water infiltration is reduced.

Landscape irrigation in planters adjacent to pavements and in “island” planters within paved areas should be carefully controlled or differential heave and/or rutting of the nearby pavements will result. Drip irrigation systems are suggested for such planters to reduce over-spray and water infiltration beyond the planters. Enclosing the soil in the planters with plastic liners and providing them with positive drainage also will reduce differential moisture increases in the surrounding subgrade soils.

In our experience, infiltration from planters adjacent to pavements is a principal source of moisture increase beneath those pavements. This wetting of the subgrade soils from infiltrating irrigation commonly leads to loss of subgrade support for the pavement with resultant accelerating distress, loss of pavement life and increased maintenance costs. This is particularly the case in the later stages of project construction after landscaping has been emplaced but heavy construction traffic has not ended. Heavy vehicle traffic over wetted subgrade commonly results in rutting and pushing of flexible pavements, and cracking of rigid pavements. In relatively flat areas where design drainage gradients necessarily are small, subgrade settlement can obstruct proper drainage and yield increased infiltration, exaggerated distress, etc. (These considerations apply to project flatwork, as well.)

Also, GROUND's experience indicates that longitudinal cracking is common in asphalt-pavements generally parallel to the interface between the asphalt and concrete structures

such as curbs, gutters, or drain pans. Distress of this type is likely to occur even where the subgrade has been prepared properly and the asphalt has been compacted properly.

The anticipated traffic loading does not include excess loading conditions imposed by heavy construction vehicles. Consequently, heavily loaded concrete, lumber, and building material trucks can have a detrimental effect on the pavement.

Most pavements will not remain in satisfactory condition and achieve their “design lives” without regular maintenance and rehabilitation procedures performed throughout the life of the pavement. Maintenance and rehabilitation measures preserve, rather than improve, the structural capacity of the pavement structure. Therefore, an effective program of regular maintenance should be developed and implemented to seal cracks, repair distressed areas, and perform thin overlays throughout the lives of the pavements. The greatest benefit of pavement overlaying will be achieved by overlaying sound pavements that exhibit little or no distress.

Crack sealing should be performed at least annually and a fog seal/chip seal program should be performed on the pavements every 3 to 4 years. After approximately 8 to 10 years after construction, patching, additional crack sealing, and asphalt overlay may be required. Prior to overlays, it is important that all cracks be sealed with a flexible, rubberized crack sealant in order to reduce the potential for propagation of the crack through the overlay. If actual traffic loadings exceed the values used for development of the pavement sections, however, pavement maintenance measures will be needed on an accelerated schedule.

Temporary Fire Access Routes Commonly, construction sites are required by local fire departments to provide temporary access for emergency response. It has been GROUND’s experience these access drives are to provide support for trucks weighing up to 90,000 pounds and are typically desired to be gravel/aggregate-surfaced.

Based on our experience, a temporary section consisting of **at least 12 inches** of material meeting the requirements of CDOT Class 5 or Class 6 Aggregate Base Course or at least **8 inches** of CDOT Class 5 or Class 6 Aggregate Base Course over **a layer of stabilization geotextile/geofabric**, such as Mirafi® RS380i or the equivalent, could be utilized provided the owner understands that this section is for temporary access during construction only and is not a replacement or an equal alternate to the pavement

section(s) that was indicated previously. The aggregate base course placed for this purpose should be compacted to at least 95 percent of the maximum modified Proctor dry density. It should be noted that the aggregate base course sections indicated above are not intended to support fire truck outriggers without cribbing or similar measures.

The aggregate comprising such a wearing course will be displaced and rutted under the loads imposed by heavy vehicles. Therefore, regular maintenance including re-grading and application of additional aggregate should be implemented to ensure proper drainage, repair distressed/damaged areas, and re-establish grades. Additionally, the ability of a temporary aggregate-surfaced route to accommodate loads as indicated above is directly related to the quality of the subgrade materials on which the aggregate is placed, not only on the aggregate section. If water infiltrates these areas, additional rutting and other distress, including a reduction in capacity, will result, requiring additional maintenance.

EXTERIOR FLATWORK

We anticipate that the exterior of the proposed building and other portions of the site will be provided with concrete flatwork. Like other site improvements, flatwork will experience post-construction movements as soil moisture contents increase after construction and distress likely will result. The following measures will help to reduce damages to these improvements, but will not prevent all movements. Critical flatwork, which may include flatwork at entrances and exits, should be constructed as a structural floor or slab-on-grade floor supported on aggregate piers in a similar manner to project floors. Such areas should be identified by the owner.

- 1) The subgrade under exterior flatwork or other (non-building) site improvements should consist of approximately **36 inches** of moisture-conditioned and re-compacted site generated materials as outlined in the Project Earthwork section of this report.

Greater depths of subgrade preparation will tend to reduce the extent and frequency of maintenance for these surficial improvements.

- 2) Prior to placement of flatwork, a proof roll should be performed to identify areas that exhibit instability and deflection. The deleterious soils in these areas should be removed and replaced with properly compacted fill. The contractor should take

care to achieve and maintain compaction behind curbs to reduce differential sidewalk settlements. Passing a proof roll is an additional requirement to placing and compacting the subgrade fill soils within the specified ranges of moisture content and relative compaction in the *Project Earthwork* section of this report. Subgrade stabilization may be cost-effective in this regard.

- 3) Flatwork should be provided with control joints extending to an effective depth and spaced no more than **10 feet** apart, both ways. Narrow flatwork, such as sidewalks, likely will require more closely spaced joints.
- 4) In no case should exterior flatwork extend to under any portion of the building where there is less than **2 inches** of vertical clearance between the flatwork and any element of the building. Exterior flatwork in contact with brick, rock facades, or any other element of the building can cause damage to the structure if the flatwork experiences movements.

Construction and Drainage Between Buildings and Pavements Proper design, drainage, construction and maintenance of the areas between individual buildings and parking/driveway areas are critical to the satisfactory performance of the project. Sidewalks, entranceway slabs and roofs, fountains, raised planters and other highly visible improvements commonly are installed within these zones, and distress in or near these improvements is common. Commonly, proper soil preparation in these areas receives little attention during overlot construction because they fall between the building and pavement areas which typically are built with heavy equipment. Subsequent landscaping and hardscape installation often is performed by multiple sub-contractors with light or hand equipment, and necessary over-excavation and soil processing is not performed. Consequently, subgrade soil conditions commonly deviate significantly from specified ranges. Therefore, the contractor should take particular care with regard to proper subgrade preparation in the immediate building exteriors.

Concrete Scaling Climatic conditions in the project area including relatively low humidity, large temperature changes and repeated freeze – thaw cycles, make it likely that project sidewalks and other exterior concrete will experience surficial scaling or spalling. The likelihood of concrete scaling can be increased by poor workmanship during construction, such as ‘over-finishing’ the surfaces. In addition, the use of de-icing salts on exterior concrete flatwork, particularly during the first winter after construction, will increase the

likelihood of scaling. Even use of de-icing salts on nearby roadways, from where vehicle traffic can transfer them to newly placed concrete, can be sufficient to induce scaling. Typical quality control / quality assurance tests that are performed during construction for concrete strength, air content, etc., do not provide information with regard to the properties and conditions that give rise to scaling.

We understand that some municipalities require removal and replacement of concrete that exhibits scaling, even if the material was within specification and placed correctly. The contractor should be aware of the local requirements and be prepared to take measures to reduce the potential for scaling and/or replace concrete that scales.

In GROUND's experience, the measures below can be beneficial for reducing the likelihood of concrete scaling. Which measures, if any, used should be based on cost and the owner's tolerance for risk and maintenance. It must be understood, however, that because of the other factors involved, including weather conditions and workmanship, surface damage to concrete can develop, even where all of these measures were followed. Also, the mix design criteria should be coordinated with other project requirements including criteria for sulfate resistance presented in the *Water-Soluble Sulfates* section of this report.

- 1) Maintaining a maximum water/cement ratio of 0.45 by weight for exterior concrete mixes.
- 2) Include Type F fly ash in exterior concrete mixes as 20 percent of the cementitious material.
- 3) Specify a minimum, 28-day, compressive strength of 4,500 psi for all exterior concrete.
- 4) Including 'fibermesh' in the concrete mix also may be beneficial for reducing surficial scaling.
- 5) Cure the concrete effectively at uniform temperature and humidity. This commonly will require fogging, blanketing and/or tenting, depending on the weather conditions. As long as 3 to 4 weeks of curing may be required, and possibly more.

- 6) Avoid placement of concrete during cold weather so that it is not exposed to freeze-thaw cycling before it is fully cured.
- 7) Avoid the use of de-icing salts on given reaches of flatwork through the first winter after construction.

We understand that sometimes it is not practical to implement some of these measures for reducing scaling due to safety considerations, project scheduling, etc. In such cases, where these measures are not implemented, additional costs for flatwork maintenance or reconstruction should be incorporated into project budgets.

Frost and Ice Considerations Nearly all soils other than relatively coarse, clean, granular materials are susceptible to loss of density if allowed to become saturated and exposed to freezing temperatures and repeated freeze – thaw cycling. The formation of ice in the underlying soils can result in heaving of pavements, flatwork, and other hardscaping (“ice jacking”) in sustained cold weather up to 2 inches or more. This heaving can develop relatively rapidly. A portion of this movement typically is recovered when the soils thaw, but due to loss of soil density, some degree of displacement will remain. This can result even where the subgrade soils were prepared properly.

Where hardscape movements are a design concern, e.g., at doorways, replacement of the subgrade soils with 3 or more feet of clean, coarse sand or gravel should be considered or supporting the element on foundations similar to the building and spanning over a void. Detailed guidance in this regard can be provided upon request. It should be noted that where such open graded granular soils are placed, water can infiltrate and accumulate in the subsurface relatively easily, which can lead to increased settlement or heave from factors unrelated to ice formation. Therefore, where a section of open graded granular soils are placed, a local underdrain system should be provided to discharge collected water. GROUND will be available to discuss these concerns upon request.

CLOSURE AND LIMITATIONS

Geotechnical Review The author of this report or a GROUND principal should be retained to review project plans and specifications to evaluate whether they comply with the intent of the measures discussed in this report. The review should be requested in writing.

**Erie Police Station Addition
Erie, Colorado**

The geotechnical conclusions and parameters presented in this report are contingent upon observation and testing of project earthwork by representatives of GROUND. If another geotechnical consultant is selected to provide materials testing, then that consultant must assume all responsibility for the geotechnical aspects of the project by concurring in writing with the parameters in this report, or by providing alternative parameters.

Materials Testing The Erie Public Works department should consider retaining a geotechnical engineer to perform materials testing during construction. The performance of such testing or lack thereof, however, in no way alleviates the burden of the contractor or subcontractor from constructing in a manner that conforms to applicable project documents and industry standards. The contractor or pertinent subcontractor is ultimately responsible for managing the quality of his work; furthermore, testing by the geotechnical engineer does not preclude the contractor from obtaining or providing whatever services that he deems necessary to complete the project in accordance with applicable documents.

Limitations This report has been prepared for the Town of Erie Public Works department as it pertains to design of the proposed Erie Police Station Addition in Erie, Colorado as described herein. It should not be assumed to contain sufficient information for other parties or other purposes. The Client has agreed to the terms, conditions, and liability limitations outlined in our agreement between the Town of Erie Public Works department and GROUND. Reliance upon our report is not granted to any other potential owner, contractor, or lender. Requests for third-party reliance should be directed to GROUND in writing; granting reliance by GROUND is not guaranteed.

In addition, GROUND has assumed that project construction will commence by winter of 2025. Any changes in project plans or schedule should be brought to the attention of a geotechnical engineer, in order that the geotechnical conclusions in this report may be re-evaluated and, as necessary, modified.

The geotechnical conclusions in this report were based on subsurface information from a limited number of exploration points, as shown in Figure 1, as well as the means and methods described herein. Subsurface conditions were interpolated between and extrapolated beyond these locations. It is not possible to guarantee the subsurface conditions are as indicated in this report. Actual conditions exposed during construction may differ from those encountered during site exploration. In addition, a contractor who

**Erie Police Station Addition
Erie, Colorado**

obtains information from this report for development of his scope of work or cost estimates does so solely at his own risk and may find the geotechnical information in this report to be inadequate for his purposes or find the geotechnical conditions described herein to be at variance with his experience in the greater project area. The contractor should obtain the additional geotechnical information that is necessary to develop his workscope and cost estimates with sufficient precision. This includes, but is not limited to, information regarding excavation conditions, earth material usage, current depths to groundwater, etc.

If during construction, surface, soil, bedrock, or groundwater conditions appear to be at variance with those described herein, a geotechnical engineer should be retained at once, so that our conclusions for this site may be re-evaluated in a timely manner and dependent aspects of project design can be modified, as necessary.

The materials present on-site are stable at their natural moisture content, but may change volume or lose bearing capacity or stability with changes in moisture content. Performance of the proposed structure and pavement will depend on implementation of the conclusions and information in this report and on proper maintenance after construction is completed. Because water is a significant cause of volume change in soils and rock, allowing moisture infiltration may result in movements, some of which will exceed estimates provided herein and should therefore be expected by the Town of Erie Public Works department.

ALL DEVELOPMENT CONTAINS INHERENT RISKS. It is important that ALL aspects of this report, as well as the estimated performance (and limitations with any such estimations) of proposed improvements are understood by the Town of Erie Public Works department. Utilizing the geotechnical parameters and measures herein for planning, design, and/or construction constitutes understanding and acceptance of the conclusions with regard to risk and other information provided herein, associated improvement performance, as well as the limitations inherent within such estimates. Ensuring correct interpretation of the contents of this report by others is not the responsibility of GROUND. If any information referred to herein is not well understood, it is imperative that the Town of Erie Public Works department contact the author or a GROUND principal immediately. We will be available to meet to discuss the risks and remedial approaches presented in this report, as well as other potential approaches, upon request.

Current applicable codes may contain criteria regarding performance of structures and/or site improvements which may differ from those provided herein. Our office should be contacted regarding any apparent disparity.

**Erie Police Station Addition
Erie, Colorado**

GROUND makes no warranties, either expressed or implied, as to the professional data, opinions or conclusions contained herein. Because of numerous considerations that are beyond GROUND's control, the economic or technical performance of the project cannot be guaranteed in any respect.

This document, together with the concepts and conclusions presented herein, as an instrument of service, is intended only for the specific purpose and client for which it was prepared. Re-use of, or improper reliance on this document without written authorization and adaption by GROUND Engineering Consultants, Inc., shall be without liability to GROUND Engineering Consultants, Inc.

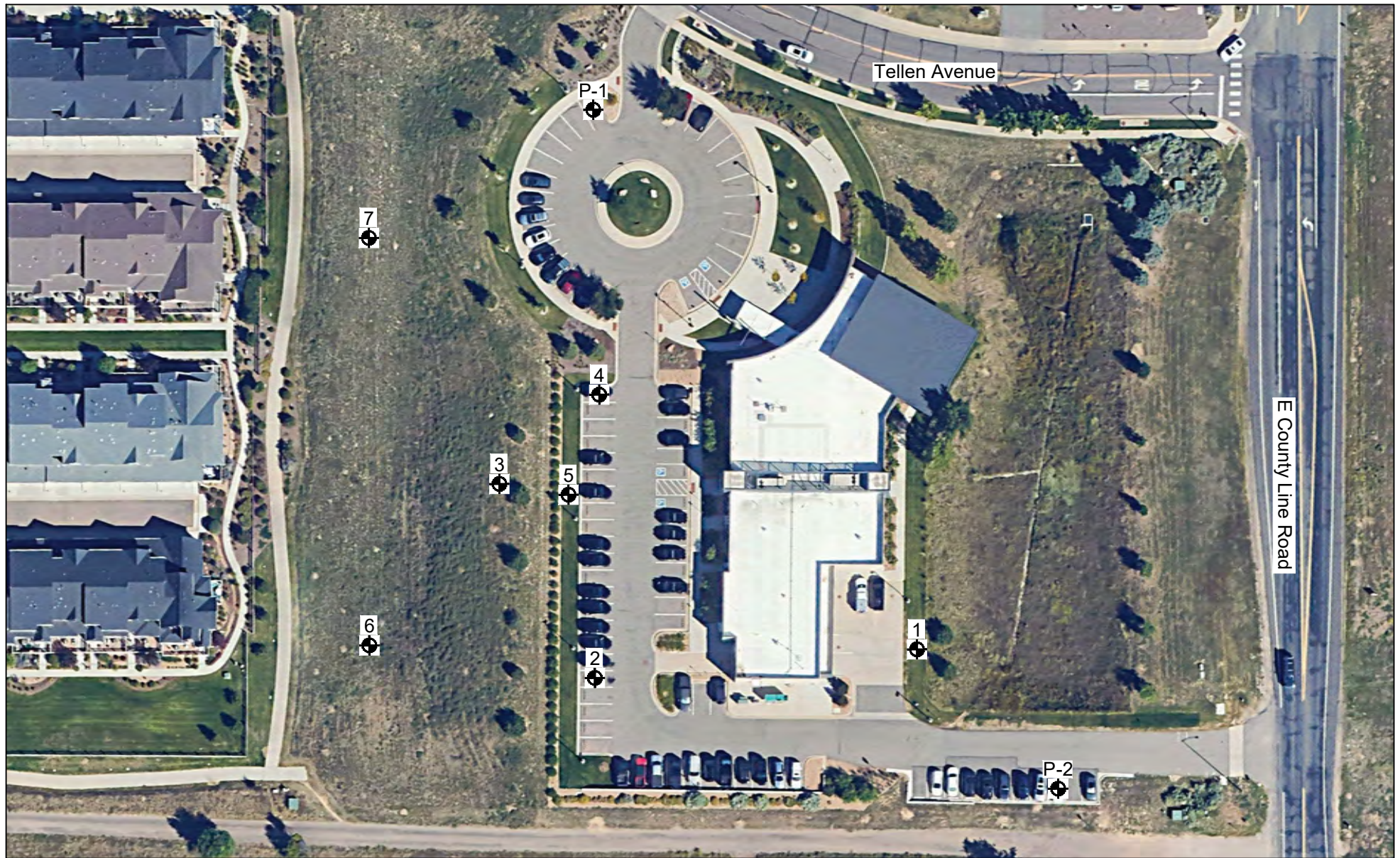
GROUND appreciates the opportunity to complete this portion of the project.

Sincerely,
GROUND Engineering Consultants, Inc.

Kelsey Van Bemmell, P.E.

Reviewed by Joseph Zorack, P.E.





GOOGLE EARTH AERIAL IMAGE (09/04/2023)

7



Indicates test hole numbers and approximate locations.



NOT TO SCALE

GROUND
ENGINEERING

JOB NO.: 25-0003

FIGURE: 1

LOCATION OF TEST HOLES

PROJECT: Erie Police Station Additions
CLIENT: Town of Erie Public Works

JOB NO: 25-0003
SITE LOCATION: Erie, CO

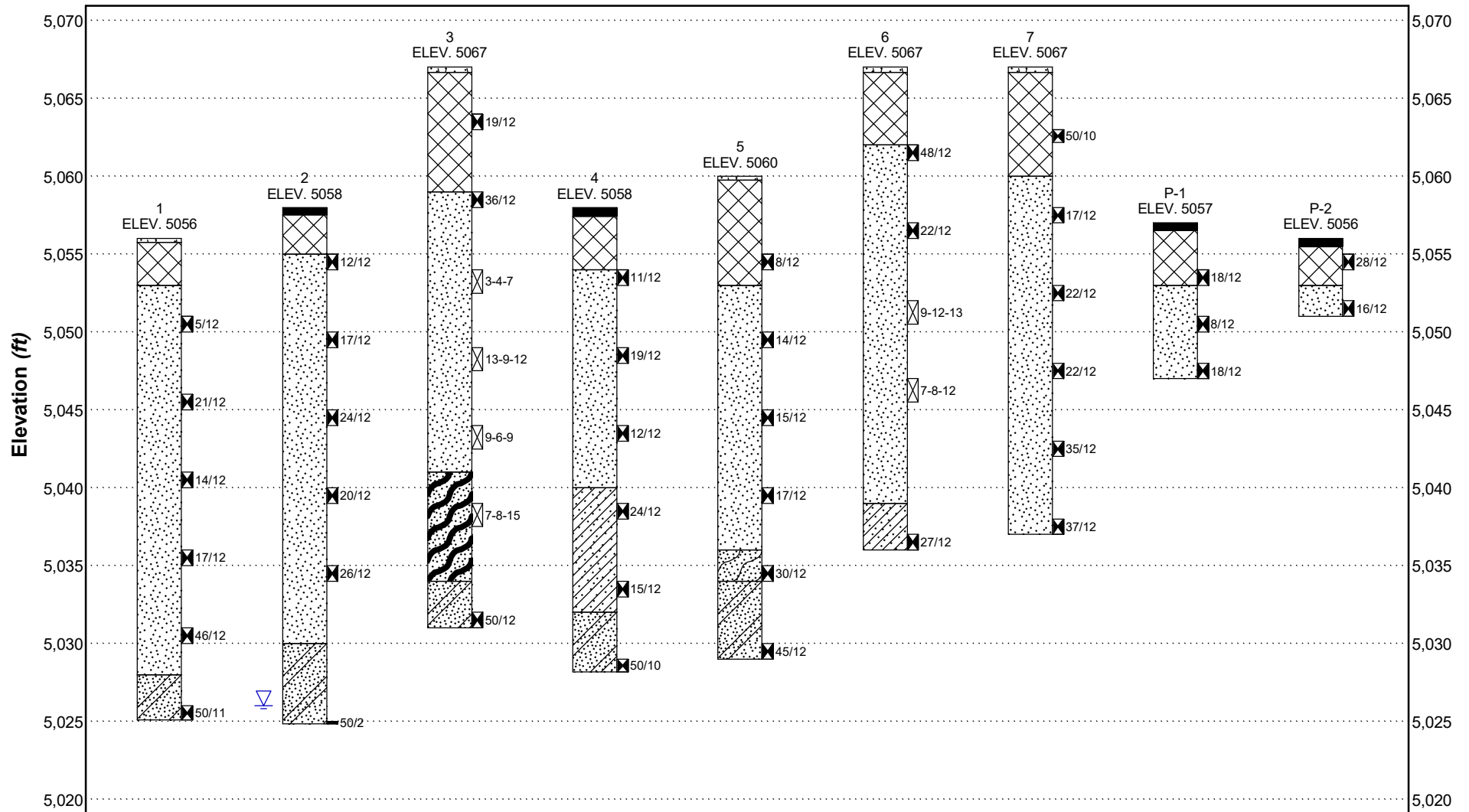


FIGURE: 2

PROJECT: Erie Police Station Additions

JOB NO: 25-0003

CLIENT: Town of Erie Public Works

SITE LOCATION: Erie, CO

MATERIAL SYMBOLS



ASPHALT



TOPSOIL



FILL



SAND



SANDY CLAY



WEATHERED CLAYSTONE and SANDSTONE



WEATHERED SANDSTONE



SANDSTONE BEDROCK

NOTE: See Detailed Logs for Material descriptions.

SAMPLER SYMBOLS



Modified California Liner Sampler

23 / 12 Drive sample blow count indicates 23 blows of a 140 pound hammer falling 30 inches were required to drive the sampler 12 inches.



Standard Penetration Test Sampler

20-25-30 Drive sample blow count, indicates 20, 25, and 30 blows of a 140 pound hammer falling 30 inches were required to drive the sampler 18 inches in three 6 inch increments.

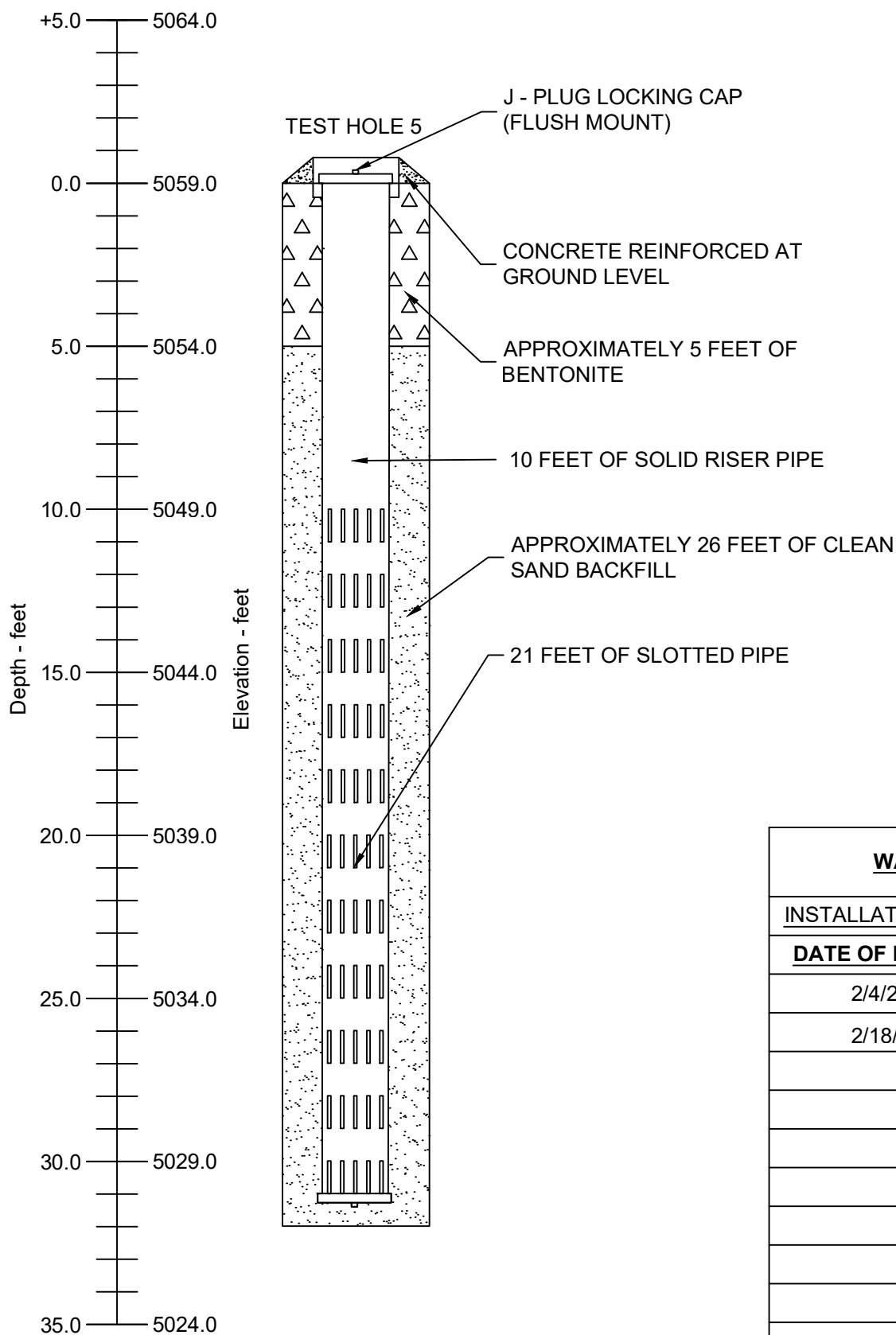
NOTES

1. Test holes were drilled on 1/27/2025 & 2/4/2025 with 4" solid stem auger.
2. Locations of the test holes were determined in the field using a hand held GPS device by GROUND.
3. Elevations of test holes were estimated from client provided documents and the logs of test holes are hung to elevation.
4. The test hole locations and elevations should be considered accurate only to the degree implied by the method used.
5. The lines between materials shown on the test hole logs represent the approximate boundaries between material types and the transitions may be gradual.
6. Groundwater level readings shown on the logs were made at the time and under the conditions indicated. Fluctuations in the water level may occur with time.
7. The material descriptions on these logs are for general classification purposes only. See full text of this report for descriptions of the site materials & related information.
8. All test holes were immediately backfilled upon completion of drilling, unless otherwise specified in this report.

ABBREVIATIONS

- ▽ Water Level at Time of Drilling, or as Shown
- ▼ Water Level at End of Drilling, or as Shown
- ▽ Water Level After 24 Hours, or as Shown

NV No Value
NP Non-Plastic



NOTES:

- Bore hole is approximately $4\frac{1}{4}$ inches in diameter.
- PVC is schedule 40 and 2 inches in diameter
slots for slotted PVC are 0.010" wide.
- Test hole elevations estimated from client provided documents and the logs of test holes are hung to elevation.

WATER READINGS

INSTALLATION DATE: 2/4/2025

DATE OF READING **DEPTH**

2/4/2025 Dry

2/18/2025 Dry

GROUND
ENGINEERING

JOB NO.: 25-0003

FIGURE: 4

**MONITORING AND OBSERVATION
HOLE ASSEMBLY**

Erie Police Station

TABLE 1: SUMMARY OF LABORATORY TEST RESULTS

Sample Location		Natural Moisture Content (%)	Natural Dry Density (pcf)	Gradation			Atterberg Limits		Swell/Consolidation		USCS Equivalent Classification	AASHTO Equivalent Classification (Group Index)	Sample Description
Test Hole No.	Depth (feet)			Gravel (%)	Sand (%)	Fines (%)	Liquid Limit	Plasticity Index	Volume Change (%)	Surcharge Pressure (psf)			
1	5	6.5	106.6	0	80	20.4	18	2	-1.3	500	SM	A-2-4 (0)	silty SAND
1	10	3.8	117.6	3	84	12.8	NV	NP	-2.3	1,250	SM	A-2-4 (0)	silty SAND
1	15	7.1	115.0	0	73	27.2	19	3	-3.2	1,750	SM	A-2-4 (0)	silty SAND
1	20	7.3	104.7	0	76	24.0	NV	NP	-3.8	2,500	SM	A-2-4 (0)	silty SAND
2	3	9.6	114.3	0	68	32.4	21	4	-0.8	500	SC-SM	A-2-4 (0)	silty/clayey SAND
2	8	9.9	114.0	0	58	42.5	25	2	-0.9	1,000	SM	A-4 (0)	silty SAND
2	13	9.0	109.4	0	69	31.3	23	5	-2.5	1,500	SC-SM	A-2-4 (0)	silty/clayey SAND
2	18	11.3	117.1	0	47	53.4	28	9	-2.9	2,000	s(CL)	A-4 (2)	SAND and CLAY
3	3	5.1	103.5	4	60	35.7	20	4	-3.4	500	SC-SM	A-4 (0)	Fill: silty/clayey SAND
3	8	8.9	118.0	1	54	44.7	23	4	-0.1	1,000	SC-SM	A-4 (0)	silty/clayey SAND
3	13	5.2	-	0	64	35.9	19	2	-	-	SM	A-4 (0)	silty
4	4	12.4	116.5	0	49	51.1	22	6	-1.2	500	s(CL-ML)	A-4 (0)	Fill: SAND and CLAY/SILT
4	9	9.4	112.8	0	58	42.1	23	9	-	-	SC	A-4 (1)	clayey SAND
4	14	7.2	106.5	0	73	26.7	20	3	-1.2	1,500	SM	A-2-4 (0)	silty SAND
4	19	15.2	112.1	0	20	79.6	32	10	-0.9	2,000	(CL)s	A-4 (7)	CLAY with sand
5	5	12.1	113.4	0	55	45.2	26	6	-0.6	750	SC-SM	A-4 (0)	Fill: silty/clayey SAND
5	10	10.2	118.2	0	61	39.2	25	6	-1.5	1,250	SC-SM	A-4 (0)	silty/clayey SAND
5	15	15.7	110.1	0	10	89.7	44	20	-0.7	1,750	CL	A-7-6 (20)	CLAY
5	20	17.3	107.9	0	21	78.8	33	12	-0.8	2,500	(CL)s	A-6 (9)	CLAY with sand
6	5	5.6	117.8	0	73	26.8	21	4	-1.5	500	SM	A-2-4 (0)	silty SAND
6	10	4.1	112.7	0	59	41.2	21	5	-2.0	1,250	SC-SM	A-4 (0)	silty/clayey SAND
6	20	8.9	-	10	49	41.2	24	8	-	-	SC	A-4 (0)	clayey SAND
7	4	6.7	124.0	7	53	40.5	21	6	1.6	500	SC-SM	A-4 (0)	Fill: silty/clayey SAND
7	9	5.4	109.6	0	68	32.0	20	4	-2.0	1,000	SC-SM	A-2-4 (0)	silty/clayey SAND
7	14	5.1	104.0	0	71	29.3	18	2	-3.1	1,500	SM	A-2-4 (0)	silty SAND
7	19	6.3	107.1	0	62	38.5	19	3	-3.4	2,000	SM	A-4 (0)	silty SAND
P-1	3	10.5	119.6	1	66	33.2	21	5	-	-	SC-SM	A-2-4 (0)	silty/clayey SAND

SD = Sample disturbed, NV = No value, NP = Non-plastic

Job No. 25-0003

Erie Police Station

TABLE 2: SUMMARY OF SOIL CORROSION TEST RESULTS

Sample Location		Water-Soluble Sulfates (%)	pH	Redox Potential (mv)	Sulfide Reactivity	Resistivity (ohm-cm)	USCS Equivalent Classification	AASHTO Equivalent Classification (Group Index)	Sample Description
Test Hole No.	Depth (feet)								
4	9	0.02	8.2	- 68	Trace	4,000	SC	A-4 (1)	clayey SAND

Job No. 25-0003

Appendix A

Detailed Test Hole Logs

PROJECT: Erie Police Station Additions

JOB NO: 25-0003

CLIENT: Town of Erie Public Works

SITE LOCATION: Erie, CO

Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits		Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
5056	0								Liquid Limit	Plasticity Index			
			TOPSOIL: Approximately 3 inches of topsoil.										
			FILL: Silty / clayey sand and were low to non-plastic, fine to medium grained, dry to slightly moist, and brown in color with localized caliche.										
5051	5		SAND: Ranged from silty to clayey, were fine to medium grained, low to non- plastic, dry to slightly moist, loose to dense, and brown in color.										
			<i>Roots noted in sample at 5 feet.</i>										
5046	10				5/12	6.5	106.6	20	18	2	-1.3 (500)		SM
5041	15				21/12	3.8	117.6	13	NV	NP	-2.3 (1250)		SM
5036	20				14/12	7.1	115	27	19	3	-3.2 (1750)		SM
5031	25				17/12	7.3	104.7	24	NV	NP	-3.8 (2500)		SM
					46/12								
5026	30		SANDSTONE BEDROCK: Materials were non-plastic, fine grained, slightly moist, hard to very hard, and orange-brown to gray-brown in color with iron staining.										
					50/11								

Bottom of test hole at approx. 30.92 feet.

PROJECT: Erie Police Station Additions

JOB NO: 25-0003

CLIENT: Town of Erie Public Works

SITE LOCATION: Erie, CO

Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits		Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
									Liquid Limit	Plasticity Index			
5058	0		ASPHALT: Approximately 6 inches of asphalt.										
			FILL: Silty / clayey sand and were low to non-plastic, fine to medium grained, dry to slightly moist, and brown in color with localized caliche.										
				12/12	9.6	114.3	32	21	4	-0.8 (500)			SC-SM
5053	5		SAND: Ranged from silty to clayey, were fine to medium grained, low to non- plastic, dry to slightly moist, loose to dense, and brown in color.										
				17/12	9.9	114	42	25	2	-0.9 (1000)			SM
5048	10												
				24/12	9	109.4	31	23	5	-2.5 (1500)			SC-SM
5043	15												
				20/12	11.3	117.1	53	28	9	-2.9 (2000)			s(CL)
5038	20												
				26/12									
5033	25												
5028	30		SANDSTONE BEDROCK: Materials were non-plastic, fine grained, slightly moist, hard to very hard, and orange-brown to gray-brown in color with iron staining.										
			High resistance encountered from 28 feet to termination depth.										
			Groundwater encountered at 32 feet at the time of drilling.										

Bottom of test hole at approx. 33.17 feet.

50/2

PROJECT: Erie Police Station Additions

JOB NO: 25-0003

CLIENT: Town of Erie Public Works

SITE LOCATION: Erie, CO

Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits	Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
5067	0		TOPSOIL: Approximately 4 inches of topsoil.									
			FILL: Silty / clayey sand and were low to non-plastic, fine to medium grained, dry to slightly moist, and brown in color with localized caliche.									
				▲	19/12	5.1	103.5	36	20	4	-3.4 (500)	SC-SM
5062	5											
			SAND: Ranged from silty to clayey, were fine to medium grained, low to non-plastic, dry to slightly moist, loose to dense, and brown in color.	▲	36/12	8.9	118	45	23	4	-0.1 (1000)	SC-SM
5057	10											
				×	3-4-7	5.2		36	19	2		SM
5052	15											
				×	13-9-12							
5047	20											
				×	9-6-9							
5042	25											
			WEATHERED CLAYSTONE / SANDSTONE: Was interbedded, fine grained, non to highly plastic, moist, and brown in color.	×	7-8-15							
5037	30											
			SANDSTONE BEDROCK: Materials were non-plastic, fine grained, slightly moist, hard to very hard, and orange-brown to gray-brown in color with iron staining.	▲	50/12							
5032	35											

Bottom of test hole at approx. 36 feet.

PROJECT: Erie Police Station Additions

JOB NO: 25-0003

CLIENT: Town of Erie Public Works

SITE LOCATION: Erie, CO

Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits		Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
									Liquid Limit	Plasticity Index			
5058	0		ASPHALT: Approximately 7 inches of asphalt.										
			FILL: Silty / clayey sand and were low to non-plastic, fine to medium grained, dry to slightly moist, and brown in color with localized caliche.										
5053	5		SAND: Ranged from silty to clayey, were fine to medium grained, low to non- plastic, dry to slightly moist, loose to dense, and brown in color.	✖	11/12	12.4	116.5	51	22	6	-1.2 (500)		s(CL-ML)
5048	10			✖	19/12	9.4	112.8	42	23	9			SC
5043	15			✖	12/12	7.2	106.5	27	20	3	-1.2 (1500)		SM
5038	20		SANDY CLAY: Was fine to medium grained, low to medium plastic, moist to very moist, stiff to very stiff, and brown with local iron staining.	✖	24/12	15.2	112.1	80	32	10	-0.9 (2000)		(CL)s
5033	25			✖	15/12								
			SANDSTONE BEDROCK: Materials were non-plastic, fine grained, slightly moist, hard to very hard, and orange-brown to gray-brown in color with iron staining.										
				✖	50/10								

Bottom of test hole at approx. 29.83 feet.

PROJECT: Erie Police Station Additions

JOB NO: 25-0003

CLIENT: Town of Erie Public Works

SITE LOCATION: Erie, CO

Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits		Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
									Liquid Limit	Plasticity Index			
5060	0		TOPSOIL: Approximately 3 inches of topsoil.										
			FILL: Silty / clayey sand and were low to non-plastic, fine to medium grained, dry to slightly moist, and brown in color with localized caliche.										
5055	5			8/12	12.1	113.4	45	26	6		-0.6 (750)		SC-SM
			SAND: Ranged from silty to clayey, were fine to medium grained, low to non- plastic, dry to slightly moist, loose to dense, and brown in color.										
5050	10			14/12	10.2	118.2	39	25	6		-1.53 (1250)		SC-SM
5045	15			15/12	15.7	110.1	90	44	20		-0.73 (1750)		CL
5040	20			17/12	17.3	107.9	79	33	12		-0.8 (2500)		(CL)s
5035	25		WEATHERED SANDSTONE: Materials were non-plastic, fine grained, slightly moist, and brown in color with iron staining.	30/12									
			SANDSTONE BEDROCK: Materials were non-plastic, fine grained, slightly moist, hard to very hard, and orange-brown to gray-brown in color with iron staining.										
5030	30			45/12									

Bottom of test hole at approx. 31 feet.

PROJECT: Erie Police Station Additions

JOB NO: 25-0003

CLIENT: Town of Erie Public Works

SITE LOCATION: Erie, CO

Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits		Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
									Liquid Limit	Plasticity Index			
5067	0		TOPSOIL: Approximately 4 inches of topsoil.										
			FILL: Silty / clayey sand and were low to non-plastic, fine to medium grained, dry to slightly moist, and brown in color with localized caliche.										
5062	5												
			SAND: Ranged from silty to clayey, were fine to medium grained, low to non- plastic, dry to slightly moist, loose to dense, and brown in color.										
5057	10												
5052	15												
5047	20												
5042	25												
5037	30		SANDY CLAY: Was fine to medium grained, low to medium plastic, moist to very moist, stiff to very stiff, and brown with local iron staining.										

Bottom of test hole at approx. 31 feet.

PROJECT: Erie Police Station Additions

JOB NO: 25-0003

CLIENT: Town of Erie Public Works

SITE LOCATION: Erie, CO

Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits		Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
									Liquid Limit	Plasticity Index			
5067	0		TOPSOIL: Approximately 4 inches of topsoil.										
			FILL: Silty / clayey sand and were low to non-plastic, fine to medium grained, dry to slightly moist, and brown in color with localized caliche.										
5062	5			50/10	6.7	124	40	21	6		1.6 (500)		SC-SM
5057	10		SAND: Ranged from silty to clayey, were fine to medium grained, low to non- plastic, dry to slightly moist, loose to dense, and brown in color.	17/12	5.4	109.6	32	20	4		-2 (1000)		SC-SM
5052	15			22/12	5.1	104	29	18	2		-3.1 (1500)		SM
5047	20			22/12	6.3	107.1	38	19	3		-3.4 (2000)		SM
5042	25			35/12									
5037	30			37/12									

Bottom of test hole at approx. 30 feet.

PROJECT: Erie Police Station Additions

JOB NO: 25-0003

CLIENT: Town of Erie Public Works

SITE LOCATION: Erie, CO

Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits		Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
									Liquid Limit	Plasticity Index			
5057	0		ASPHALT: Approximately 6 inches of asphalt.										
			FILL: Silty / clayey sand and were low to non-plastic, fine to medium grained, dry to slightly moist, and brown in color with localized caliche.										
					18/12	10.5	119.6	33	21	5			SC-SM
5052	5		SAND: Ranged from silty to clayey, were fine to medium grained, low to non- plastic, dry to slightly moist, loose to dense, and brown in color.										
					8/12								
5047	10				18/12								

Bottom of test hole at approx. 10 feet.

PROJECT: Erie Police Station Additions

JOB NO: 25-0003

CLIENT: Town of Erie Public Works

SITE LOCATION: Erie, CO

Elevation (ft)	Depth (ft)	Graphic Log	Material Descriptions and Drilling Notes	Sample Type	Blow Count	Natural Moisture Content (%)	Natural Dry Density (pcf)	Percent Passing No. 200 Sieve	Atterberg Limits		Swell/Consolidation (%) at Surcharge Pressure (psf)	Unconfined Compressive Strength (ksf)	USCS Equivalent Classification
									Liquid Limit	Plasticity Index			
5056	0		ASPHALT: Approximately 6.5 inches of asphalt.										
			FILL: Silty / clayey sand and were low to non-plastic, fine to medium grained, dry to slightly moist, and brown in color with localized caliche.		28/12								
5051	5		SAND: Ranged from silty to clayey, were fine to medium grained, low to non- plastic, dry to slightly moist, loose to dense, and brown in color.		16/12								

Bottom of test hole at approx. 5 feet.

Appendix B

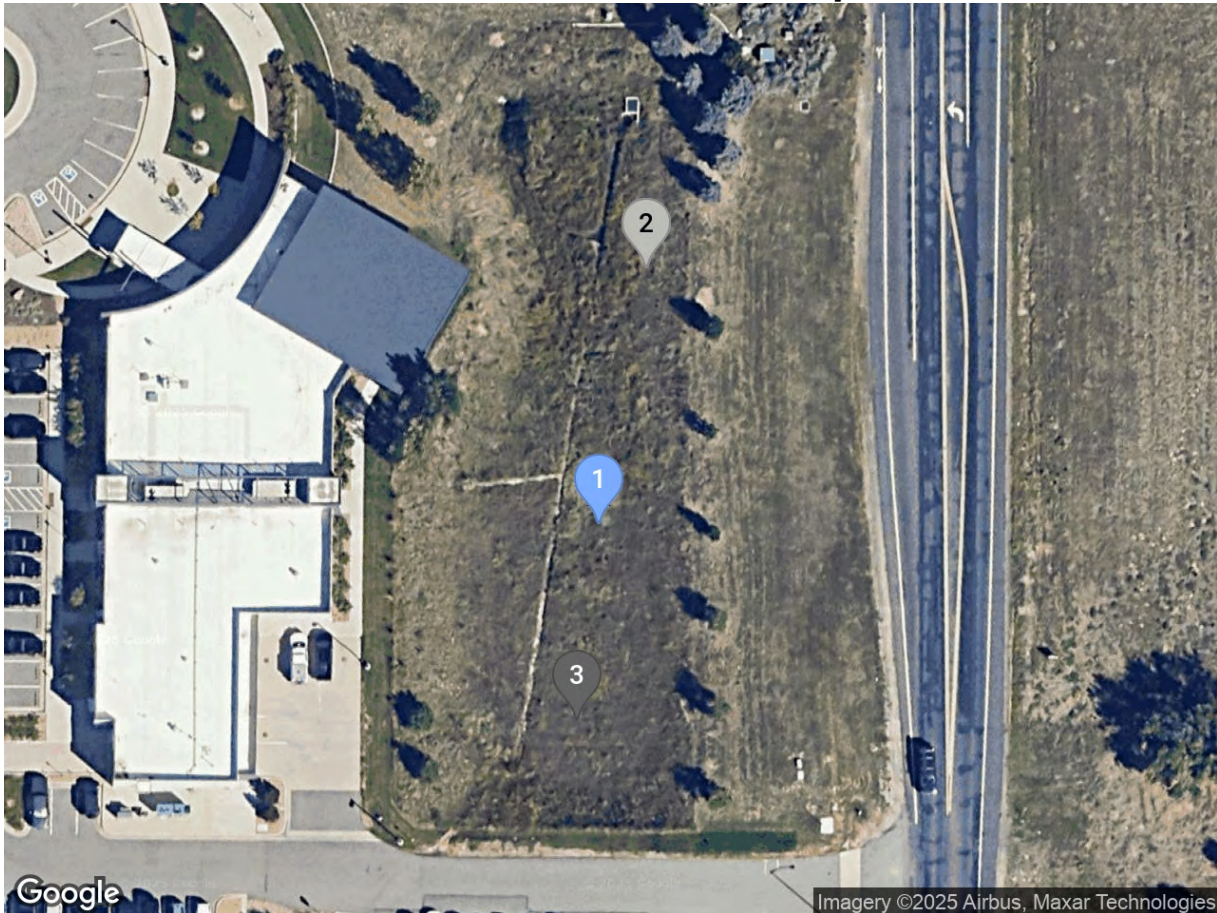
MPD – Infiltration Data

Infiltration Report

GROUND Engineering Consultants, Inc.
Erie Police Station - 25-0003 -

K_{sat} best-fit site average: 45 mm/hr or 1.78 in/hr

GPS Infiltration Test Site Map



Map Pin #	Test #	Test Name	Ksat (mm/hr)	Ksat (in/hr)	C (mm)	RMS Error of Regression (s)	Normalized RMS
1	28564	MPDCentral	29	1.15	-44.4	1.7	0.04%
2	28565	MPDNorth	31	1.21	-342.8	7.8	0.2%
3	28566	MPDSouth	90	3.53	-171.1	15.8	0.7%

Infiltration Report

GROUND Engineering Consultants, Inc.

Erie Police Station - 25-0003 -

This report summarizes the results of a set of Modified Philip Dunne (MPD) Infiltrometer tests performed at the above referenced site. GROUND Engineering Consultants, Inc. personnel performed the field tests. The software used to compute saturated hydraulic conductivity (K_{sat}) and generate this report assumes that the field personnel used infiltrmeters manufactured by Upstream Technologies Inc. and followed the procedures outlined in "Manual – Modified Philip - Dunne Infiltrometer" by Ahmed, Gulliver, and Nieber.

The following paragraphs describe the individual tests, input values used in the analysis, and methods used to compute the K_{sat} value.

After individual K_{sat} values were calculated, the method used to determine the overall site K_{sat} value ($K_{best-fit}$) is described in "Effective Saturated Hydraulic Conductivity of an Infiltration-Based Stormwater Control Measure" by Weiss and Gulliver 2015, "A relationship to more consistently and accurately predict the best-fit value of saturated hydraulic conductivity used a weighted sum of 0.32 times the arithmetic mean and 0.68 times the geometric mean."

METHOD USED TO COMPUTE K_{sat}

The MPD Infiltrometer software uses the following procedure described in "The Comparison of Infiltration Devices and Modification of the Philip-Dunne Permeameter for the Assessment of Rain Gardens" by Rebecca Nestigen, University of Minnesota, November 2007.

The steps are as follows:

1. For each measurement of head, use the following equation to find the corresponding distance to the sharp wetting front.

$$[H_0 - H(t)]r_1^2 = \frac{\theta_1 - \theta_2}{3} [2[R(t)]^3 + 3[R(t)]^2 L_{max} - L_{max}^3 - 4r_0^3]$$

2. Estimate the change in head with respect to time and the change in wetting front distance with respect to time by using the backward difference for all values of $R(t)$ equal to or greater than the distance

$$\sqrt{r_1^2 + L_{max}^2}$$

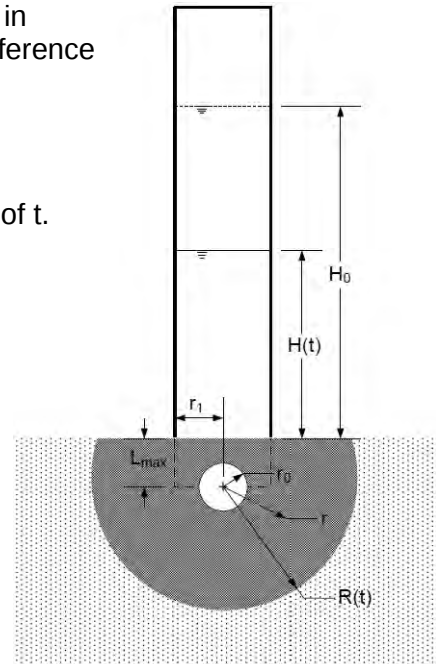
3. Make initial guesses for K and C .

4. Solve the following equations for $\Delta P(t)$ at each incremental value of t .

$$\Delta P(t) = \frac{\pi^2}{8} \left\{ \theta_1 - \theta_0 \frac{[R(t)]^2 + [R(t)]L_{max}}{K} \frac{dr}{dt} - 2r_0^2 \right\} \frac{\ln \left[\frac{R(t)[r_0 + L_{max}]}{r_0[R(t) + L_{max}]} \right]}{L_{max}}$$

$$\Delta P(t) = C - H(t) - L_{max} + \frac{L_{max}}{K} \frac{dh}{dt}$$

5. Minimize the absolute difference between the two solutions found in Step 4 by adjusting the values of K and C .



Parameters for Equations

Θ_0 = volumetric water content of soil before MPD test

Θ_1 = volumetric water content of soil after MPD test

Infiltration Report

GROUND Engineering Consultants, Inc.

Erie Police Station - 25-0003 -

MPDCentral

Date	3/12/2025
Time	8:01 AM
Latitude	40.044374
Longitude	-105.055990
Initial Volumetric Moisture	20.00 %
Final Volumetric Moisture	25.00 %
Cylinder Size	3 Liter

MPDCentral Results

Map Pin #	1
Test Number	30187
Ksat - mm/hr	29
Ksat - in/hr	1.15
Capillary Pressure C mm	-44.4
RMS Error of Regression	1.7
Normalized RMS	0.04%

Readings

#	Time	Head	#	Time	Head	#	Time	Head	#	Time	Head
1	0 s	37.48 cm	26	1500 s	30.8 cm	51	3000 s	25.29 cm	76	4500 s	20.71 cm
2	60 s	37.21 cm	27	1560 s	30.55 cm	52	3060 s	25.09 cm	77	4560 s	20.53 cm
3	120 s	36.94 cm	28	1620 s	30.32 cm	53	3120 s	24.89 cm	78	4620 s	20.35 cm
4	180 s	36.67 cm	29	1680 s	30.07 cm	54	3180 s	24.69 cm	79	4680 s	20.18 cm
5	240 s	36.41 cm	30	1740 s	29.84 cm	55	3240 s	24.49 cm	80	4740 s	19.99 cm
6	300 s	36.14 cm	31	1800 s	29.6 cm	56	3300 s	24.3 cm	81	4800 s	19.81 cm
7	360 s	35.88 cm	32	1860 s	29.37 cm	57	3360 s	24.11 cm	82	4860 s	19.62 cm
8	420 s	35.59 cm	33	1920 s	29.15 cm	58	3420 s	23.92 cm	83	4920 s	19.44 cm
9	480 s	35.31 cm	34	1980 s	28.91 cm	59	3480 s	23.72 cm	84	4980 s	19.26 cm
10	540 s	35.02 cm	35	2040 s	28.69 cm	60	3540 s	23.54 cm	85	5040 s	19.08 cm
11	600 s	34.74 cm	36	2100 s	28.46 cm	61	3600 s	23.35 cm	86	5100 s	18.89 cm
12	660 s	34.45 cm	37	2160 s	28.25 cm	62	3660 s	23.17 cm	87	5160 s	18.71 cm
13	720 s	34.17 cm	38	2220 s	28.03 cm	63	3720 s	22.99 cm			
14	780 s	33.9 cm	39	2280 s	27.81 cm	64	3780 s	22.8 cm			
15	840 s	33.62 cm	40	2340 s	27.59 cm	65	3840 s	22.63 cm			
16	900 s	33.34 cm	41	2400 s	27.38 cm	66	3900 s	22.45 cm			
17	960 s	33.08 cm	42	2460 s	27.15 cm	67	3960 s	22.28 cm			
18	1020 s	32.8 cm	43	2520 s	26.94 cm	68	4020 s	22.09 cm			
19	1080 s	32.54 cm	44	2580 s	26.73 cm	69	4080 s	21.92 cm			
20	1140 s	32.28 cm	45	2640 s	26.53 cm	70	4140 s	21.75 cm			
21	1200 s	32.03 cm	46	2700 s	26.31 cm	71	4200 s	21.58 cm			
22	1260 s	31.78 cm	47	2760 s	26.1 cm	72	4260 s	21.41 cm			
23	1320 s	31.53 cm	48	2820 s	25.9 cm	73	4320 s	21.24 cm			
24	1380 s	31.29 cm	49	2880 s	25.69 cm	74	4380 s	21.07 cm			
25	1440 s	31.04 cm	50	2940 s	25.49 cm	75	4440 s	20.89 cm			

Infiltration Report

GROUND Engineering Consultants, Inc.

Erie Police Station - 25-0003 -

MPDNorth

Date	3/12/2025
Time	8:10 AM
Latitude	40.044650
Longitude	-105.055923
Initial Volumetric Moisture	18.00 %
Final Volumetric Moisture	21.00 %
Cylinder Size	3 Liter

MPDNorth Results

Map Pin #	2
Test Number	30188
Ksat - mm/hr	31
Ksat - in/hr	1.21
Capillary Pressure C mm	-342.8
RMS Error of Regression	7.8
Normalized RMS	0.2%

Readings

#	Time	Head	#	Time	Head	#	Time	Head
1	0 s	35.5 cm	26	1500 s	24.41 cm	51	3000 s	15.28 cm
2	60 s	34.97 cm	27	1560 s	24.02 cm	52	3060 s	14.95 cm
3	120 s	34.47 cm	28	1620 s	23.65 cm	53	3120 s	14.62 cm
4	180 s	33.97 cm	29	1680 s	23.28 cm	54	3180 s	14.29 cm
5	240 s	33.49 cm	30	1740 s	22.9 cm	55	3240 s	13.96 cm
6	300 s	33.0 cm	31	1800 s	22.54 cm	56	3300 s	13.64 cm
7	360 s	32.51 cm	32	1860 s	22.18 cm	57	3360 s	13.32 cm
8	420 s	32.04 cm	33	1920 s	21.83 cm	58	3420 s	13.0 cm
9	480 s	31.57 cm	34	1980 s	21.46 cm	59	3480 s	12.69 cm
10	540 s	31.12 cm	35	2040 s	21.06 cm	60	3540 s	12.37 cm
11	600 s	30.66 cm	36	2100 s	20.69 cm	61	3600 s	12.06 cm
12	660 s	30.21 cm	37	2160 s	20.31 cm	62	3660 s	11.76 cm
13	720 s	29.76 cm	38	2220 s	19.92 cm	63	3720 s	11.46 cm
14	780 s	29.32 cm	39	2280 s	19.55 cm	64	3780 s	11.16 cm
15	840 s	28.87 cm	40	2340 s	19.18 cm	65	3840 s	10.86 cm
16	900 s	28.44 cm	41	2400 s	18.8 cm	66	3900 s	10.56 cm
17	960 s	28.02 cm	42	2460 s	18.43 cm	67	3960 s	10.26 cm
18	1020 s	27.6 cm	43	2520 s	18.07 cm	68	4020 s	9.98 cm
19	1080 s	27.17 cm	44	2580 s	17.71 cm	69	4080 s	9.69 cm
20	1140 s	26.77 cm	45	2640 s	17.36 cm	70	4140 s	9.41 cm
21	1200 s	26.37 cm	46	2700 s	17.0 cm	71	4200 s	9.12 cm
22	1260 s	25.97 cm	47	2760 s	16.65 cm	72	4260 s	8.85 cm
23	1320 s	25.57 cm	48	2820 s	16.3 cm			
24	1380 s	25.18 cm	49	2880 s	15.96 cm			
25	1440 s	24.79 cm	50	2940 s	15.62 cm			

Infiltration Report

GROUND Engineering Consultants, Inc.
Erie Police Station - 25-0003 -

MPDSouth

Date	3/12/2025
Time	7:55 AM
Latitude	40.044162
Longitude	-105.056021
Initial Volumetric Moisture	16.00 %
Final Volumetric Moisture	17.00 %
Cylinder Size	3 Liter

MPDSouth Results

Map Pin #	3
Test Number	30189
Ksat - mm/hr	90
Ksat - in/hr	3.53
Capillary Pressure C mm	-171.1
RMS Error of Regression	15.8
Normalized RMS	0.7%

Readings

#	Time	Head	#	Time	Head	#	Time	Head	#	Time	Head
1	0 s	33.37 cm	26	749 s	22.25 cm	51	1499 s	14.02 cm	76	2249 s	6.88 cm
2	29 s	32.84 cm	27	779 s	21.88 cm	52	1529 s	13.73 cm	77	2279 s	6.64 cm
3	59 s	32.31 cm	28	809 s	21.51 cm	53	1559 s	13.45 cm	78	2309 s	6.41 cm
4	89 s	31.8 cm	29	839 s	21.14 cm	54	1589 s	13.17 cm	79	2339 s	6.18 cm
5	119 s	31.29 cm	30	869 s	20.77 cm	55	1619 s	12.88 cm			
6	149 s	30.79 cm	31	899 s	20.42 cm	56	1649 s	12.62 cm			
7	179 s	30.31 cm	32	929 s	20.06 cm	57	1679 s	12.34 cm			
8	209 s	29.83 cm	33	959 s	19.71 cm	58	1709 s	12.07 cm			
9	239 s	29.36 cm	34	989 s	19.36 cm	59	1739 s	11.8 cm			
10	269 s	28.89 cm	35	1019 s	19.02 cm	60	1769 s	11.53 cm			
11	299 s	28.43 cm	36	1049 s	18.68 cm	61	1799 s	11.25 cm			
12	329 s	27.98 cm	37	1079 s	18.35 cm	62	1829 s	10.99 cm			
13	359 s	27.54 cm	38	1109 s	18.02 cm	63	1859 s	10.71 cm			
14	389 s	27.1 cm	39	1139 s	17.69 cm	64	1889 s	10.43 cm			
15	419 s	26.66 cm	40	1169 s	17.37 cm	65	1919 s	10.17 cm			
16	449 s	26.24 cm	41	1199 s	17.05 cm	66	1949 s	9.89 cm			
17	479 s	25.82 cm	42	1229 s	16.73 cm	67	1979 s	9.62 cm			
18	509 s	25.41 cm	43	1259 s	16.42 cm	68	2009 s	9.35 cm			
19	539 s	24.99 cm	44	1289 s	16.11 cm	69	2039 s	9.08 cm			
20	569 s	24.59 cm	45	1319 s	15.8 cm	70	2069 s	8.8 cm			
21	599 s	24.19 cm	46	1349 s	15.49 cm	71	2099 s	8.54 cm			
22	629 s	23.8 cm	47	1379 s	15.19 cm	72	2129 s	8.26 cm			
23	659 s	23.39 cm	48	1409 s	14.9 cm	73	2159 s	7.99 cm			
24	689 s	23.01 cm	49	1439 s	14.6 cm	74	2189 s	7.73 cm			
25	719 s	22.63 cm	50	1469 s	14.31 cm	75	2219 s	7.1 cm			