

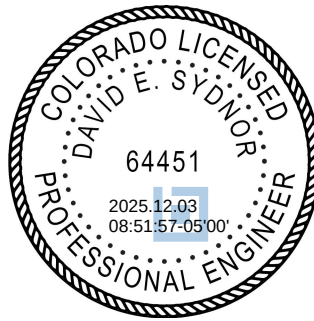
GEOTECHNICAL ENGINEERING REPORT

Elmwood Lane and West 79th Place PRV Station
Thornton, Colorado

Prepared for:

City of Thornton

Thornton, Colorado



December 2025

Olsson Project No. 024-03983





December 3, 2025

City of Thornton
Attn: Mark Mazzucco
9500 Civic Center Drive
Thornton, CO 80229

RE: Geotechnical Engineering Report
Elmwood Lane and West 79th Place PRV Station
Thornton, Colorado
Olsson Project No. 024-03983

Dear Mr. Mazzucco:

Olsson, Inc. has completed the authorized geotechnical exploration for the above-referenced project. The geotechnical exploration was conducted to evaluate the subsurface conditions with respect to design and construction of the project. The enclosed report summarizes the project characteristics as we understand them, presents the findings of the exploration and laboratory testing, discusses the observed subsurface conditions, and provides our geotechnical engineering recommendations.

We appreciate the opportunity to provide our geotechnical engineering services for this project and are prepared to provide construction phase services as well. If you have any questions or need further assistance, please contact us at your convenience.

Respectfully submitted,
Olsson, Inc.

A handwritten signature in blue ink, appearing to read "DSYD", is positioned above the printed name of David Sydnor.

David Sydnor, PE
Engineer

dsydnor@olsson.com

A handwritten signature in blue ink, appearing to read "Tyler A. Compton", is positioned above the printed name of Tyler A. Compton.

Tyler A. Compton, PE
Senior Geotechnical Engineer

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EXECUTIVE SUMMARY

In October of 2025, Olsson conducted a geotechnical exploration to assess subsurface conditions and offer geotechnical recommendations for a pressure reducing valve (PRV) station, located southeast of the intersection of Elmwood Lane and West 79th Place in Thornton, Colorado. This exploration involved drilling one geotechnical boring to a depth of 25 feet to provide subsurface insights and earthwork recommendations for the proposed structure.

Based on the findings from the geotechnical exploration, and our analyses, the site is generally suitable for the proposed construction. Surficial lean clay and clayey sand soils were encountered to a depth of 3.5 feet overlying claystone bedrock extending to the termination depth of our boring. The surficial clay and sand appear suitable for reuse as structural fill at the site, however we do not anticipate they will be encountered in appreciable quantities and import fill will be required. Claystone bedrock is not suitable for reuse as structural fill.

This Executive Summary provides a limited overview of the report and is subject to any and all clarifications, conditions, contingencies, limitations and/or qualifications that may exist in the body of the report. The Client nor any other party may rely solely on this Executive Summary. Client and any other party using this report must review the entire report and interpret the information contained in this Executive Summary in conjunction with the remainder of the report.

1. PROJECT UNDERSTANDING

1.1 Project Information

We understand the proposed project consists of the replacement of an existing pressure reducing valve (PRV) station. The new proposed PRV station will bear on a 12-foot by 12-foot concrete pad at a depth of 13 to 15 feet below existing site grades.

If the foundation size or site grading exceed these values, the geotechnical engineer should be contacted to verify that the recommendations contained in this report remain valid.

The need for and/or location(s) of potential retaining walls at the site have not been determined. The lateral earth pressures and any related allowable soil bearing pressure recommendations provided in this report are applicable only to the design of rigid below-grade or retaining walls subject to slight rotation, such as cast-in-place cantilever concrete walls.

1.2 Site Description

The proposed project site is located southeast of the intersection of Elmwood Lane and West 79th Place in Thornton, Colorado. At the time of our field exploration, the site surface comprised grass and asphalt sidewalks and was accessible to a truck-mounted drilling rig.

From our review of readily available historic aerial images dating back to 1985, the project area has remained undeveloped with the exception of existing utilities in the vicinity of the proposed PRV station. Residential dwellings surround the project site to the north, west, and south, and the Western Hills water tank is located to the east of the proposed PRV station. The approximate location of the new PRV station is shown in **Figure 1**.



Figure 1. 2023 Aerial Photograph/Project Location Plan.

2. SUBSURFACE CONDITIONS

2.1 Subsurface Profile

The appended borehole reports represent subsurface conditions at the specific boring location at the time of our field exploration; variations may occur between or beyond the boring. The stratification lines shown on the log represents the approximate boundary between material types. However, the transition between layers may be gradual. The depths referenced in the following paragraphs are relative to the site grade at the time of our exploration.

The subsurface soils at this site comprised lean clay and clayey sand overlying claystone bedrock. The general characteristics of each soil stratum are summarized below, with more detailed descriptions provided on the borehole report in [Appendix A](#).

Surfacing and Topsoil

We encountered a 6-inch-thick topsoil zone in the boring.

Clay Soils

We encountered clay soil underlying the topsoil and extending to a depth of 1.5 feet below existing site grades. The clay soil comprised lean clay with sand (CL) and was described as very stiff, dark brown, and moist.

Sand Soils

We encountered sand soil underlying the lean clay and extending to a depth of 3.5 feet below existing site grades. The sand soil comprised clayey sand (SC) and was described as dense, light brown, and moist.

Bedrock

We encountered claystone bedrock underlying the site soils and extending to the termination depth of the boring. The claystone was generally described as highly weathered, dark gray to gray, and very moist.

2.2 Water Level Observations

Subsurface water was not encountered in the soil test borings at the time of drilling operations. However, water levels will fluctuate over time with variations in precipitation, site grading, drainage, and adjacent land use. Perched subsurface water conditions can also develop in seams of loose or granular soil, or within seams in the claystone bedrock.

Long-term monitoring with piezometers generally provides a more representative indication of the potential range of subsurface water conditions. Such monitoring was not completed as part

of this exploration. Olsson can provide additional monitoring upon written request of the Owner and/or Olsson's client. Recommendations for addressing effects of water in design and during construction are presented in **Section 4.3** of this report. Any foundation design must take into consideration changing water conditions.

3. GEOTECHNICAL CONSIDERATIONS

We encountered weathered claystone bedrock at a depth of 3.5 feet in our boring. Based on anticipated embedment depth of 13 to 15 feet, excavations into bedrock will be necessary at this site. Excavations into bedrock can require additional time and expense and may require specialized equipment and/or techniques. The contractor should be prepared to utilize such techniques in areas where excavations extend into bedrock. Excavations into highly weathered bedrock can typically be accomplished with excavation equipment that is in good working order and properly sized and equipped for difficult excavations.

Based on the results of our laboratory testing, the claystone exhibited low swelling properties. In our experience, claystone bedrock in the project vicinity can have moderate to high expansive potentials. The resulting magnitude of volume change for these potentially expansive materials depends on various factors including composition, in-situ moisture content, in-situ density, changes in moisture content, and changes in applied loads. The presence of potentially expansive materials increases the risk of post-construction movements and associated distress. Based on our understanding of the project and encountered subsurface conditions, foundations will bear within the potentially expansive claystone and foundations could experience movements greater than 1 inch. Remedial earthwork consisting of overexcavation and replacement of the claystone with suitable compacted structural fill is recommended to reduce estimated foundation movements to approximately 1 inch.

Laboratory testing indicates the native lean clay and clayey sand soils appear suitable for reuse as structural fill, however these soils are not expected to be encountered in appreciable quantities, and the contractor should plan to import structural fill that meets the requirements outlined in **Section 4.2**. Claystone bedrock is not suitable for reuse as structural fill.

Based on the results of the soil chemistry testing, sulfate exposure class S0 appears appropriate for below grade concrete components.

4. EARTHWORK AND CONSTRUCTION CONSIDERATIONS

The following sections discuss general earthwork and construction considerations for overall site preparation. Additional earthwork or soil improvements may be required beneath foundations. These additional recommendations, if applicable, are presented in [Section 5](#).

4.1 Site Preparation

4.1.1 General

Vegetation, topsoil, roots, and other deleterious materials deemed unsuitable by an Olsson geotechnical engineer or their authorized field representative should be removed from the proposed construction area and replaced with controlled fill. We recommend site clearing, grubbing, and stripping be performed during dry weather conditions. Operation of heavy equipment on the site during wet conditions could result in excessive rutting and mixing of organic debris with the underlying soils.

The base of foundation excavations are expected to be within the claystone bedrock. After grubbing, stripping, rough grading, and any required overexcavations, but prior to placement of structural fill, any loose materials should be removed from the exposed bedrock. Areas to accept new fill and the base of new construction excavations should be evaluated by Olsson to document uniform and stable subgrade conditions prior to the placement of new fill soils or foundation concrete.

In the event that wet or unstable soil conditions are identified during development, such soils should be scarified and recompact in accordance with [Section 4.2](#) of this report. Soils which cannot be adequately densified in-place should be removed and replaced with approved structural fill or stabilized under the direction of an Olsson representative. The extent of areas requiring removal or stabilization will depend on the conditions observed at the time of construction. Undercut areas should be backfilled with stable fill material similar in composition to the surrounding soils.

If necessary, one or more layers of crushed stone may be considered to stabilize areas where wet soil or water are present. Geogrid or geosynthetic fabric may be used in conjunction with the crushed stone to provide additional stabilization.

4.2 Structural Fill

We recommend that fill materials placed within 3 feet of the base of the PRV station slab have a liquid limit less than 45, a plasticity index less than 25, more than 35 percent passing the No.

200 sieve, and a maximum swell potential of 1 percent when remolded as structural fill under an inundation pressure of 500 psf. Soils with Atterberg limits greater than these values will require removal and replacement with suitable fill soils. All structural fill soils should also be relatively free of organic materials (less than about 2 percent by weight), debris, and particles larger than 3 inches in nominal diameter.

The on-site claystone bedrock is not suitable for reuse as structural fill. Due to the minimal quantity of surficial soil present on the site, imported fill should be planned for.

Samples of all proposed structural fill, including on-site soils, should be submitted to Olsson at least seven days before placement for testing and approval.

Proper lift thickness depends on the type of compaction equipment used, but we recommend a maximum lift thickness of 8 inches in loose measurement. Thinner lifts could be required in confined areas such as around the vault or within footing and utility trenches. Soils should be compacted using equipment of appropriate type and size to achieve the recommendations presented in this report. Water flooding is not an acceptable compaction method for any soil type.

We recommend that structural fill and backfill be compacted in accordance with the criteria stated in **Table 1**. Utility trenches, foundation excavations, retaining walls, and pavement curbs should be backfilled as soon as possible to reduce the risk of water infiltration into the subgrade.

An Olsson field representative should periodically observe fill placement operations and perform field moisture-density tests to document whether moisture content and compaction requirements are being achieved.

The moisture content of suitable borrow soils should be within the ranges specified in **Table 1**. More stringent moisture limits may be necessary with certain soils. Adjustment of moisture content may be necessary to allow compaction in accordance with project specifications.

Table 1. Structural Fill Placement Guidelines.

Area of Fill Placement	Suitable Material (USCS or Description)	Compaction (ASTM D698 - Standard Proctor)	Moisture Content (Percent of Optimum)
Structural Fill – site grading or subgrade preparation below foundations	On-site surficial soils or imported fill	95%*	-3 to +3 for cohesionless soils
Non-structural landscaped areas	On-site excavated or imported soils	92%	Or Optimum to +3 for cohesive soils
* Or 70 percent Relative Density as described below			

Granular fill materials may not produce a definable moisture-density curve when tested in accordance with ASTM D698 (Standard Proctor). Such materials could alternatively be compacted to a minimum of 70 percent relative density as determined by ASTM D4253 (Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table) and D4254 (Standard Test Methods for Minimum Index Density and Unit Weight of Soils and Calculations of Relative Density).

Controlled low strength material (CLSM) or flowable fill may be considered for utility or other small backfills. We recommend flowable fill have a compressive strength between 100 and 300 pounds per square inch (psi). CLSM with a maximum compressive strength less than 150 psi can be readily excavated with a backhoe. CLSM can be placed in a single lift, without personnel entering the excavation and without the need for compaction equipment.

4.3 Dewatering Considerations

We did not encounter groundwater at the time of our exploration. As such, we do not anticipate groundwater will impact construction activities. Variations in groundwater elevation could occur because of seasonal changes in rainfall, temperature, snowmelt, runoff, localized irrigation demand, or other factors. Saturated soils and higher groundwater elevations should be anticipated in areas near drainage channels and ditches.

The contractor should utilize their experience in this area and experience with similar projects to determine the most effective method of dewatering and the effects of such methods on nearby structures, utilities, or pavements.

4.4 Temporary Slopes and Excavations

Construction site safety is the sole responsibility of the general contractor. The contractor is also responsible for the means, methods, techniques, sequencing, and operations used during construction. Slope height, slope inclination, and excavation depths (including utility trench excavations) should in no case exceed those specified in local, state, or federal safety regulations.

4.5 Equipment Mobility

Some of the soils encountered at this site may be susceptible to softening under the action of construction equipment traffic in combination with wet weather. Mitigation of equipment mobility problems and management of soft surficial soils will depend on the severity of the problem, the season in which construction is performed, and prevailing weather conditions.

During construction, provisions should be made to quickly remove seepage water or storm water from excavations. Water should not be allowed to collect near foundations either during or after construction. Undercut or excavated areas should be sloped toward one corner to facilitate the collection and removal of rainwater or surface runoff. Site grading should provide rapid drainage of water away from the structure and pavement areas throughout construction.

Additional guidelines for reducing equipment mobility problems are as follows:

- Optimize surface water drainage at the site.
- Allow for rain days in the construction schedule and wait for dry weather conditions to prevail whenever possible. Avoid operating construction equipment on the site during wet conditions. Rutting the surface will aggravate mobility problems.
- Use construction equipment that is suited for the intended job under the site conditions. Heavy rubber-tired equipment typically requires better site conditions than light, track-mounted equipment.

Ultimately, it may be necessary to take steps to aggressively improve equipment mobility if construction must proceed under unfavorable conditions. More aggressive methods for addressing equipment mobility problems may range from removing several feet of soft wet soils to utilizing crushed stone materials and/or appropriate stabilization fabrics or geogrids. Other methods include chemical stabilization with Portland cement, fly ash, or cement kiln dust (CKD). The stabilization approach should be determined at the time of construction in consultation with an Olsson geotechnical engineer.

The contractor is responsible for creating and maintaining a stable working platform. Soils that are disturbed by construction activity or adverse weather conditions should be corrected by the contractor to conform with project specifications and this report.

5. STRUCTURES DESIGN

Based on the results of our borings, laboratory testing, and engineering evaluation, our opinion is that the subsurface conditions are suitable for supporting the proposed PRV station on a mat foundation.

5.1 Mat Foundations

In our opinion, the proposed slabs for the PRV station can be designed as a mat foundation if the recommendations of this report are followed. We recommend at least 3 feet of claystone below foundations be overexcavated and replaced with properly compacted structural fill. The overexcavations should extend laterally at least 3 feet beyond the edges of the foundation. The base of the overexcavation should be prepared as discussed in [Section 4.1](#) and the base of the excavation observed and documented by Olsson. The overexcavated claystone should be replaced with suitable structural fill meeting the material recommendations of [Section 4.2](#) and should be compacted according to the recommendations of [Table 1](#).

Mat foundations should be designed for a maximum net allowable bearing pressure of 2,500 pounds per square foot (psf). The net allowable bearing capacity can be increased by 1/3 for transient loading such as wind or seismic. Mat foundations constructed on compacted structural fill are expected to experience total post-construction movements on the order of 1 inch and differential movements of approximately ½ inch.

The thickness and reinforcement for the mat foundation should be designed by the project structural engineer. A mat foundation can also be designed using a subgrade modulus value. If the structural designer would prefer this design methodology, Olsson recommends a modulus of subgrade reaction of 150 pounds per cubic inch (pci) for the design of the mat foundation bearing on approved structural fill as discussed in this report. This value is a unit value for use with a 1-foot square foundation. The modulus should be reduced in accordance with the following equation when used with larger foundations:

$$K_{(B \times B)} = K_1 \left(\frac{1}{B} \right)$$

where $K_{(B \times B)}$ is the subgrade modulus for a mat foundation (dimensions B x B), K_1 is the unit subgrade modulus (dimensions 1 foot by 1 foot), and B is the foundation width (in feet).

The following equation can be used to correct the modulus value for a foundation having dimensions B by L:

$$K_{(B \times L)} = \frac{K_{(B \times B)} \left(1 + 0.5 * \left(\frac{B}{L} \right) \right)}{1.5}$$

A friction coefficient value of 0.35 may be used for concrete mat foundations bearing directly against granular prepared subgrades or approved structural fill.

The long-term performance of the mat foundation will greatly depend on minimizing or reducing moisture infiltration into the backfill soils.

5.2 Lateral Earth Pressures

The following soil parameters are provided for designing cast-in-place concrete cantilevered retaining walls and/or foundation walls subject to lateral earth pressures. These parameters are based on the assumption that retained soils will be similar in composition to the on-site soils encountered during this investigation. These recommendations are not applicable to the design of modular block - geogrid reinforced backfill (MSE) or gravity block walls. Recommendations for these types of wall systems are beyond the scope of this report.

Walls which are rigidly restrained at the top and are essentially unable to deflect or rotate should be designed for at rest earth pressure conditions. Walls that are unrestrained at the top and are free to deflect or rotate slightly may be designed for active earth pressure conditions. The passive earth pressure condition is used to evaluate the resistance of soil to lateral loads. **Table 2** presents recommended values of earth pressure coefficients and equivalent fluid densities based on our experience with soils in the area.

Table 2. Earth Pressure Parameters.

Earth Pressure Coefficient (K)			Equivalent Fluid Density (G)	
			Drained Condition (pcf)	Undrained Condition (pcf)
At Rest (K_0)	Cohesive	0.56	60	90
	Granular*	0.50	55	--
Active (K_a)	Cohesive	0.39	45	80
	Granular*	0.33	40	--
Passive (K_p)	Cohesive	2.56	280	185
	Granular*	3.00	330	--
*Granular values may be used for design only if a drainage system is installed and the granular backfill extends upward from the base of the wall as noted below.				

The above parameters are based on the considerations noted below and are shown in **Figure 2**, below.

- Equivalent fluid densities do not include a factor of safety or consider the effects of hydrostatic pressure, surcharge loading (P_1), point loads, or construction equipment loads.
- Mobilization of active pressure requires the wall rotate about the base, with top-of-wall movements (d) on the order of $0.002 \cdot Z$ to $0.004 \cdot Z$ (granular) or $0.010 \cdot Z$ to $0.020 \cdot Z$ (cohesive), where Z is the wall height.
- Mobilization of passive pressure requires a lateral wall movement (d) on the order of $0.020 \cdot Z$ to $0.060 \cdot Z$ (granular) or $0.020 \cdot Z$ to $0.040 \cdot Z$ (cohesive), where Z is the wall height.
- Drained earth pressure parameters assume a permanent drainage system is installed behind the wall to prevent the development of hydrostatic pressure.
- Backfill has a maximum unit weight of 110 pcf.
- The ground surface in front of and behind the wall is horizontal.

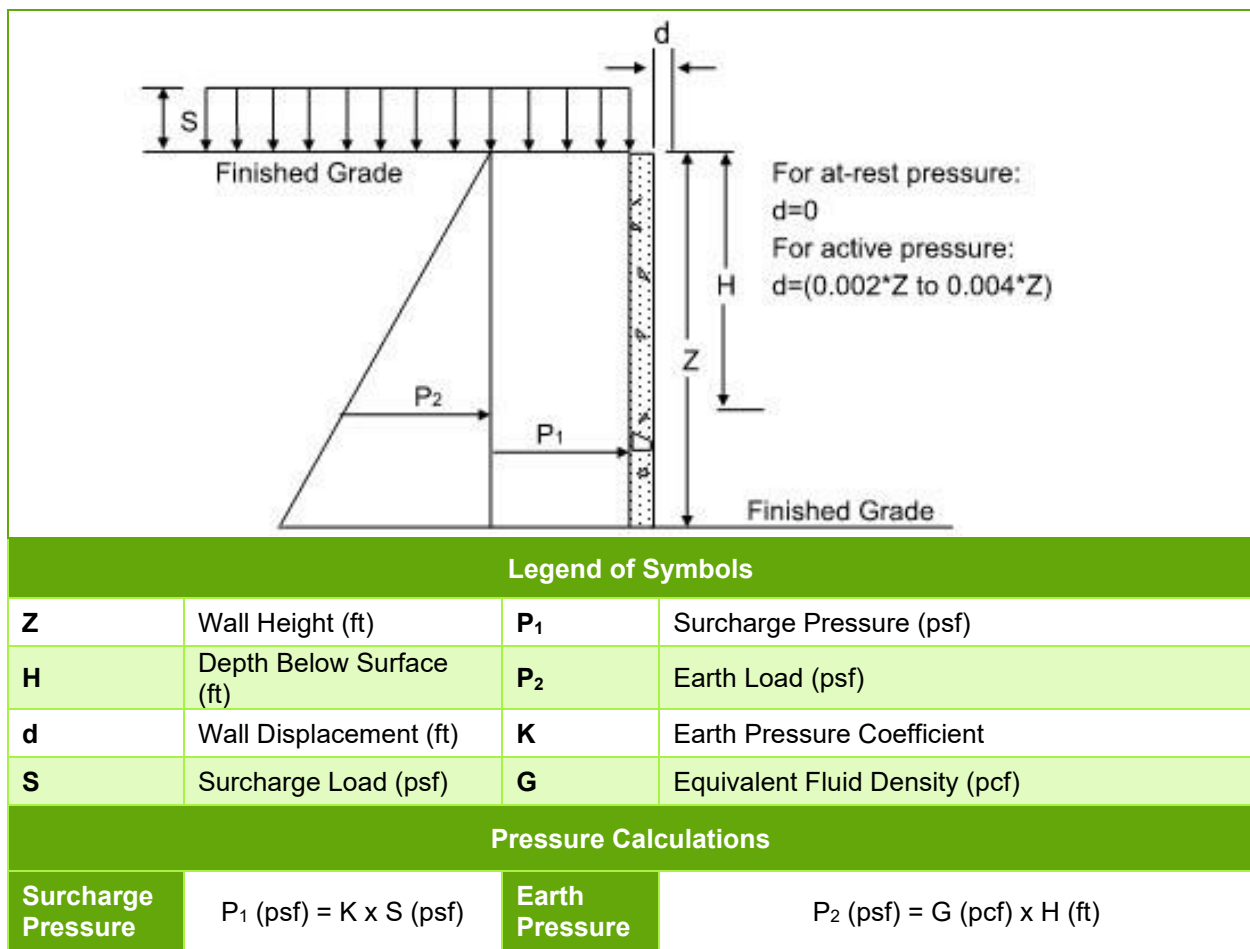


Figure 2. Lateral Earth Pressure Calculations.

Backfill soils placed within a lateral distance from the face of the wall to 70 percent of the wall height should consist of granular material or select lean clay with a liquid limit less than 45. To utilize earth pressure parameters for granular materials, the granular backfill must extend out from the base of the wall at angles of 45 and 60 degrees from the vertical for the active and passive cases, respectively.

Sliding resistance along the base of a wall supported on suitable native soils or properly compacted structural fill may be evaluated using an ultimate sliding friction value of 0.35. Appropriate factors of safety should be applied to the calculated lateral earth pressures and sliding friction resistance. This factor of safety typically ranges from 1.5 to 2.0. Passive earth pressure resistance should be neglected within the frost zone (typically 3 feet).

If a foundation key is considered to resist lateral sliding loads using passive earth pressure, the key should be placed below the wall stem or to the toe side of the wall stem. Sliding resistance along the base of the foundation should be neglected within the passive earth pressure zone.

5.3 Seismic Site Classification

For this project site, the soil conditions encountered at the boring are consistent with Site Class “C” as defined by ASCE 7-22. Our review of the site class is based on the soil conditions encountered in the borings during the exploration and our assumption that the encountered soil conditions are underlain by similar native materials to those encountered which extend to a depth of 100 feet.

5.4 Permanent Slopes

We recommend permanent cut or fill slopes be shallower than 3(H):1(V) to maintain long-term stability and to provide ease of maintenance. The crest or toe of cut or fill slopes should be at least 10 feet from any foundation and at least 5 feet from the edge of any pavements.

Permanent slopes should be vegetated as soon as practical to minimize the potential for erosion. Slopes steeper than 3(H):1(V) are susceptible to erosion, will be difficult to maintain, and could experience problems with instability. Such slopes may require additional slope stability analysis, which is beyond the scope of this report.

The performance of the proposed structure depends on maintaining the moisture content of the subgrade soils throughout the life of the facility. To reduce the effects of moisture infiltration near the structure, efficient drainage of rainfall or surface runoff should be provided. We recommend a minimum slope of 2 percent for pavement areas and 5 percent for grass or landscaped areas.

6. REPORT LIMITATIONS

The conclusions and recommendations presented in this report are based on the information available regarding the proposed construction, geotechnical information obtained from our field exploration and laboratory testing, as well as our experience with similar projects. Our borings and testing represent a limited statistical sampling of the subsurface. Conditions may be encountered during construction that are substantially different from those encountered in this exploration and adjustments to design and construction may be necessary.

In the event of any changes in the nature of the proposed project as outlined in this report, the opinions in this report cannot be considered valid unless Olsson reviews the changes, and the opinions of this report are modified or affirmed by Olsson.

The scope of this exploration did not include any environmental assessment for the presence of wetlands and/or hazardous or toxic materials in the soil or groundwater on or near the site. Any statements in this report regarding odors, discoloration, or suspicious conditions are strictly for the information of our client.

This report is based on generally accepted professional geotechnical engineering practice at the time of this report, within this geographic area. No warranty, express or implied, is intended or made. This report has been prepared for the exclusive use of the City of Thornton and their authorized representatives for specific application to the discussed project.

APPENDIX A

Exploration Maps, Log, and Information

Field Exploration

Our drill crew advanced the boring for this exploration with a truck-mounted drill rig using continuous-flight augers. The boring location was located in the field by the drill crew using a hand-held GPS unit. The approximate locations of the boring is shown on the Boring Location Map.

Samples were obtained using the methods and at the depths shown on the log. Recovered samples were sealed in containers, labeled, and protected for transportation to the laboratory for testing.

The drill crew prepared the field boring log during drilling operations. The field log includes drilling and sampling methods, sampling intervals, groundwater measurements, and general descriptions of the observed soil conditions. The final boring log represents our engineering interpretation of the field log based on visual classification and laboratory testing of the collected samples.



LEGEND

 SOIL BORING LOCATION

NOT TO SCALE

olsson

PROJECT: 024-03983

DRAWN BY: DS

DATE: 11/19/25

BORING LOCATION MAP
THORNTON, COLORADO

5180 Smith Rd Unit F
Denver, CO 80216

TEL 303.237.2072
www.olsson.com

BORING LOG NO. B-1							Sheet 1 of 1					
PROJECT NAME Elmwood Lane and West 79th Place PRV Station					CLIENT City of Thornton							
PROJECT NUMBER 024-03983					LOCATION Thornton, Colorado							
ELEVATION (ft)	MATERIAL DESCRIPTION		GRAPHIC LOG	DEPTH (ft)	SAMPLE TYPE NUMBER	CLASSIFICATION (USCS)	BLOWS/6" (N-VALUE)	UNC STR (tsf)	MOISTURE (%)	DRY DENSITY (pcf)	LL/PI (%)	ADDITIONAL DATA/REMARKS
	LAT: 39.841478 LONG: -104.997322											
	TOPSOIL		0.5'	1	MC-01		16-20		24.9	81.0		P-200 = 86%
	LEAN CLAY WITH SAND (CL), very stiff, dark brown, moist, fine to coarse sand, trace roots		1.5'	2								
	CLAYEY SAND (SC), dense, light brown, moist, fine to coarse sand, trace fine gravel, iron oxide staining		3.5'	3								
-5.0	CLAYSTONE, highly weathered, dark gray to gray, iron oxide staining, trace fine sand, very moist			4	SPT-02		11-14-18 (32)		33.7			
				5	MC-03		14-25		26.6			
				6								
				7	SPT-04		14-17-25 (42)					
				8								
-10.0				9	MC-05		50/6"		32.2	85.9		
				10								
				11								
				12								
				13								
-15.0				14	SPT-06		29-30-50/5"		26.0			
				15								
				16								
				17								
-20.0				18								
				19	MC-07		50/3"					
				20								
				21								
				22								
				23								
-25.0				24	SPT-08		28-29-50/6"					
	BASE OF BORING AT 25.5 FEET		25.5'	25								
				26								
				27								
				28								
				29								
-30.0				30								
				31								
				32								
				33								
				34								
WATER LEVEL OBSERVATIONS							STARTED: Oct 24 2025		FINISHED: Oct 24 2025			
WD	≡	Not Encountered		DRILL CO.: Vine Labs			EQUIPMENT: CME 55					
IAD	▼	Not Encountered		DRILLER: Vine Labs			LOGGED BY: M. Gross					
AD	≡	Not Performed		METHOD: Continuous Flight Auger								

SYMBOLS AND NOMENCLATURE

DRILLING NOTES

DRILLING AND SAMPLING SYMBOLS

SS: Split-Spoon Sample (1.375" ID, 2.0" OD)	HSA: Hollow Stem Auger	NE: Not Encountered
U: Thin-Walled Tube Sample (3.0" OD)	CFA: Continuous Flight Auger	NP: Not Performed
CS: Continuous Sample	HA: Hand Auger	NA: Not Applicable
BS: Bulk Sample	CPT: Cone Penetration Test	% Rec: Percent of Recovery
MC: Modified California Sampler	WB: Wash Bore	WD: While Drilling
GB: Grab Sample	FT: Fish Tail Bit	IAD: Immediately After Drilling
SPT: Standard Penetration Test Blows per 6.0"	RB: Rock Bit	AD: After Drilling
	PP: Pocket Penetrometer	CI: Cave-In

DRILLING PROCEDURES

Soil samples designated as "U" samples on the boring logs were obtained in using Thin-Walled Tube Sampling techniques. Soil samples designated as "SS" samples were obtained during Penetration Test using a Split-Spoon Barrel sampler. The standard penetration resistance 'N' value is the number of blows of a 140 pound hammer falling 30 inches to drive the Split-Spoon sampler one foot. Soil samples designated as "MC" were obtained in using Thick-Walled, Ring-Lined, Split-Barrel Drive sampling techniques. Recovered samples were sealed in containers, labeled, and protected for transportation to the laboratory for testing.

WATER LEVEL MEASUREMENTS

Water levels indicated on the boring logs are levels measured in the borings at the times indicated. In relatively high permeable materials, the indicated levels may reflect the location of groundwater. In low permeability soils, the accurate determination of groundwater levels is not possible with only short-term observations.

SOIL PROPERTIES & DESCRIPTIONS

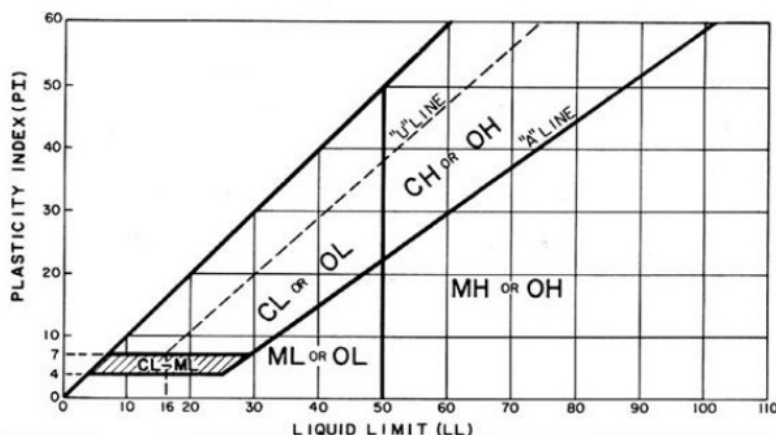
Descriptions of the soils encountered in the soil test borings were prepared using Visual-Manual Procedures for Descriptions and Identification of Soils.

PARTICLE SIZE

Boulders	12 in. +	Coarse Sand	4.75mm-2.0mm	Silt	0.075mm-0.005mm
Cobbles	12 in.-3 in.	Medium Sand	2.0mm-0.425mm	Clay	<0.005mm
Gravel	3 in.-4.75mm	Fine Sand	0.425mm-0.075mm		

COHESIVE SOILS		COHESIONLESS SOILS		COMPONENT %	
Consistency	Unconfined Compressive Strength (Qu) (tsf)	Relative Density	'N' Value	Description	Percent (%)
Very Soft	<0.25	Very Loose	0 - 3	Trace	<5
Soft	0.25 - 0.5	Loose	4 - 9	Few	5 - 10
Firm	0.5 - 1.0	Medium Dense	10 - 29	Little	15 - 25
Stiff	1.0 - 2.0	Dense	30 - 49	Some	30 - 45
Very Stiff	2.0 - 4.0	Very Dense	≥ 50	Mostly	50 - 100
Hard	> 4.0				

PLASTICITY CHART

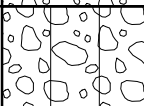
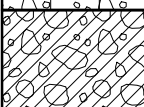
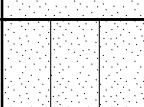
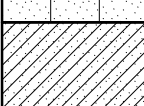
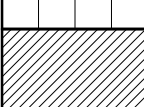
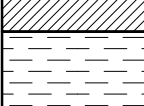
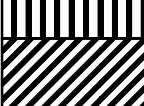
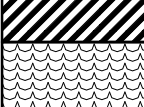
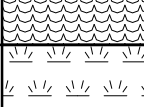


ROCK QUALITY DESIGNATION (RQD)

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

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SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS	
			GRAPH	LETTER		
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES	
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES	
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES	
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES	
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES	
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES	
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES	
		FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
					CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
	OL			ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY		
SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50			MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS		
			CH	INORGANIC CLAYS OF HIGH PLASTICITY		
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS		
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS	

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

APPENDIX B

Laboratory Test Results

SUMMARY OF LABORATORY RESULTS

PROJECT NAME:	Elmwood Lane and West 79th Place PRV Station
PROJECT NUMBER:	024-03983

CLIENT: City of Thornton CO

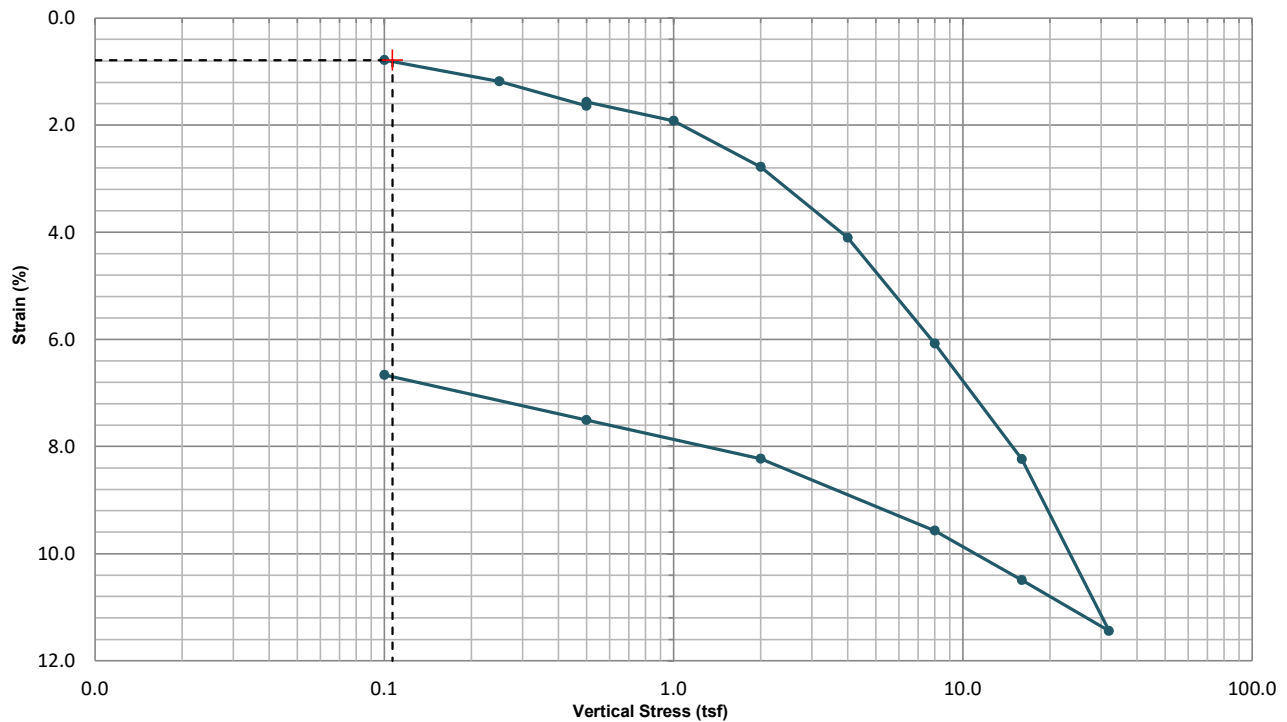
PROJECT LOCATION: Thornton, Colorado

[illegible]



ASTM D4546
One-Dimensional Swell or Collapse of Soils

Project Information			
Project Name:	Elmwood Lane and West 79th Place PRV Station		
Project Number:	024-03983		
Client Name:	City of Thornton CO		
Sample Information		Test Results	
Boring No.:	B-1		
Sample No.:	MC-5	Specimen Mass (g)	Before After
Depth (ft):	9 - 10	Water Content (%):	73.4 78.4
Date Tested:	November 5, 2025	Dry Unit Weight (pcf):	32.2 37.0
Sample Description:	Claystone, light brown	Saturation (%):	85.9 94.0
Test Information		Void Ratio:	92.2 100.0
Test Method:	C	Initial Height, h (in.):	0.926 0.758
Preparation Method:	Intact	Height After Swell, h1 (in.):	0.835
Water Type:	Distilled	Magnitude of Swell (%):	0.851
Inundation Pressure (tsf):	0.50	Swell Pressure (tsf):	0.1
Atterberg Results		Final Height, h2 (in):	0.1
Liquid Limit: --	Plastic Limit: --		0.786
Plasticity Index: --		Remarks	





5180 Smith Road
Denver, CO 80216

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FAX 303.237.2659

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Soil Sulfate Test

Project Information	
Project Name:	Elmwood Lane and West 79th Place PRV Station
Project Number:	024-03988
Client Name:	City of Thornton
Project Location:	Thornton, Colorado
Sample and Test Information	
Sample Location:	B-1 Bulk, 1-5 feet
Sample Description:	Sandy Lean Clay/Clayey Sand, light brown to dark brown
Laboratory Technician:	NR
Date Tested:	10/7/2025
Test Results	

Water Soluble Sulfate (Colorado Procedure CP-L-2103)			
Dilution	Reading	Concentration, mg/L	Concentration, % mass
100:1	1	100	0.01

ELMWOOD LANE AND WEST 79TH PLACE PRV STATION

Thornton, Colorado

December 2025

Olsson Project No. 024-03983