ADDENDUM NO. 2

DATE: **May 15, 2026**

PROJECT: **CM/GC RFP**
New Fire Station #4
Montrose Fire Protection District

PROJECT NO.: **26004**

NOTICE

THIS ADDENDUM IS ISSUED BY ARCHITECT TO ALL KNOWN INDIVIDUALS, FIRM OR CORPORATIONS WHO HOLD RFP DOCUMENTS FOR ABOVE LISTED PROJECT.

THIS ADDENDUM IS HEREBY MADE A PORTION OF RFQ DOCUMENTS, AS APPROPRIATE. ACKNOWLEDGE RECEIPT OF THIS ADDENDUM IN APPROPRIATE SPACE ON THE PROPOSAL FORM.

INDEX **PAGE**

ARCHITECT:	Blythe Group + co.
CIVIL ENGINEER:	Del-Mont Consultants, Inc.
MECHANICAL/ELECTRICAL ENGINEER:	Bighorn Consulting Engineers, Inc.
STRUCTURAL ENGINEER:	HCL Engineering and Surveying, LLC
LANDSCAPE ARCHITECT:	Mitch Rewold Landscape Architect (MRLA)

ADVERTISEMENT

AD-2, Item 01: (Owner's Budget and Funding)

Reference Section 1.1 (Project Description / Scope of Work)

Clarification: Add the following paragraph to the end of this section:
"*Budget / Funding: The construction budget is $5,800,000 for costs requested in this RFP. Funding has been secured. Currently, funding is not contingent on any grant approvals or passing of bond initiatives and does not require the involvement of the CMGC.*".

AD-2, Item 02: (Schedule)

Reference Section 1.4 (Tentative Project Schedule),
Section 2.4 (Key Events Schedule),

Clarification: Modify schedules as indicated below:

- Add the following category "May 26, 2026 – Noon – Notify shortlist firms"
- Change the interview date from May 28, 2026, to June 2, 2026
- Change / Add "CMGC Selection and Notification" date from 6/1/2026 to 06/04/2026.
- Add the following category "June 12, 2026 – Finalize MFPD/CMGC Agreement (Subject to Board Approval)".
- Change / Add "CMGC Agreement Finalized" date from 06/12/2026 to 06/15/2026

AD-2, Item 03: (Owner's Budget and Estimate Clarification)

Reference Section 2.5 (Award of Contract)

Clarification: Add the following sentences to the end of this section:

"*The initial cost estimate will be a limited part of the qualifications ranking / selection process and serve as a starting point for further value engineering after the selection process is complete.*"

"The following are as needed costs assumed to be part of the "Anticipated General Conditions" which is intended to be a not-to-exceed value. All other costs should be part of the CM Preconstruction Fee, Construction Fee, or the Direct Cost of Work.:

<u>Preconstruction</u> – NA

<u>Project Budgeting and Cost Control</u> – NA

<u>Sub-Contracting Selection and Purchasing</u> – NA

<u>Contract Documents Coordination</u> – NA

<u>Construction Phase Staff (On-Site)</u> – Project Director / Executive, Project Manager(s), Project Engineer(s), Project Superintendent(s), M/E Coordinator, Field Supervision, Field Coordination, General Foreman, Clerical/Secretarial.

<u>Travel and Lodging</u> – Moving and Subsistence Cost, Apartment Rent, Home/Office Travel.

<u>Temporary Facility</u> – Safety Barrier / Warnings / Handrails, Field Office / Trailer, Onsite Storage for CM/GC, Office Furniture, Project Signs, Construction Fencing, Fire Extinguishers.

<u>Temporary On-Site Utilities and Services</u> – Temp Power, Temp Water, Cell Phones, Trailer Phone/Internet, Drinking Water, Temp Toilets / Sanitary Facilities, Printer/Copier, IT, Dumpster and Disposal Fees, Jobsite Cleanup (Excludes Final Cleanup), Temporary Walkways, Drives, and Parking, Roadway Maintenance, Dust Controls, Loading and Unloading Material, Material Handling.

<u>On-Site Equipment</u> – Automobile and Equipment Rent, Fuel/Tires/Maintenance, Small Tools, Forklift, Air Compressor, generator, snow removal.

<u>General Temporary Heating</u> – Snow and Ice Removal, Field Office Heat. (Note: Temp weather protection & heating for subcontractors are Cost of the Work).

<u>Reproduction/Printing and Data Processing</u> – Postage/Delivery, Office Supplies, Office Equipment, Office Cleanup, Printing / Reproduction.

<u>Quality Control</u> – Quality Control, Progress Photos & Videos, File Management Software (I.E. Procore), Safety Coordination, Safety Equipment, As-Built Drawings.

<u>Permits and Special Fees</u> – Construction Equipment Permits.

<u>Other</u> – First Aid.

<u>Off-Site Services</u> – NA (Included in either Pre-Construction Service or Construction Services Fee).

APPENDIX D – PLANS & NARRATIVES

AD-2, Item 04: (Site Assumptions)

Reference: The "Preliminary Site Plan" drawing of the Schematic Design Report.

Clarification: Add the following general notes to this sheet:
"*1. Assume a 'balanced' site that would require no material to be brought in or removed outside of structural fill.*
2. Assume utilities shown as existing will be in-place when construction is mobilized."

APPENDIX E – PROPOSAL FORM

AD-2, Item 05: (Proposal Form)
Reference Current Proposal Form:

Clarification: Replace the current Proposal Form with the revised Proposal Form included as Attachment 01 of this Addendum which has the following items integrated.

- Modify Section titled "Initial Estimate of Construction Cost", paragraph 1 to read as follows: "*Initial, non-binding estimate of construction cost based on the RFP information including any design and construction contingencies but exclusive of fees included below.* ***An 'Order of Magnitude' estimate using square footages and assembly-level pricing is expected***."
- Add the following sentences after the section titled "Construction Phase Fees":
"*(Fees should be for total project costs and not monthly)"*
"*(Assume schedule included in the RFP is the correct duration)".*
- Add the following paragraph to Section titled "Construction Phase Fees" before "5. Bonding" and adjust numbering as required:
"4. Insurance $_______ ____%"

APPENDIX F – SOILS REPORT

AD-2, Item 06: (Soils Report)
Reference The end of the RFP

Clarification: Add Attachment 02 of this RFP ("Geotechnical Engineering Report" dated May 04, 2026) as Appendix F of the RFP.

APPENDIX G – SITE VISIT ATTENDANCE

AD-2, Item 07: (Site Visit Attendance Sheet)
Reference The end of the RFP

Clarification: Add Attachment 03 of this RFP ("Non-Mandatory Site Visit" dated May 12, 2026) as Appendix G of the RFP.

--END OF ADDENDUM 02---

APPENDIX E – Proposal Submittal Form

PROPOSAL FORM

CM/GC SERVICES PROPOSAL FOR THE

NEW FIRE STATION #4

SUNNYSIDE ROAD / 6720 ROAD INTERSECTION
MONTROSE, OK
BG+co JN: 26004

Montrose Fire Protection District
441 South Uncompahgre Ave.
Montrose, CO 81401

To Whom it May Concern:

Having carefully examined the Request For Proposal (RFP) for the new Fire Station #4 in Montrose Colorado, the undersigned proposes to furnish all Work called for by said documents for the sums set forth as follows:

PRINT OR TYPE INFORMATION BELOW:

PROPOSAL SUBMITTED BY:

____

Contractor Name | Date

____

Contractor Address

____

Contact Person | Title | Phone

____

Email address

TOTAL PROJECT COST:

Initial Estimate of Construction Cost

1. Initial, non-binding estimate of construction cost based on the RFP information including any design and construction contingencies but exclusive of fees included below. An 'Order of Magnitude' estimate using square footages and assembly-level pricing is expected.

$____________

Preconstruction Phase Fees

2. CM Preconstruction Fee $____________

Construction Phase Fees

(Fees should be for the total project costs and not monthly)
(Assume schedule included in the RFP is the correct duration)

1. CM/GC Construction Fee $____________ _______%
2. Anticipated General Conditions $____________
3. Other Reimbursable General Conditions $____________
4. Insurance $____________ _______%
5. Bonding $____________ _______%
6. Other (include list) $____________ _______%

Total GC Fee $____________

Sub-Contractor Mark-up:

1. Indicate the value, up to but not-to-exceed, in a percentage format of CM/GC mark-ups on scope changes to any sub-contractor: _______%
2. Percentage if scope reduction is different from scope increase: _______%

Addenda

The undersigned acknowledges receipt of _____ Addenda issued during the proposal period and that the changes included therein are included in this proposal.

Modification or Withdrawal of Proposal:

The undersigned agrees that this Proposal will not be modified nor withdrawn for a period of thirty (30) calendar days from the date hereof.

Respectfully Submitted:

______________________________ ______________________________

DATE PROPOSER PRINTED

SEAL (IF PROPOSED BY A CORPORATION)

____

PROPSER TITLE

____

PROPOSER SIGNATURE

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

GEOTECHNICAL ENGINEERING STUDY

PROPOSED FIRE STATION #4

SUNNYSIDE ROAD AND 6720 ROAD

MONTROSE, COLORADO

Prepared for:

MONTROSE FIRE PROTECTION DISTRICT

PROJECT NUMBER: M26010GE

MAY 4, 2026

P.O. Box 3986
Grand Junction, CO 81502
(970) 245 6506

P.O. Box 45
Montrose, CO 81402
(970) 249 2154

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

May 4, 2026

Montrose Fire Protection District
441 S Uncompahgre Avenue
Montrose, Colorado

Attention: Mr. Tad J Rowan, Fire Chief

PN: M26010GE

Subject: Geotechnical Engineering Study for the
Proposed Fire Station 4 Structure
Sunnyside Road and 6720 Road
Montrose, Colorado

Mr. Rowan:

Lambert and Associates is pleased to present our geotechnical engineering study for the subject project. The field study was completed on March 30, 2026. The laboratory study was completed on April 22, 2026. The analysis was performed and the report prepared from April 22 through May 4, 2026. Our geotechnical engineering report is attached.

We are available to provide material testing services for soil and concrete and provide foundation excavation observations during construction. We recommend that Lambert and Associates, the geotechnical engineer, for the project provide material testing services to maintain continuity between design and construction phases.

If you have any questions concerning the geotechnical engineering aspects of your project please contact us. Thank you for the opportunity to perform this study for you.

Respectfully submitted,

LAMBERT AND ASSOCIATES

Daniel R. Lambert, P.E.

P.O. Box 3986
Grand Junction, CO 81502
(970) 245 6506

P.O. Box 45
Montrose, CO 81402
(970) 249 2154

ADDENDUM 02, ATTACHMENT 02

TABLE OF CONTENTS

1.0 INTRODUCTION

This report presents the results of the geotechnical engineering study we conducted for the proposed Montrose Fire Protection District Fire Station #4 facility. The study was conducted at the request of Mr. Tad Rowan, Fire Chief, Montrose Fire Protection District, in general accordance with our proposal for the project dated March 11, 2026.

The conclusions, suggestions and recommendations presented in this report are based on the data gathered during our site and laboratory study and on our experience with similar soil conditions. Factual data gathered during the field and laboratory work are summarized in Appendices A and B.

1.1 Proposed Construction

It is our understanding the proposed construction is to include a fire station structure and associated paved parking and drive areas.

1.2 Scope of Services

Our services included geotechnical engineering field and laboratory studies, analysis of the acquired data and report preparation for the proposed site. The scope of our services is outlined below.

- The field study consisted of describing and sampling the soil materials encountered in eight (8) small diameter test borings in the general vicinity of the proposed improvements.

- The materials encountered in the test borings were described and samples retrieved for the subsequent laboratory study.

- The laboratory study included tests of select soil samples obtained during the field study to help assess:

  - the soil strength potential (internal friction angle and cohesion) of samples tested,
  - the swell and expansion potential of the samples tested,
  - the settlement/consolidation potential of the samples tested, and
  - the moisture content and density of samples tested.

- This report presents our geotechnical engineering comments, suggestions and recommendations for planning and design of site development including:

- viable foundation types for the conditions encountered,
- allowable bearing pressures for the foundation types,
- lateral earth pressure recommendations for design of laterally loaded walls,
- geotechnical engineering considerations and recommendations for concrete slab on grade floors,
- geotechnical engineering considerations and recommendations for compacted structural fill and
- several roadway pavement section thickness alternatives.

- Our comments, suggestions and recommendations are based on the subsurface soil and ground water conditions encountered during our site and laboratory studies.

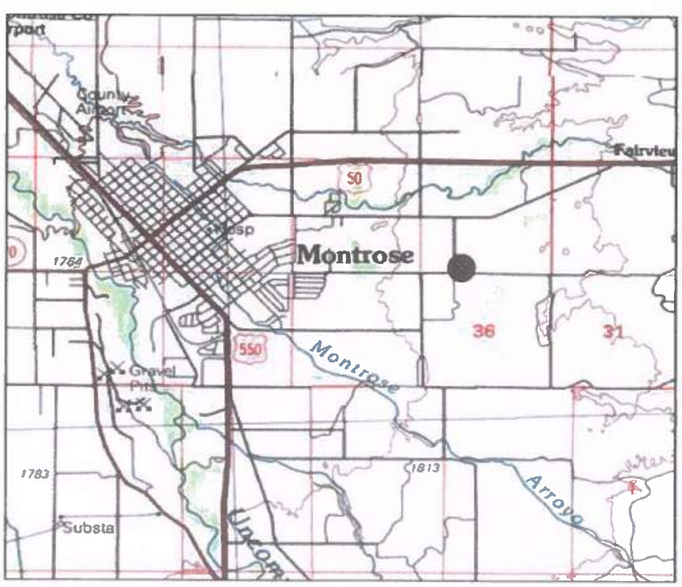
- Our study did not include any environmental or geologic hazard issues.

2.0 SITE CHARACTERISTICS

Site characteristics include observed existing and pre-existing site conditions that may influence the geotechnical engineering aspects of the proposed site development.

2.1 Site Location

The site is located north of Sunnyside Road and east of 6720 Road, Montrose, Colorado.



● Indicates approximate project location

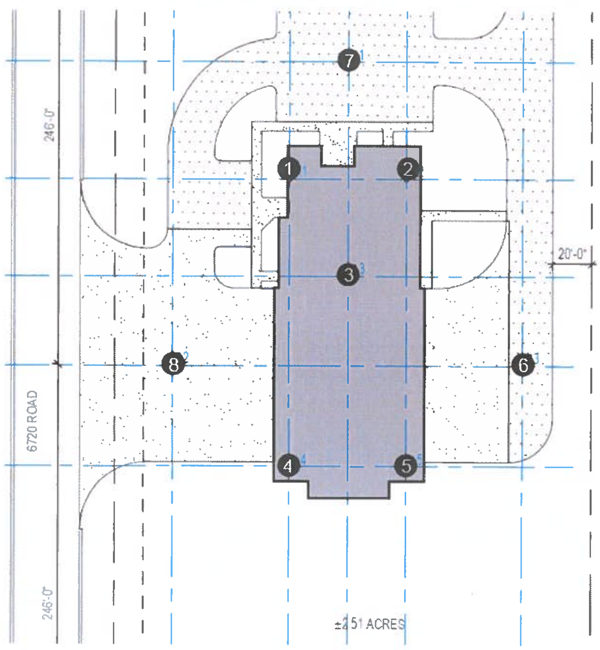
Figure 1. Project Vicinity Map

2.2 Site Conditions

The area is currently vacant. The area was previously used for agricultural purposes. The site is vegetated with grasses and brush, is relatively flat and exhibits positive surface drainage to the south. The area is bordered to the west by 6720 Road, to the south by an existing irrigation ditch and Sunnyside Road and to the north and east by land similar in terrain to the subject site.

2.3 Subsurface Conditions

The subsurface exploration consisted of observing, describing and sampling the soil materials encountered in eight (8) small diameter test borings.



● Approximate Test Boring Location

Figure 2. Test Boring Location Sketch

The materials encountered within the test borings generally consisted of silty clay materials extending to the underlying granular materials and/or formational shale materials. The formational shale materials were encountered in Test Boring Nos. 1 through 5 at approximate depths of eight and one half (8-1/2) to sixteen (16) feet below existing site grades and extended to the depths explored.

Free subsurface water was encountered in Test Boring No. 5 at an approximate depth of fourteen and one half (14-1/2) feet below existing site grade during the drilling activities. The logs describing the soil materials encountered in the test borings are presented in Appendix A.

At the time of our field study the proposed development site was not irrigated. It has been our experience that after the site is developed and once landscape irrigation begins the free subsurface water level may tend to rise. In some cases the free subsurface water level rise, as a result of landscape irrigation and other development influences, can be fairly dramatic and the water level may become shallow.

It is difficult to predict if unexpected subsurface conditions will be encountered during construction. Since such conditions may be found, we suggest that the owner and the contractor make provisions in their budget and construction schedule to accommodate unexpected subsurface conditions.

2.4 Site Geology

A brief discussion of the general geology of the area near the site is presented in Appendix C. The surface geology of the site was determined by observation of the surface conditions at the site and observing the soils encountered in the test borings on the site.

2.5 Seismicity

According to the International Building Code, 2018 Edition, and ASCE Standard ASCE/SEI 7-10, Table 20.3-1 Site Classification, based on the subsurface conditions encountered and the assumption that the soils described in the test borings are likely representative of the top 100 feet of the soil profile, we recommend that the site soil profile be S_D, Stiff soil.

3.0 PLANNING AND DESIGN CONSIDERATIONS

A geologic hazard study was not requested as part of the scope of this report.

All of the suggestions and design parameters presented in this report are based on high quality craftsmanship, care during construction and post construction cognizance of the potential for swell or settlement of the site support materials and appropriate post construction maintenance.

All construction excavations should be sloped to prevent excavation wall collapse. We suggest that as a minimum the excavation walls should be sloped at an inclination of one-and-one-half (1-1/2) to one (1) (horizontal to vertical) or flatter. The area above the foundation excavations should be observed at least daily for evidence of slope movement during construction. If evidence of slope movement is observed we should be contacted immediately.

We anticipate that excavation and fill placement operations may be associated with the proposed site development. Excavations in the area which generate vertical or sloped exposures should be kept to a minimum.

Excavations which result in cut slopes with a vertical height greater than about four (4) feet or with a slope or structure above should be analyzed on a site specific basis. Temporary excavation cut slopes in competent material should not exceed a one-and-one-half to one (1 -1/2 to 1) (horizontal to vertical) inclination. All construction excavations should conform to Occupational Safety and Health Administration (OSHA) standards or safer. All permanent slopes should be constructed with inclinations of three to one or flatter.

Slope and excavation surfaces should be protected by vegetation and/or other means to prevent erosion. Surface runoff should not be allowed to cascade over the top of a slope or to pond at the toe of any slope.

Generally, fill material placed on a site surface which will be used to support structures or additional fill material should be placed so that the contact between the existing site surface and the added fill material will be strong enough to support the added load. This should be addressed on a site and fill area specific basis. The technique recommended will be based on the site configuration, the finished fill configuration the actual material to be used for the fill material and the size of the area thus constructed. Frequently the preparation of the site area to receive fill material will include removing organic and loose near surface native material in the area to receive fill material, placing the material in thin horizontal lifts which are compacted at the appropriate moisture content. Some fill areas could benefit from the installation of a subsurface drain system at the fill material/natural material contact. We are available to, and recommend that, we discuss this with you and provide site and fill specific recommendations when this portion of your development plan merits the additional study.

4.0 ON-SITE DEVELOPMENT CONSIDERATIONS

We anticipate that the subsurface water elevation may fluctuate with seasonal and other varying conditions. Excavations may encounter subsurface water and soils that tend to cave or yield. If water is encountered it may be necessary to dewater construction excavations to provide more suitable working conditions. Excavations should be well braced or sloped to prevent wall collapse. Federal, state and local safety codes should be observed. All construction excavations should conform to Occupational Safety and Health Administration (OSHA) standards or safer.

The site construction surface should be graded to drain surface water away from the site excavations. Surface water should not be allowed to accumulate in excavations during construction. Accumulated water could negatively influence the site soil conditions. Construction surface drainage should include swales, if necessary to divert surface water away from the construction excavations.

Organic soil materials in areas to receive fill material or structural components should be removed. The organic soil materials are not suitable for support of the structure or structural components.

It has been our experience that sites in developed areas may contain existing subterranean structures or poor quality man placed fill. If subterranean structures or poor quality man placed fill are suspected or encountered, they should be removed and replaced with compacted structural fill as discussed under COMPACTED STRUCTURAL FILL below.

Any loose soil or voids caused by prairie dog mounds/burrows should be removed and replaced with properly conditioned and properly compacted materials.

The soil materials exposed in the bottom of the excavation may be moist and may become yielding under construction traffic during construction. It may be necessary to use techniques for placement of fill material or foundation concrete which limits construction traffic in the vicinity of the very moist soil material. If yielding should occur during construction it may be necessary to construct a subgrade stabilization fill blanket or similar to provide construction traffic access. The subgrade stabilization blanket may include over excavating the subgrade soils one (1) to several feet and replacing with aggregate subbase course type material. The stabilization blanket may also include geotextile stabilization fabric at the bottom of the excavation prior to placement of aggregate subbase course stabilization fill. Other subgrade stabilization techniques may be available. We are available to discuss this with you.

5.0 FOUNDATION RECOMMENDATIONS

Geotechnical engineering considerations which influence the foundation design and construction recommendations presented below are discussed in Appendix D.

We have analyzed spread footing foundations and micro-pile foundation systems as potential foundation systems for the proposed structure. These are discussed below. Due to the number of possible foundation types available and design and construction techniques there may be design alternatives which we have not presented in this report.

We recommend that the entire structure be supported on only one foundation type.

Combining foundation types will result in differential and unpredictable foundation performance between the varying foundation types. We recommend that the structure footprint not be traversed by the cut/fill contact which would result in a portion of the structure underlain by fill material and part of the structure underlain by materials exposed by excavated cut. If this condition will exist please contact us so that we can revise our recommendations to accommodate the cut/fill contact scenario.

All of the design parameters presented below are based on techniques performed by an experienced competent contractor and high quality craftsmanship and care during construction. We recommend post construction cognizance of the volume change potential of the near surface soil materials and the need for appropriate post construction maintenance.

The spread footing recommendations include recommended design and construction techniques to reduce the influence of movement of the soil materials supporting the foundation but should not be interpreted as solutions for completely mitigating the potential for movement from the support soil material volume change.

Exterior column supports should be supported by foundations incorporated into the foundation system of the structure not supported on flatwork. Column supports placed on exterior concrete flatwork may move if the support soils below the concrete slab on grade become wetted and swell or freeze and raise or settle. Differential movement of the exterior columns may cause stress to accumulate in the supported structure and translate into other portions of the structure.

5.1 Spread Footing Foundations

In our analysis it was necessary to assume that the materials encountered in the test borings extended throughout the building site and to a depth below the maximum depth of the influence of the foundations. We should be contacted to observe the soil materials exposed in the foundation excavations prior to placement of foundations to verify the assumptions made during our analysis.

The bottom of the foundation excavations should be thoroughly cleaned and observed when excavated. Any loose or disturbed material exposed in the foundation excavation should be removed or compacted prior to placing foundation concrete.

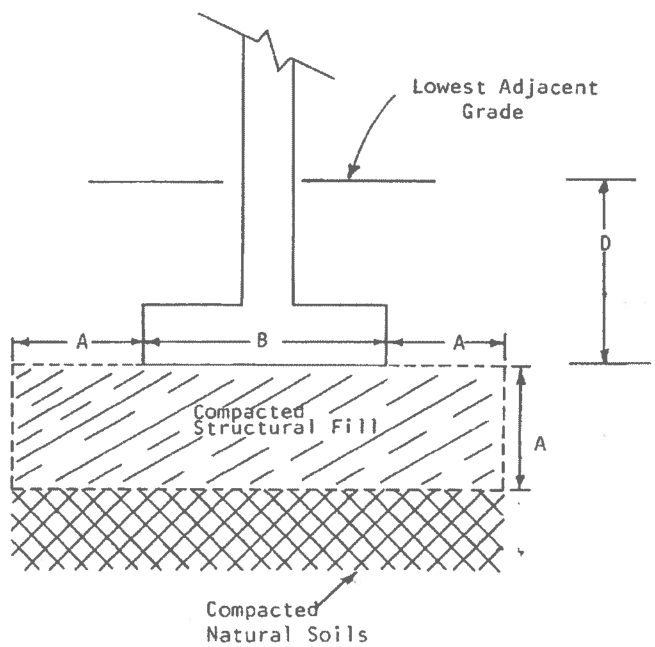
The bottom of the foundation excavations should be moisture conditioned and compacted prior to placing compacted structural fill. We suggest the materials exposed be moisture conditioned to within the range of optimum moisture to approximately four (4) percent wet of optimum moisture content and compacted to at least ninety-two (92) percent of the materials moisture content-dry density relationship (Proctor) test, ASTM D1557. Excavation compaction is to help reduce the influence of any disturbance that may occur during the excavation operations. Any areas of loose, low density or yielding soils evidenced during the excavation compaction operation should be removed and replaced with compacted structural fill. Caution should be exercised during the excavation compaction operations. Excess rolling or compacting may increase pore pressure of the subgrade soil material and degrade the integrity of the support soils. Loose or disturbed material in the bottom of the foundation excavations which are intended to support structural members will likely result in large and unpredictable amounts of settlement, if the loose or disturbed material is not removed or compacted.

The bottom of any footings exposed to freezing temperatures should be placed below the maximum depth of frost penetration for the area. Refer to the local building code for details.

All footings should be appropriately proportioned to reduce the post construction differential settlement. Footings for large localized loads should be designed for bearing pressures and footing dimensions in the range of adjacent footings to reduce the potential for differential settlement. We are available to discuss this with you.

Foundation walls should be reinforced for geotechnical engineering purposes. The structural engineer should be consulted for foundation design. The structural engineering reinforcing design tailored for this project will be more appropriate than the suggestions presented above.

We recommend the use of a blanket of structural fill material beneath the spread footing foundation members. We suggest the spread footings be placed on



B = Footing Width
A = Compacted Structural Fill Thickness and Fill Width Beyond Footing Edge
D = Footing Embedment Below Lowest Adjacent Grade

Figure 3. Conceptual Sketch of Footing Subgrade Treatment and Embedment Concept

a blanket of compacted structural fill extending to the native soil materials. The blanket of compacted structural fill is to help provide uniform support for the footings and to help reduce the theoretical calculated post construction settlement. The theoretical calculated post construction settlement and associated fill thickness supporting the footings are presented below.

We suggest that you consider the foundation be supported on a blanket of compacted structural fill material to help mask the influence of volume change soil materials supporting the footings. The blanket of compacted structural fill will not prevent movement of the footings from volume change in the support soil materials but will mask the influence of volume changes of the soils supporting the footings. If the footings are supported on a blanket of compacted structural fill the blanket of compacted structural fill should extend beyond each edge of each footing a distance at least equal to the fill thickness. This concept is shown on Figure 3. Geotechnical engineering recommendations for constructing compacted structural fill are presented below.

All footings should have a minimum depth of embedment of at least one (1) foot below the lowest adjacent grade when placed on a blanket of compacted structural fill. Deeper embedment will be needed for footings exposed to exterior climate.

Continuous and Isolated Spread Footing foundation members bearing upon a minimum two (2) foot thickness of properly placed and compacted structural fill material with a minimum embedment of one (1) foot should be designed using an allowable bearing capacity of 1,500 pounds per square foot.

If deeper embedment is considered for increased bearing capacity greater than presented above, we should be contacted to provide additional analysis and recommendations as needed. The bearing capacity design value is based on several considerations and these may change with depth.

The bearing capacity may be increased by about twenty (20) percent for transient loads such as wind and seismic loads.

It is our opinion that footings exposed to frost or freezing ground influences and all exterior footings should be embedded to frost depth or deeper. Interior footings should have a minimum depth of embedment of at least one (1) foot on all sides to provide a more predictable long term performance of the footing. It is our opinion that the performance of footings constructed without embedment may be influenced by erosion, temperature changes, moisture content changes, swell potential of the soil supporting the footings and weathering of the soils supporting the footings and will have a less predictable settlement response than footings with embedment.

Exterior footings and footings with uneven backfill may result in movement of the footings. Embedment of the footings on all sides will help reduce the potential for movement of footings with uneven backfill. We do not recommend exterior footings or footings with uneven backfill be constructed without a minimum depth of embedment of the bottom of the footing below the lowest adjacent grade of at least one (1) foot on all sides of the interior footings and frost depth for exterior footings.

The minimum depth of embedment is sufficient only to develop the bearing capacity for design purposes and does not account for frost influences. Actual design and construction should result in interior footings with one (1) foot or more embedment and exterior footings with frost depth or more embedment. Typically deeper embedment will increase bearing capacity and decrease post construction settlement and decrease the influence of expansive soils.

The soil samples of the shallow silty clay materials tested had measured swell pressures of less than 100 to approximately 600 pounds per square foot, however, the actual swell pressure of the support materials could be greater. When wetted the site soil materials may have the ability to raise supported foundation members with loads less than the swell pressure. The foundation design should be as rigid as possible with as high of a dead load as can be available. The greater the dead load on the footings the less the potential for movement from the foundation soils should they become wetted. If the soils become wetted they may swell and may raise the foundation portions supported on the wetted soils. If the structure is supported on spread footings the owner must realize that post construction movement of the footings is likely. We are available to discuss the implications of supporting foundations on swelling soils.

Interior column loads supported on spread footings which are structurally connected to the other foundation members will provide more uniform performance of the interior footings with respect to the other foundation members and will help reduce the potential differential settlement between interior and exterior foundation members. The foundation walls should be designed to act as beams to distribute stresses associated with the swelling volume changes of soils. The beam design should be addressed by the project structural engineer.

Exterior column supports should be supported by foundations incorporated into the foundation system of the structure not supported on flatwork. Column supports placed on exterior concrete flatwork may move if the support soils below the concrete slab on grade become wetted and swell or freeze and raise or settle. Differential movement of the exterior columns may cause stress to accumulate in the supported structure and translate into other portions of the structure.

The calculated theoretical estimated post construction settlement and swell potential may be reduced by placing the footings on a blanket of compacted structural fill. The calculated theoretical estimated post construction settlement and associated thickness of compacted structural fill are presented below.

THICKNESS OF COMPACTED STRUCTURAL FILL SUPPORTING FOOTINGS	CALCULATED THEORETICAL ESTIMATED POST CONSTRUCTION SETTLEMENT FOR CONTINUOUS SPREAD FOOTINGS (INCHES)
0	2-1/4 to 3
1 foot	1-5/8 to 2-1/8
2 feet	1-1/8 to 1-3/8
3 feet	5/8 to 7/8

THICKNESS OF COMPACTED STRUCTURAL FILL SUPPORTING FOOTINGS	CALCULATED THEORETICAL ESTIMATED POST CONSTRUCTION SETTLEMENT FOR ISOLATED SPREAD FOOTINGS (INCHES
0	2-1/8 to 2-7/8
1 foot	1-1/2 to 2
2 feet	1 to 1-1/4
3 feet	5/8 to 3/4

The calculated theoretical settlement estimated values above are appropriate for continuous spread footings with a width of about two (2) feet or less and isolated spread footings with a width of about three (3) feet or less. Larger footings should be analyzed on a footing, load and width specific basis.

Footings should be sized so that each footing is in a similar size and load range as nearby footings to encourage similar performance. Very large footings or heavily loaded footings will influence the support soil materials to a deeper depth than small or lightly loaded footings and therefore will have different post construction performance characteristics.

The calculated settlement estimates are theoretical only. Actual settlement could vary throughout the site and with time.

If the footings are supported on a blanket of compacted structural fill, the blanket of compacted structural fill should extend beyond each edge of each footing a distance at least equal to the fill thickness. This concept is shown on Figure 3. Compacted Structural Fill is discussed in Section 8.0 below.

The soil samples of the shallow silty clay materials tested had measured swell pressures of less than 100 to approximately 600 pounds per square foot, however, the actual swell pressure of the support material could be greater. This swell pressure was measured for soils at the initial moisture content of the soil sample tested. The swell potential of the site soil materials could vary significantly and could be greater than that measured. The measured swell pressure may be influenced by disturbance of the sample during the sampling operation and the soil suction potential and initial moisture content.

Changes in the initial moisture content will significantly influence the swell pressure of the site soils. If the initial moisture content of the foundation soils is less than that of the test sample the actual swell pressures will likely be significantly higher than measured. If the initial moisture content of the foundation soils is greater than that of the test sample the actual swell pressures may be less than measured.

The bottom of the foundation excavations should be thoroughly cleaned and observed by the project Geotechnical Engineer or his representative when excavated. Any loose or disturbed material exposed in the foundation excavation should be removed or remedied prior to additional construction.

We recommend that we be contacted to observe the foundation excavations and backfill operations during construction to verify the soil support conditions and our assumptions upon which our recommendations are based. If necessary we may revise our recommendations based on our observations. We are available to provide material testing services during the construction phase of the project.

If lightly loaded structure members are supported on spread footings on expansive soil material then the owner must realize that post construction movement of the footings is likely. These lightly loaded areas of the footing should be designed with sufficient structural integrity to resist the forces from swelling soils.

Foundation members that will have significantly small or low dead loads, such as foundations beneath wall openings such as doorways, may be provided with a strengthened grade beam and/or positive separation between the foundation concrete and the underlying soil materials. That separation may be provided by using commercial void form material. We recommend that the structural engineer be consulted concerning the void form design concept.

If the void form design concept is part of the foundation design we suggest that the foundation design may consider including a four (4) inch corrugated paper void form material beneath the footings in the lightly loaded portions of the foundation. The corrugated paper void forms provide temporary support for foundation concrete during construction. The low strength of the void form material is intended to allow the underlying soil materials to expand into the void form thereby exerting less or no uplift pressure on the foundation in the areas it is used. We are available to discuss the implications of supporting foundations on swelling soils.

5.2 Grouted Micro Pile Foundations

If the hollow bar/pressure grout method is used to install the micro piles, we suggest that the micro piles with an approximate annulus of four inches be designed using an allowable design capacity of 3,000 pounds per foot of bond length in the competent formational shale material.

We suggest a minimum bond length of fifteen (15) feet in the competent formational shale materials. The steel tendon diameter should be determined by the structural engineer based on the required load criteria. The grout strength used should have a minimum compressive strength of 4,000 psi after twenty-eight (28) days. The micro pile ultimate capacity will not be achieved until the grout has properly cured.

If the micro piles are designed and constructed as discussed above we anticipate that the post construction settlement potential of each pile may be less than about three quarters (3/4) inch.

We recommend load testing of control piles be conducted before actual production piles.

The structural engineer should be consulted to provide structural design recommendations for the micro pile foundation system.

In our analysis it was necessary to assume that the material encountered in the test borings extended throughout the building site and to a depth below the maximum depth of the influence of the foundations. We should be contacted to observe the soil materials exposed in the foundation excavations prior to placement of foundations to verify the assumptions made during our analysis.

6.0 INTERIOR FLOOR SLAB DISCUSSION

It is our understanding that, as currently planned, the floor may be either a concrete slab on grade or a supported structural floor. The natural soils that will support interior floor slabs are stable at their natural moisture content. However, the owner should realize that when wetted, the site soils may experience volume changes. The soil samples of the shallow silty clay materials tested had measured swell pressures of less than 100 to approximately 600 pounds per square foot and associated magnitudes of up to 0.9 percent of the wetted soil volume at a surcharge load of 100 pounds per square foot and the actual swell pressure could be greater.

Conditions which vary from those encountered during our field study may become apparent during excavation. We should be contacted to observe the conditions exposed at concrete slab on grade subgrade elevation to verify the assumptions made during the preparation of this report and to provide additional geotechnical engineering suggestions and recommendations as needed.

Engineering design dealing with swelling soils is an art which is still developing. The owner is cautioned that the soils on this site may have swelling potential and concrete slab on grade floors and other lightly loaded members may experience movement when the supporting soils become wetted. If the owner is willing to accept the risk of possible damage from swelling soils supporting concrete slab on grade floors, the following recommendations to help reduce the damage from swelling soils should be followed. These recommendations are based on generally accepted design and construction procedures for construction on soils that tend to experience volume changes when wetted and are intended to help reduce the damage caused by swelling soil materials. Lambert and Associates does not intend that the owner, or the owner's consultants should interpret these recommendations as a solution to the problems of swelling soils, but as measures to reduce the influence of swelling soils.

The shallow soil materials tested have a low to moderate volume change potential under light loading conditions. Concrete slab on grade floors may experience movement when supported by the natural onsite soils. Concrete slab on grade floors will perform best if designed to tolerate movement introduced by the subgrade soil materials.

Concrete flatwork, such as concrete slab on grade floors, should be underlain by compacted structural fill. The layer of compacted fill should be at least two (2) feet thick or thicker and constructed as discussed under COMPACTED STRUCTURAL FILL below. A two (2) foot thick or thicker blanket of structural fill material beneath the concrete flatwork is not sufficient to entirely mask the settlement or swell potential of the subgrade soil material but will only provide better subgrade conditions for construction. The concrete slab on grade should be designed by a structural engineer to be compatible with the site soil conditions.

The natural soil materials exposed in the areas supporting concrete slab on grade floors should be kept very moist during construction prior to placement of concrete slab on grade floors. This is to help increase the moisture regime of the potentially expansive soils supporting floor slabs and help reduce the expansion potential of the soils. We are available to discuss this concept with you.

Concrete slab on grade floors should be provided with a positive separation, such as a slip joint, from all bearing members and utility lines to allow their independent movements and to help reduce possible damage that could be caused by movement of soils supporting interior slabs. The floor slab should be constructed as a floating slab. All water and sewer pipe lines should be isolated from the slab. Any equipment placed on the floating floor slab should be constructed with flexible joints to accommodate future movement of the floor slab with respect to the structure. We suggest partitions constructed on the concrete slab on grade floors be provided with a void space above or below the partitions to relieve stresses induced by elevation changes in the floor slab.

Floor slabs should not contact/extend directly over foundations or foundation members. Floor slabs which directly contact foundations or foundation members will likely experience post construction movement as a result of foundation movements. We are available to discuss this with you.

The concrete slabs should be scored or jointed to help define the locations of any cracking. We recommend that joint spacing be designed as outlined in ACI 224R. In addition joints should be scored in the floors a distance of about three (3) feet from, and parallel to, the walls.

It should be noted that when curing fresh concrete experiences shrinkage. This shrinkage almost always results in some cracks in the finished concrete. The actual shrinkage depends on the configuration and strength of the concrete and placing and finishing techniques. The recommended joints discussed above are intended to help define the location of the cracks but should not be interpreted as a solution to shrinkage cracks. The owner must understand that concrete flatwork will contain shrinkage cracks after curing and that all of the shrinkage cracks may not be located in control joints. Some cracking at random locations may occur.

If moisture migration through the concrete slab on grade floors will adversely influence the performance of the floor or floor coverings we suggest that a moisture barrier may be installed beneath the floor slab to help discourage capillary and vapor moisture rise through the floor slab. The moisture barrier may consist of a heavy plastic membrane, six (6) mil or greater, protected on the top and bottom by clean sand. The clean sand will help to protect the plastic from puncture. The layer of clean sand on the top of the plastic membrane will help the overlying concrete slab cure properly. According to the American Concrete Institute, proper curing requires at least three (3) to six (6) inches of clean sand between the plastic membrane and the bottom of the concrete. The plastic membrane should be lapped and taped or glued and protected from punctures during construction.

If the moisture content of the slab on grade floor will be influencial to the performance of the future floor coverings then the moisture content of the slab can be measured. We are available to monitor the floor slab moisture content prior to the installation of the floor covering. If this service is needed please contact us during the construction phase of the project.

The Portland Cement Association suggests that welded wire reinforcing mesh is not necessary in concrete slab on grade floors when properly jointed. It is our opinion that welded wire mesh may help improve the integrity of the slab on grade floors. We suggest that concrete slab on grade floors should be reinforced, for geotechnical purposes, with at least 6 x 6 - W2.9 x W2.9 (6 x 6 - 6 x 6) welded wire mesh positioned midway in the slab. The structural engineer should be contacted for structural design of floor slabs.

7.0 PAVEMENT SECTION DESIGN RECOMMENDATIONS

It is our understanding that the proposed development will include paved parking and drive areas. The paved areas will include asphalt paved parking areas, concrete paved aprons and concrete sidewalks. Our pavement section analysis was based on estimated traffic volumes, laboratory test results of the soils sampled during our field study, and on our experience on similar projects. The traffic volume used in our analysis assumed 18,000 pound equivalent single axle load (ESALs) of 100,000 repetitions for a twenty (20) year life. Our analysis included pavement sections based on dynamic loading as discussed in the Colorado Department of Transportation 2014 Pavement Design Manual.

7.1 Subgrade Preparation

Proper performance of the subgrade support soils requires surface preparation, scarification and moisture conditioning, compaction, and surface and subsurface drainage during construction prior to placement of the overlying pavement section materials.

Subgrade preparation may result in areas which yield under construction traffic. If yielding areas are encountered during subgrade preparation in the paved areas, the subgrade material may be overexcavated to a depth of about one foot below the subgrade elevation or more if needed and backfilled with a compacted structural fill. The structural fill material may aid in construction of the paved areas subgrade. The structural fill material should be an aggregate subbase course or aggregate base course type material placed and compacted as discussed below.

All organic and other deleterious material should be removed from the areas proposed for pavement section construction. The soils exposed by the removal of the organic materials should be scarified to a depth of about twelve (12) inches, moisture conditioned to approximately one (1) to five (5) percent above optimum moisture content, and compacted to at least ninety-two (92) percent of maximum dry density as defined by ASTM D1557, modified moisture content-dry density relationship (Proctor) test. The moisture conditioning may require addition of water, or air drying if the soil is too moist, in either case, the material should be sufficiently mixed to promote a uniform soil moisture content. The soils should be compacted using machinery designed for soil compaction. Wheel rolling with loaded equipment and other techniques may not provide a uniform, properly compacted roadway subgrade.

Utility trench backfill in areas supporting pavement or other structural components should be placed in thin lifts and compacted to at least ninety-two (92) percent of the maximum dry density as defined by ASTM D1557 to subgrade elevation.

After the subgrade soils have been prepared the surface should be crowned or surface graded in the same orientation as the proposed final surface of the asphalt pavement. The reason for this is to promote water migration away from the roadway more readily. If the subgrade soil surface is not graded to properly drain, water may accumulate within the pavement section soils. The increased moisture content and subsequent soil strength decrease may promote pavement section support degradation. If a full section asphalt concrete design is used, the subgrade soils should be graded parallel the final asphalt concrete surface for drainage so that a uniform asphalt concrete thickness exists.

7.2 Aggregate Sub-Base and Base Course Material Characteristics and Placement

Specific aggregate types and sources for potential use on the project were not known at the time of the preparation of this report. Our analysis assumed that the proposed aggregate base course would consist of a Class 6 type material, and the aggregate sub-base course would consist of a Class 2 type material, as designated in the "Colorado Department of Highways Standard Specification for Road and Bridge Construction", 1991. If it is desirable to use material which does not meet these criteria we should be contacted to assess the specific material characteristics of the proposed road base and provide additional pavement design sections for differing materials.

The aggregate sub-base and base course materials should be placed on the prepared subgrade soils as soon as possible after the subgrade soils are compacted and graded to drain. Placement of the aggregate materials will help limit the influence of construction and other traffic on the subgrade soil conditions.

The aggregate materials should not be allowed to become segregated either at the source, prior to hauling to the project site, or during the placement of the materials. The coarser aggregate sub-base soils have a greater tendency to become segregated, particularly during the grading and placement operations. Segregated sub-base and base course do not provide as uniform support as well blended materials.

The sub-base and base course materials should be moisture conditioned and compacted to at least ninety-five (95) percent of maximum dry density as defined by ASTM D1557, modified moisture-content-dry density relationship (Proctor) test.

7.3 Asphalt Concrete Materials and Placement

The asphalt concrete should be prepared using a mix design which has been prepared by a professional engineer experienced in asphalt concrete materials. The mix design should establish, as a minimum, the quality of the aggregates used, asphalt concrete material properties, asphalt cement content, mix and lay down temperatures. Either the Marshall Method or Hveem Stabilometer method of mix design may be used for the mix design preparation. We suggest that the asphalt concrete be compacted to between ninety-two (92) and ninety-six (96) percent of the maximum mix design density.

Aggregate shape maximum size and particle size distribution are important factors influencing the performance of an asphalt concrete mix. Crushed aggregate with fractured faces and angular shapes tend to interlock and provide an asphalt concrete with high strength and limited flexibility. Natural aggregates with rounded shapes tend to provide an asphalt concrete which is more flexible and may have lower strengths than mixes produced with angular shaped aggregates. Incorrect particle or grain size distribution of the aggregate used to manufacture the asphalt concrete can result in poor performance of the in-place asphalt mix. The grain size distribution of the mix aggregate will influence the size and volume of voids and the stability of the asphalt mix. Verification of the asphalt mix design aggregate properties and the asphalt concrete mix should be performed by testing prior to and during the paving operation.

7.4 Flexible Pavement Design Sections

An estimated "CBR" value of 0.9 was used in our analysis. Alternative pavement sections are presented below. The pavement thickness sections below are based on the Design Nomograph for Flexible Pavements as recommended in the Colorado Department of Transportation 2014 pavement Design Manual.

Construction traffic may have a greater influence on the performance of the pavement section than the commercial use after construction. The design recommendations presented below are based on typical post construction commercial use and do not include accommodation for heavy loading as a result of construction traffic. It may be beneficial to consider partial pavement section construction for use during on-site development construction with the section repaired and completed after the heavy construction traffic use has ended. This technique may provide a more serviceable and structurally acceptable pavement for the completed project.

PAVEMENT THICKNESS DESIGN SECTIONS
*ESAL = 100,000

Asphalt Concrete (inches)	Aggregate Base Course Class 6 or Similar (inches)	Aggregate Subbase Course Class 2 or Similar (inches)	Reconditioned Subgrade (inches)
3	6	21	12
3	10	15	12
4	6	16	12
4	10	11	12

* Equivalent 18,000 pounds single axle load

Pavement thickness design section of less than three (3) inches of asphalt over aggregate base course may be used, although, because of the shorter life before maintenance and the relatively poor long term performance, we suggest that this be considered as an intermediate design section only. If a lesser design section is used we suggest you consider a later asphalt overlay of appropriate thickness to extend the life of the pavement section. The overlay should be constructed prior to any visible distress occurring in the pavement.

The asphalt concrete pavement should be placed on the prepared support section as soon as possible so that interim traffic does not decrease the integrity of the support section.

7.5 Rigid Pavement Thickness Design Recommendations

Our pavement thickness recommendations for rigid Portland cement concrete pavement are based on an assumed traffic volume, a modulus of rupture of 650 psi and a modulus of subgrade reaction obtained from the California Bearing Ratio test performed on the subgrade soil sample obtained during our field study. A modulus of subgrade reaction of 47 psi/inch was used in our analysis.

The rigid pavement that will see light vehicle traffic may be designed using a concrete thickness of five (5) inches for an estimated 18,000 pounds equivalent single axle load (ESAL) less than 100,000.

Rigid pavement in areas that will see heavy traffic loads, fire trucks, delivery trucks, school busses, garbage trucks, etc., may be designed using a minimum concrete thickness of eight (8) inches.

Concrete sidewalks should have a nominal thickness of four (4) inches if no vehicle traffic will be allowed on them. The concrete sidewalks and aprons may be placed on a leveling course of aggregate base course material. The leveling course should be at least four (4) inches thick and compacted as discussed above for aggregate base course. We suggest the use of rigid pavement in areas where heavy vehicles will be accelerating, decelerating or turning sharply, such as the aprons for the trash dumpster enclosures and any areas in the bus drop off/pick up areas where the buses will be turning.

The concrete should be supported on prepared subgrade which is at least twelve (12) inches thick. The prepared subgrade should consist of either compacted structural fill to establish subgrade elevation or natural soils which are scarified to a depth of twelve (12) inches, moisture conditioned to approximately one (1) to five (5) percent above optimum moisture content and recompacted to at least ninety-two (92) percent of the maximum dry density as defined by ASTM D1557, modified moisture content-dry density relationship test. If during subgrade preparation any loose or yielding area or any areas of poorly constructed man-placed fill are encountered they should be removed and replaced with compacted structural fill. Suggestions for constructing compacted structural fill are presented below.

The Portland cement concrete should be from an approved concrete mix design stating the proportions and mixtures of the mix. We recommend verification of the mix design prior to paving. The coarse and fine aggregate used in the concrete mix should be tested for their suitability for use as concrete aggregate.

The concrete pavement should be appropriately jointed and structurally reinforced to help control the location of cracking. The structural engineer should be contacted to provide structural design recommendations or structural reinforcement and joint design of the concrete pavement.

8.0 COMPACTED STRUCTURAL FILL

Material characteristics desirable for compacted structural fill are discussed in Appendix D. Areas that are over excavated or slightly below grade should be backfilled to grade with properly compacted structural fill or concrete, not loose fill material. If backfilled with other than compacted structural fill material or concrete there will be significant post construction settlement proportional to the amount of loose material.

The natural on site shallow clayey soils are not suitable for use as compacted structural fill material supporting building or structure members because of their clay content and swell potential. The natural on-site soils may be used as compacted fill in areas that will not influence the structure such as to establish general site grade. We are available to discuss this with you.

All areas to receive compacted structural fill should be properly prepared prior to fill placement. The preparation should include removal of all organic or deleterious material. The areas to receive fill material should be compacted after the organic deleterious material has been removed prior to placing the fill material. The area may need to be moisture conditioned for compaction. Any areas of soft, yielding, or low density soil, evidenced during the excavation compaction operation should be removed. The area excavated to receive fill should be moisture conditioned to within approximately one (1) to five (5) percent above optimum moisture content and recompacted to at least ninety-two (92) percent of the maximum dry density as defined by ASTM D1557 as part of the preparation to receive fill. Fill should be moisture conditioned, placed in thin lifts not exceeding six (6) inches in compacted thickness and compacted to at least ninety-two (92) percent of maximum dry density as defined by ASTM D1557, modified moisture content-dry density (Proctor) test.

After placement of the structural fill the surface should not be allowed to dry prior to placing concrete or additional fill material. This may be achieved by periodically moistening the surface of the compacted structural fill as needed to prevent drying of the structural fill. We are available to discuss this with you.

The soil materials exposed in the bottom of the excavation may be very moist and may become yielding under construction traffic during construction. It may be necessary to use techniques for placement of fill materials or foundation concrete which limit construction traffic in the very moist soil materials. If yielding should occur during construction it may be necessary to construct a subgrade stabilization fill blanket or similar to provide construction traffic access.

Structural fills with thicknesses greater than one (1) foot should incorporate a subgrade stabilization fabric material on top of the prepared native material prior to the placement of structural fill material or on top of the first foot of properly placed and compacted structural fill material.

We recommend that the geotechnical engineer or his representative be present during the excavation compaction and fill placement operations to observe and test the material.

9.0 LATERAL EARTH PRESSURES

Laterally loaded walls supporting soil, such as basement walls, will act as retaining walls and should be designed as such. Walls that are designed to deflect and mobilize the internal soil strength should be designed for active earth pressures. Walls that are restrained so that they are not able to deflect to mobilize internal soil strength should be designed for at-rest earth pressures. The values for the lateral earth pressures will depend on the type of soil retained by the wall, backfill configuration and construction technique. If the backfill is not compacted the lateral earth pressures will be very different from those noted below.

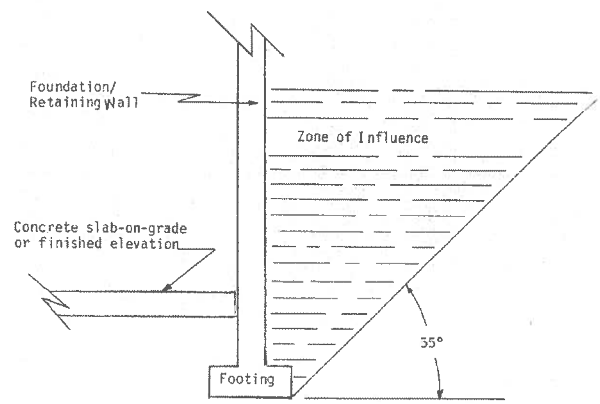


Figure 4. Backfill Zone of Influence Concept

Lateral earth pressure (L.E.P.) values are presented below:

	Level Backfill with on-site soils (pounds per cubic foot per foot of depth)
Active L.E.P.	68
At-rest L.E.P.	85
Passive L.E.P.	195

The soil samples tested had measured swell pressures of less than 100 to approximately 600 pounds per square foot and the actual swell pressure of the backfill material could be greater. Our experience has shown that the actual swell pressure may be higher. If the retained soils should become moistened after construction the soil may swell against retaining walls. The walls should be designed to resist the swell pressure of the soil materials if these are used as part of the backfill within the zone of influence. The zone of influence concept is presented on Figure 4.

The above lateral earth pressures may be reduced by overexcavating the wall backfill area beyond the zone of influence and backfilling with crushed rock type material.

The lateral earth pressure design parameters may change significantly if the area near the wall is loaded or surcharged or is sloped. If any of these conditions occur we should be contacted for additional design parameters tailored to the specific site and structure conditions.

Suggested lateral earth pressure (L.E.P.) values if the backfill is overexcavated beyond the zone of influence and backfilled with crushed rock are presented below.

	Level Backfill with crushed rock material (pounds per cubic foot per foot of depth)
Active L.E.P.	25
At-rest L.E.P.	40

If the area behind a wall retaining soil material is sloped we should be contacted to provide lateral earth pressure design values tailored for the site specific sloped conditions.

Resistant forces used in the design of the walls will depend on the type of soil that tends to resist movement. We suggest that you consider a coefficient of friction of 0.17 for the on site soil.

The lateral earth pressure values provided above, for design purposes, should be treated as equivalent fluid pressures. The lateral earth pressures provided above are for level well drained backfill and do not include surcharge loads or additional loading as a result of compaction of the backfill. Unlevel or non-horizontal backfill either in front of or behind walls retaining soils will significantly influence the lateral earth pressure values. Care should be taken during construction to prevent construction and backfill techniques from overstressing the walls retaining soils. Backfill should be placed in thin lifts and compacted, as discussed in this report to realize the lateral earth pressure values.

Walls retaining soil should be designed and constructed so that hydrostatic pressure will not accumulate or will not affect the integrity of the walls. Drainage plans should include a subdrain behind the wall at the bottom of the backfill to provide positive drainage. Exterior retaining walls should be provided with perimeter drain or weep holes to help provide an outlet for collected water behind the wall. The ground surface adjacent to the wall should be sloped to permit rapid drainage of rain, snow melt and irrigation water away from the wall backfill. Sprinkler systems should not be installed directly adjacent to retaining or basement walls.

10.0 DRAIN SYSTEM

A drain system should be provided around building spaces below the finished grade and behind any walls retaining soil. The drain systems are to help reduce the potential for hydrostatic pressure to develop behind retaining walls. A sketch of the drain system is shown on Figure 5.

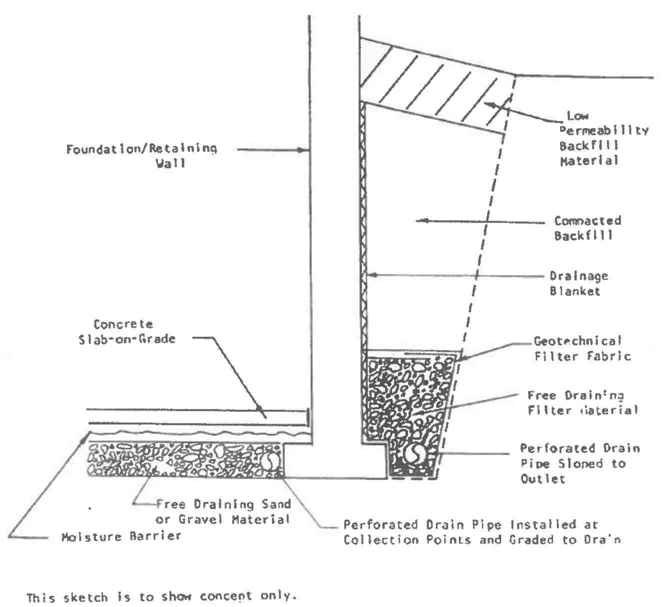


Figure 5. Conceptual Sketch of Foundation Drain System

Subdrains should consist of a three (3) or four (4) inch diameter perforated rigid pipe surrounded by a filter. The filter should consist of a filter fabric or a graded material such as washed concrete sand or pea gravel. If sand or gravel is chosen the pipe should be placed in the middle of about four (4) cubic feet of aggregate per linear foot of pipe. The drain system should be sloped to positive gravity outlets. If the drains are daylighted the drains should be provided with all weather outlets and the outlets should be maintained to prevent them from being plugged or frozen. We do not recommend that the drains be discharged to dry well type structures. Dry well structures may tend to fail if the surrounding soil material becomes wetted and swells or if the ground water rises to a elevation of or above the discharge elevation in the dry well. We should be called to observe the soil exposed in the excavations and to verify the details of the drain system.

11.0 BACKFILL

Backfill areas and utility trench backfill should be constructed such that the backfill will not settle after completion of construction, and that the backfill is relatively impervious for the upper few feet. The backfill material should be free of trash and other deleterious material. It should be moisture conditioned and compacted to at least ninety-two (92) percent relative compaction using a modified moisture content-dry density (Proctor) relationship test (ASTM D1557). Only enough water should be added to the backfill material to allow proper compaction. Do not pond, puddle, float or jet backfill soil materials.

Improperly placed backfill material will allow water migration more easily than properly recompacted fill. Improperly compacted fill is likely to settle, creating a low surface area which further enhances water accumulation and subsequent migration to the foundation soils.

Improperly placed backfill will allow water to migrate along the utility trench or backfill areas to gain access to the subgrade support soils with subsequent mobilization of the swell or settlement mechanism resulting in movement of the supported structure. Moisture migration could also result in the inconvenience of free water in the crawl space.

Backfill placement techniques should not jeopardize the integrity of existing structural members. We recommend recently constructed concrete structural members be appropriately cured prior to adjacent backfilling.

12.0 SURFACE DRAINAGE

The foundation soil materials should be prevented from becoming wetted after construction. Post construction wetting of the soil support soil materials can initiate swell potential or settlement potential as well as decrease the bearing capacity of the support soil materials. Protecting the foundation from wetting can be aided by providing positive and rapid drainage of surface water away from the structure.

The final grade of the ground surface adjacent to the structure should have a well defined slope away from the foundation walls on all sides. The ability to establish proper site surface drainage away from the structure foundation system may be influenced by the existing topography, existing structure elevations and the grades and elevations of the ground surface adjacent to the proposed structure. We suggest where possible a minimum fall of the surface grade away from the structure be that which will accommodate other project grading constraints and provide rapid drainage of surface water away from the structure. If there are no other project constraints we suggest a fall of about one (1) foot in the first ten (10) feet away from the structure foundation. Appropriate surface drainage should be maintained for the life of the project. Future landscaping plans should include care and attention to the potential influence on the long term performance of the foundation if improper surface drainage is not maintained.

Roof runoff should be collected in appropriate roof drainage collection devices, such as eve gutters or similar, and directed to discharge in appropriate roof drainage systems. Roof runoff should not be allowed to fall on or near foundations, backfill areas, flatwork, paved areas or other structural members. Downspouts and faucets should discharge onto splash blocks that extend beyond the limits of the backfill areas. Splash blocks should be sloped away from the foundation walls. Snow storage areas should not be located next to the structure. Proper surface drainage should be maintained from the onset of construction through the proposed project life.

If significant water concentration and velocity occurs erosion may occur. Erosion protection may be considered to reduce soil erosion potential. A landscape specialist or civil engineer should be consulted for surface drainage design, erosion protection and landscaping considerations.

13.0 LANDSCAPE IRRIGATION

An irrigation system should not be installed next to foundations, concrete flatwork or paved areas. If an irrigation system is installed, the system should be placed so that the irrigation water does not fall or flow near foundations, flatwork or pavements. The amount of irrigation water should be controlled.

We recommend that wherever possible xeriscaping concepts be used. Generally, the xeriscape includes planning and design concepts which will reduce irrigation water. The reason we suggest xeriscape concepts for landscaping is because the reduced landscape water will decrease the potential for water to influence the long term performance of the structure foundations and flatwork. Many publications are available which discuss xeriscape. Colorado State University Cooperative Extension has several useful publications and most landscape architects are familiar with the subject.

Due to the expansive nature of the soils tested we suggest that the owner consider landscaping with only native vegetation which requires only natural precipitation to survive. Additional irrigation water will greatly increase the likelihood of damage to the structure as a result of volume changes of the material supporting the structure.

Impervious geotextile material may be incorporated into the project landscape design to reduce the potential for irrigation water to influence the foundation soils.

14.0 SOIL CORROSIVITY TO CONCRETE

The chemical tests to help identify the potential for soil corrosivity to concrete were not complete at the time of this report. The chemical tests will be presented when available.

It has been our experience that much of the soils in the area contain sufficient water soluble sulfate content to be corrosive to concrete. We suggest sulfate resistant cement be used in concrete which will be in contact with the on-site soils. American Concrete Institute recommendations for sulfate resistant cement based on the water soluble sulfate content should be used.

15.0 RADON CONSIDERATIONS

Our experience indicates that many of the soils in western Colorado produce small quantities of radon gas. Radon gas may tend to collect in closed poorly ventilated structures. Radon considerations are presented in Appendix D.

16.0 POST DESIGN CONSIDERATIONS

The project geotechnical engineer should be consulted during construction of the project to observe site conditions and open excavations during construction and to provide materials testing of soil and concrete.

This subsurface soil and foundation condition study is based on limited sampling; therefore, it is necessary to assume that the subsurface conditions do not vary greatly from those encountered in the field study. Our experience has shown that significant variations are likely to exist and can become apparent only during additional on site excavation. For this reason, and because of our familiarity with the project, Lambert and Associates should be retained to observe foundation excavations prior to foundation construction, to observe the geotechnical engineering aspects of the construction and to be available in the event any unusual or unexpected conditions are encountered. The cost of the geotechnical engineering observations and material testing during construction or additional engineering consultation is not included in the fee for this report. We recommend that your construction budget include site visits early during construction schedule for the project geotechnical engineer to observe foundation excavations and for additional site visits to test compacted soil.

We recommend that the observation and material testing services during construction be retained by the owner or the owner's engineer or architect, not the contractor, to maintain third party credibility. We are experienced and available to provide material testing services. We have included a copy of a report prepared by Van Gilder Insurance which discusses testing services during construction. It is our opinion that the owner, architect and engineer be familiar with the information. If you have any questions regarding this concept please contact us.

We suggest that your construction plans and schedule include provisions for geotechnical engineering observations and material testing during construction and your budget reflect these provisions.

It is difficult to predict if unexpected subsurface conditions will be encountered during construction. Since such conditions may be found, we suggest that the owner and the contractor make provisions in their budget and construction schedule to accommodate unexpected subsurface conditions.

16.1 Structural Fill Quality

It is our understanding that the proposed development may include compacted structural fill. The quality of compacted structural fill will depend on the type of material used as structural fill, fill lift thickness, fill moisture condition and compactive effort used during construction of the structural fill. Engineering observation and testing of structural fill is essential as an aid to safeguard the quality and performance of the structural fill.

Testing of the structural fill normally includes tests to determine the grain size distribution, swell potential and moisture-density relationship of the fill material to verify the material suitability for use as structural fill. As the material is placed the in-place moisture content and dry density are tested to indicate the relative compaction of the placed structural fill. We recommend that your budget include provisions for observation and testing of structural fill during construction.

Testing of the compacted fill material should include tests of the moisture content and density of the fill material placed and compacted prior to placement of additional fill material. We suggest that a reasonable number of density tests of the fill material can best be determined on a site, material and construction basis although as a guideline we suggest one test per about each 300 to 500 square feet of each lift of fill material. Utility trench backfill may need to be tested about every 100 linear feet of lift of backfill.

16.2 Concrete Quality

It is our understanding current plans include reinforced structural concrete for foundations and walls and may include concrete slabs on grade and pavement. To insure concrete members perform as intended, the structural engineer should be consulted and should address factors such as design loadings, anticipated movement and deformations.

The quality of concrete is influenced by proportioning of the concrete mix, placement, consolidation and curing. Desirable qualities of concrete include compressive strength, water tightness and resistance to weathering. Engineering observations and testing of concrete during construction is essential as an aid to safeguard the quality of the completed concrete.

Testing of the concrete is normally performed to determine compressive strength, entrained air content, slump and temperature. We recommend that your budget include provisions for testing of concrete during construction. We suggest that a reasonable frequency of concrete tests can best be determined on a site, materials and construction specific basis although as a guideline American Concrete Institute, ACI, suggests one test per about each fifty (50) cubic yards or portion thereof per day of concrete material placed.

17.0 LIMITATIONS

It is the owner's and the owner's representatives' responsibility to read this report and become familiar with the recommendations and suggestions presented. We should be contacted if any questions arise concerning the geotechnical engineering aspects of this project as a result of the information presented in this report.

The scope of services for this study does not include either specifically or by implication any environmental or biological (such as mold, fungi, bacteria, etc.) Assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be performed.

The proposed building site contains soil materials with significant swell potential. For this reason we suggest that you consult, as suggested by Senate Bill 13, a copy of Colorado

Geological Survey Special Publication 11, "Home Construction on Shrinking and Swelling Soils", and a copy of CGS Special Publication 14, "Home Landscaping and Maintenance on Swelling Soils". We are available to discuss this with you.

The recommendations outlined above are based on our understanding of the currently proposed construction. We are available to discuss the details of our recommendations with you and revise them where necessary. This geotechnical engineering report is based on the proposed site development and scope of services as provided to us by Mr. Billy Harris, Blythe Group + co. on the type of construction planned, existing site conditions at the time of the field study, and on our findings. Should the planned, proposed use of the site be altered, Lambert and Associates must be contacted, since any such changes may make our suggestions and recommendations inappropriate. This report should be used ONLY for the planned development for which this report was tailored and prepared, and ONLY to meet information needs of the owner and the owner's representatives. In the event that any changes in the future design or location of the building are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions of this report are modified or verified in writing. It is recommended that the geotechnical engineer be provided the opportunity for a general review of the final project design and specifications in order that the earthwork and foundation recommendations may be properly interpreted and implemented in the design and specifications.

This report does not provide earthwork specifications. We can provide guidelines for your use in preparing project specific earthwork specifications. Please contact us if you need these for your project.

This report presents both suggestions and recommendations. The suggestions are presented so that the owner and the owner's representatives may compare the cost to the potential risk or benefit for the suggested procedures.

This report contains suggestions and recommendations which are intended to work in concert with recommendations provided by the other design team members to provide somewhat predictable foundation performance. If any of the recommendations are not included in the design and construction of the project it may result in unpredictable foundation performance or performance different than anticipated. We recommend that we be requested to provide geotechnical engineering observation and materials testing during the construction phase of the project as discussed in this report. The purpose for on site observation and testing by us during construction is to help provide continuity of service from the planning of the project through the construction of the project. This service will also allow us to revise our recommendations if conditions occur or are discovered during construction that were not evidenced during the initial study. We suggest that the owner and the contractor make provisions in their construction budget and construction schedule to accommodate unexpected subsurface conditions.

We represent that our services were performed within the limits prescribed by you and with the usual thoroughness and competence of the current accepted practice of the geotechnical engineering profession in the area. No warranty or representation either expressed or implied is included or intended in this report or our contract. We are available to discuss our findings with you. If you have any questions please contact us. The supporting data for this report is included in the accompanying figures and appendices.

This report is a product of Lambert and Associates. Excerpts from this report used in other documents may not convey the intent or proper concepts when taken out of context, or they may be misinterpreted or used incorrectly. Reproduction, in part or whole, of this document without prior written consent of Lambert and Associates is prohibited.

This report and information presented can be used only for this site, for this proposed development, and only for the client for whom our work was performed. Any other circumstances are not appropriate applications of this information. Other development plans will require project specific review by us.

Please call when further consultation or observations and tests are required.

If you have any questions concerning this report or if we may be of further assistance, please contact us.

Respectfully submitted:

COLORADO REGISTERED PROFESSIONAL ENGINEER 37151

LAMBERT AND ASSOCIATES

Daniel R. Lambert, P.E.

APPENDIX A

The field study was performed on March 30, 2026. The field study consisted of logging and sampling the soils encountered in eight (8) test borings. The approximate locations of the test borings are shown on Figure 2. The log of the soils encountered in the test borings are presented on Figures A2 through A9.

The test borings were logged by Lambert and Associates and samples of significant soil types were obtained. The samples were obtained from the test borings using a Modified California Barrel sampler and bulk disturbed samples were obtained. Penetration blow counts were determined using a 140 pound hammer free falling 30 inches. The blow counts are presented on the logs of the test borings such as 7/6 where 7 blows with the hammer were required to drive the sampler 6 inches.

The engineering field description and major soil classification are based on our interpretation of the materials encountered and are prepared according to the Unified Soil Classification System, ASTM D2488. The description and classification which appear on the test boring log is intended to be that which most accurately describes a given interval of the test boring (frequently an interval of several feet). Occasionally discrepancies occur in the Unified Soil Classification System nomenclature between an interval of the soil log and a particular sample in the interval. For example, an interval on the test boring log may be identified as a silty sand (SM) while one sample taken within the interval may have individually been identified as a sandy silt (ML). This discrepancy is frequently allowed to remain to emphasize the occurrence of local textural variations in the interval.

The stratification lines presented on the logs are intended to present our interpretation of the subsurface conditions encountered in the test boring. The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

LOG OF TEST BORING

Date Drilled: March 30, 2026 **Field Engineer:** DRL **Boring Number:** 1

Location: See test boring location diagram **Elevation:**

Diameter: 4 inches **Total Depth:** 15 feet **Depth to Water at Time of Drilling:** None Encountered

Symbol	Depth	Sample Type	Sample N	Soil Description	Laboratory Test Results
	0			Clay, silty, stiff, moist, brown	
	5	C	7/6" 10/6"		Swell/Consolidation Test: DD: 101 pcf MC: 9.9%
	10	C	20/6" 31/6"	Sand, gravel, clayey, dense, moist, brown, gray	
				Formational Shale Material	
	15			Bottom of Test Boring at 15 feet	
	20				
	25				

Project Name: Fire Station #4 - Montrose **Project Number:** M26010GE **Figure:** A2

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

LOG OF TEST BORING

Date Drilled: March 30, 2026 **Field Engineer:** DRL **Boring Number:** 2

Location: See test boring location diagram **Elevation:**

Diameter: 4 inches **Total Depth:** 20 feet **Depth to Water at Time of Drilling:** None Encountered

Depth	Sample Type	N	Soil Description	Laboratory Test Results
0			Clay, silty, stiff, moist, brown	
5	C	6/6" 7/6"		Direct Shear Test: DD: 127 pcf MC: 11.9%
			* Intermittent Gravels Encountered	
10	C	23/6" 28/6"	Sand, gravel, clayey, dense, moist, brown, gray	Swell/Consolidation Test: DD: 112 pcf MC: 5.9% Swell/Consolidation Test: DD: 119 pcf MC: 4.9%
15			Formational Shale Material	
20			Bottom of Test Boring at 20 feet	
25				

Project Name: Fire Station #4 - Montrose **Project Number:** M26010GE **Figure:** A3

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

ADDENDUM 02, ATTACHMENT 02

LOG OF TEST BORING

Date Drilled: March 30, 2026 **Field Engineer:** DRL **Boring Number:** 3

Location: See test boring location diagram **Elevation:**

Diameter: 4 inches **Total Depth:** 15 feet **Depth to Water at Time of Drilling:** None Encountered

Symbol	Depth	Sample Type	Sample N	Soil Description	Laboratory Test Results
	0			Clay, silty, stiff, moist, brown	
	5	C	6/6" 7/6"		Swell/Consolidation Test: DD: 91 pcf MC: 8.4%
				* Intermittent Gravels Encountered	
	10	C	12/6" 18/6"	Sand, gravel, clayey, dense, moist, brown, gray	
				Formational Shale Material	
	15			Bottom of Test Boring at 15 feet	
	20				
	25				

Project Name: Fire Station #4 - Montrose **Project Number:** M26010GE **Figure:** A4

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CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

LOG OF TEST BORING

Date Drilled: March 30, 2026 **Field Engineer:** DRL **Boring Number:** 4

Location: See test boring location diagram **Elevation:**

Diameter: 4 inches **Total Depth:** 15 feet **Depth to Water at Time of Drilling:** None Encountered

Symbol	Depth	Sample Type	Sample N	Soil Description	Laboratory Test Results
	0			Clay, silty, stiff, moist, brown	
	5	C	11/6" 12/6"		Swell/Consolidation Test: DD: 99 pcf MC: 6.9%
				* Intermittent Gravels Encountered	
	10	C	23/6" 26/6"	Formational Shale Material	Swell/Consolidation Test: DD: 112 pcf MC: 15.9%
	15			Bottom of Test Boring at 15 feet	
	20				
	25				

Project Name: Fire Station #4 - Montrose **Project Number:** M26010GE **Figure:** A5

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

ADDENDUM 02, ATTACHMENT 02

LOG OF TEST BORING

Date Drilled: March 30, 2026 **Field Engineer:** DRL **Boring Number:** 5

Location: See test boring location diagram **Elevation:**

Diameter: 4 inches **Total Depth:** 20 feet **Depth to Water at Time of Drilling:** 14-1/2 feet

Depth (ft)	Sample Type	N	Soil Description	Laboratory Test Results
0–14.5			Clay, silty, stiff, moist, brown	
5	C	10/6" 12/6"		Swell/Consolidation Test: DD: 99 pcf MC: 8.3%
10	C	9/6" 12/6"		
			* Intermittent Gravels Encountered	
14.5–16			Sand, gravel, clayey, dense, wet, brown, gray	
16–20			Formational Shale Material	
20			Bottom of Test Boring at 20 feet	

Project Name: Fire Station #4 - Montrose **Project Number:** M26010GE **Figure:** A6

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LOG OF TEST BORING

Date Drilled: March 30, 2026 **Field Engineer:** DRL **Boring Number:** 6

Location: See test boring location diagram **Elevation:**

Diameter: 4 inches **Total Depth:** 10 feet **Depth to Water at Time of Drilling:** None Encountered

Symbol	Depth	Sample Type	Sample N	Soil Description	Laboratory Test Results
	0–10	Bulk		Clay, silty, stiff, moist, brown	
	10			Bottom of Test Boring at 10 feet	

Depth scale: 0, 5, 10, 15, 20, 25

Project Name: Fire Station #4 - Montrose **Project Number:** M26010GE **Figure:** A7

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

LOG OF TEST BORING

Date Drilled: March 30, 2026 **Field Engineer:** DRL **Boring Number:** 7

Location: See test boring location diagram **Elevation:**

Diameter: 4 inches **Total Depth:** 10 feet **Depth to Water at Time of Drilling:** None Encountered

Symbol	Depth	Sample Type	Sample N	Soil Description	Laboratory Test Results
	0–8.5	Bulk (1–5)		Clay, silty, stiff, moist, brown	
	8.5–10			Sand, gravel, clayey, dense, wet, brown, gray	
	10			Bottom of Test Boring at 10 feet	

Project Name: Fire Station #4 - Montrose **Project Number:** M26010GE **Figure:** A8

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

ADDENDUM 02, ATTACHMENT 02

LOG OF TEST BORING

Date Drilled: March 30, 2026 **Field Engineer:** DRL **Boring Number:** 8

Location: See test boring location diagram **Elevation:**

Diameter: 4 inches **Total Depth:** 10 feet **Depth to Water at Time of Drilling:** None Encountered

Symbol	Depth	Sample Type	Sample N	Soil Description	Laboratory Test Results
	0–10	Bulk		Clay, silty, stiff, moist, brown	
	10			Bottom of Test Boring at 10 feet	

Depth scale: 0, 5, 10, 15, 20, 25

Project Name: Fire Station #4 - Montrose **Project Number:** M26010GE **Figure:** A9

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CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

ADDENDUM 02, ATTACHMENT 02

APPENDIX B

The laboratory study consisted of performing:

- Moisture content and dry density tests,
- Swell-consolidation tests, and
- Direct Shear Strength tests,
- California bearing ratio tests, and
- Moisture Content-dry density relationship tests.

It should be noted that samples obtained using a drive type sleeve sampler may experience some disturbance during the sampling operations. The test results obtained using these samples are used only as indicators of the in situ soil characteristics.

TESTING

Moisture Content and Dry Density

Moisture content and dry density were determined for each sample tested of the samples obtained. The moisture content was determined according to ASTM Test Method D2216 by obtaining the moisture sample from the drive sleeve. The dry density of the sample was determined by using the wet weight of the entire sample tested. The results of the moisture and dry density determinations are presented on the logs of borings, Figures A2 through A9.

Swell Tests

Loaded swell tests were performed on drive samples obtained during the field study. These tests are performed in general accordance with ASTM Test Method D2435 to the extent that the same equipment and sample dimensions used for consolidation testing are used for the determination of expansion. A sample is subjected to static surcharge, water is introduced to produce saturation, and volume change is measured as in ASTM Test Method D2435. Results are reported as percent change in sample height.

Consolidation Tests

One dimensional consolidation properties of drive samples were evaluated according to the provisions of ASTM Test Method D2435. Water was added in all cases during the test. Exclusive of special readings during consolidation rate tests, readings during an increment of load were taken regularly until the change in sample height was less than 0.001 inch over a two hour period. The results of the swell-consolidation load test are summarized on Figures B1 through B7, swell-consolidation tests.

It should be noted that the graphic presentation of consolidation data is a presentation of volume change with change in axial load. As a result, both expansion and consolidation can be illustrated.

Direct Shear Strength Tests

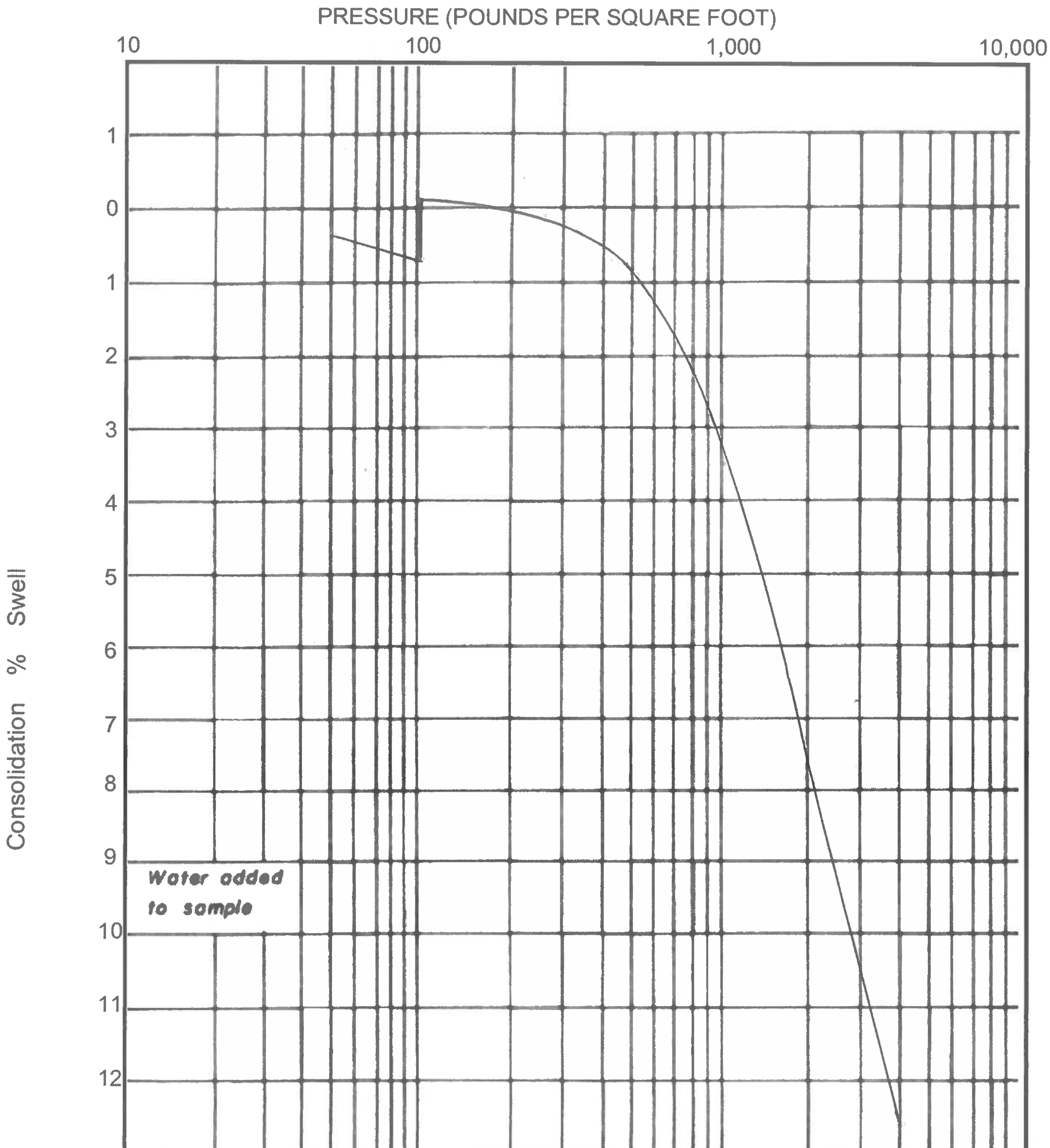
Direct shear strength properties of drive samples were evaluated in general accordance with testing procedures defined by ASTM Test Method D3080. The results of the direct shear strength test are summarized on Figure B8, direct shear test.

California Bearing Ratio Tests

California bearing ratio tests were conducted on select soil samples obtained during our field study. The California bearing ratio tests were conducted in accordance with ASTM Test Method D1883. The results of the California bearing ratio tests are presented on Figure B9.

Moisture Content-Dry Density Relationship Tests

Moisture content-dry density relationship tests were conducted on select soils subgrade samples obtained during our field study. The moisture-density relationship tests were conducted in accordance with ASTM Test Method D1557. The results of the moisture-density relationship tests are presented on Figure B10.



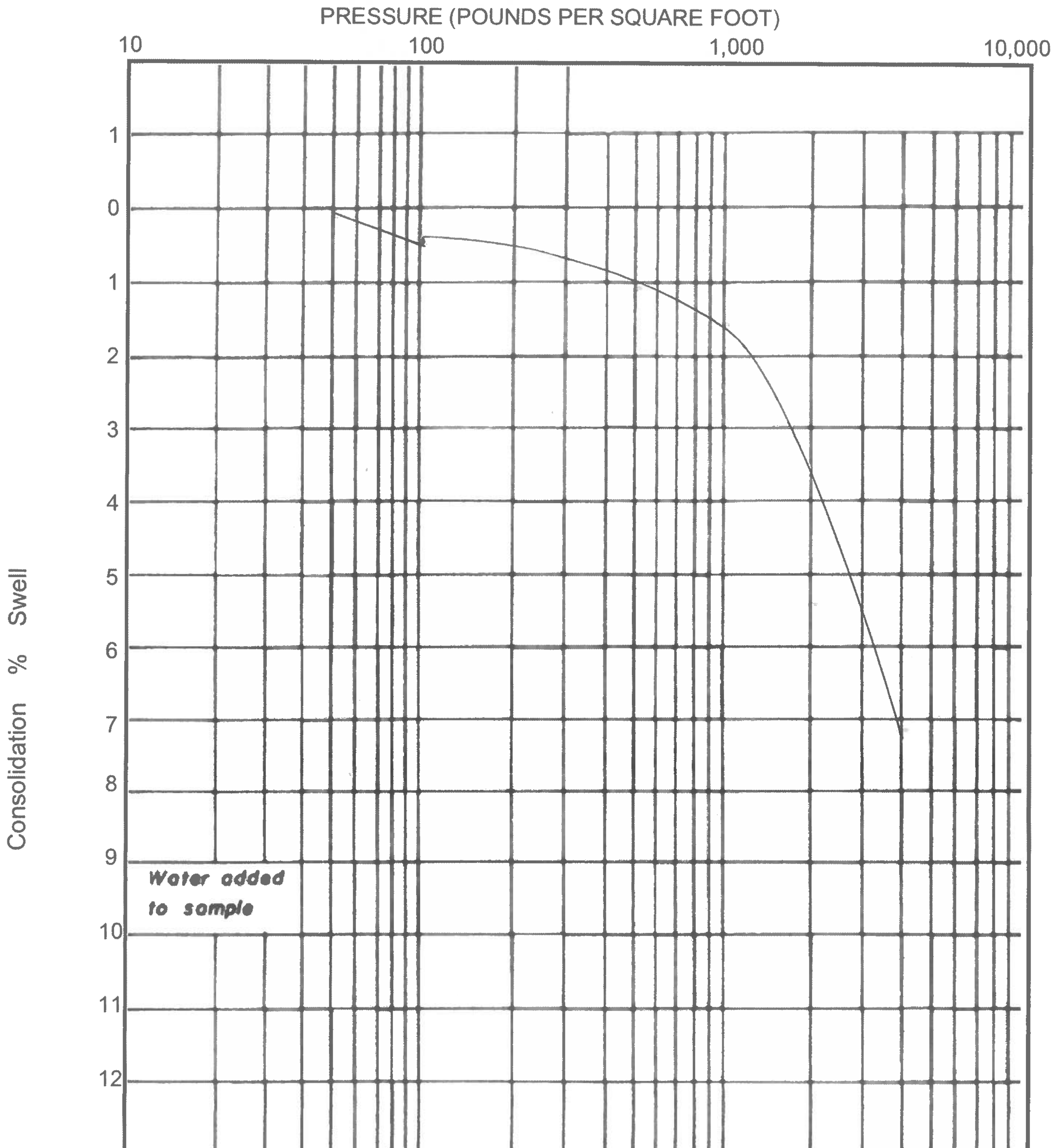
	SUMMARY OF TEST RESULTS				
Boring No. 1 Depth 4-5 ft	Moisture Content %	Dry Density PCF	Height in	Diameter In	Swell Pressure PSF
Initial	9.9	101	1.00	1.94	± 600
Final	22.6	115	0.873	1.94	
Soil Description	Clay, silty, brown				

SWELL-CONSOLIDATION TEST

Lambert and Associates

Project No.	M26010GE
Date:	May 4, 2026
Figure:	B1

ADDENDUM 02, ATTACHMENT 02



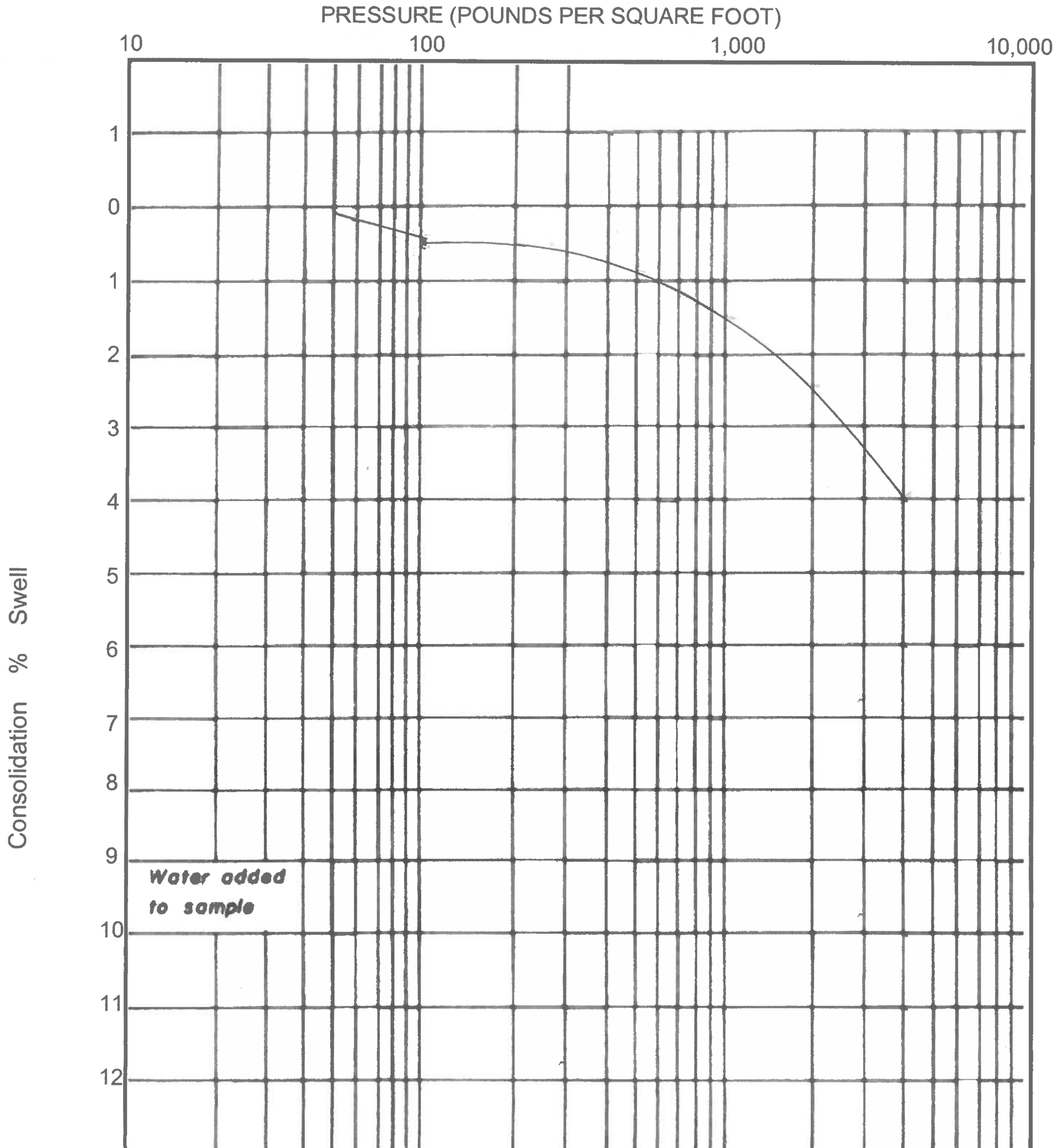
Boring No. 2 Depth 9-9.5 ft	SUMMARY OF TEST RESULTS Moisture Content %	Dry Density PCF	Height in	Diameter In	Swell Pressure PSF
Initial	5.9	112	1.00	1.94	± 200
Final	15.6	116	0.928	1.94	
Soil Description	Sand, gravel, clayey, brown				

SWELL-CONSOLIDATION TEST

Lambert and Associates

Project No.	M26010GE
Date:	May 4, 2026
Figure:	B2

ADDENDUM 02, ATTACHMENT 02

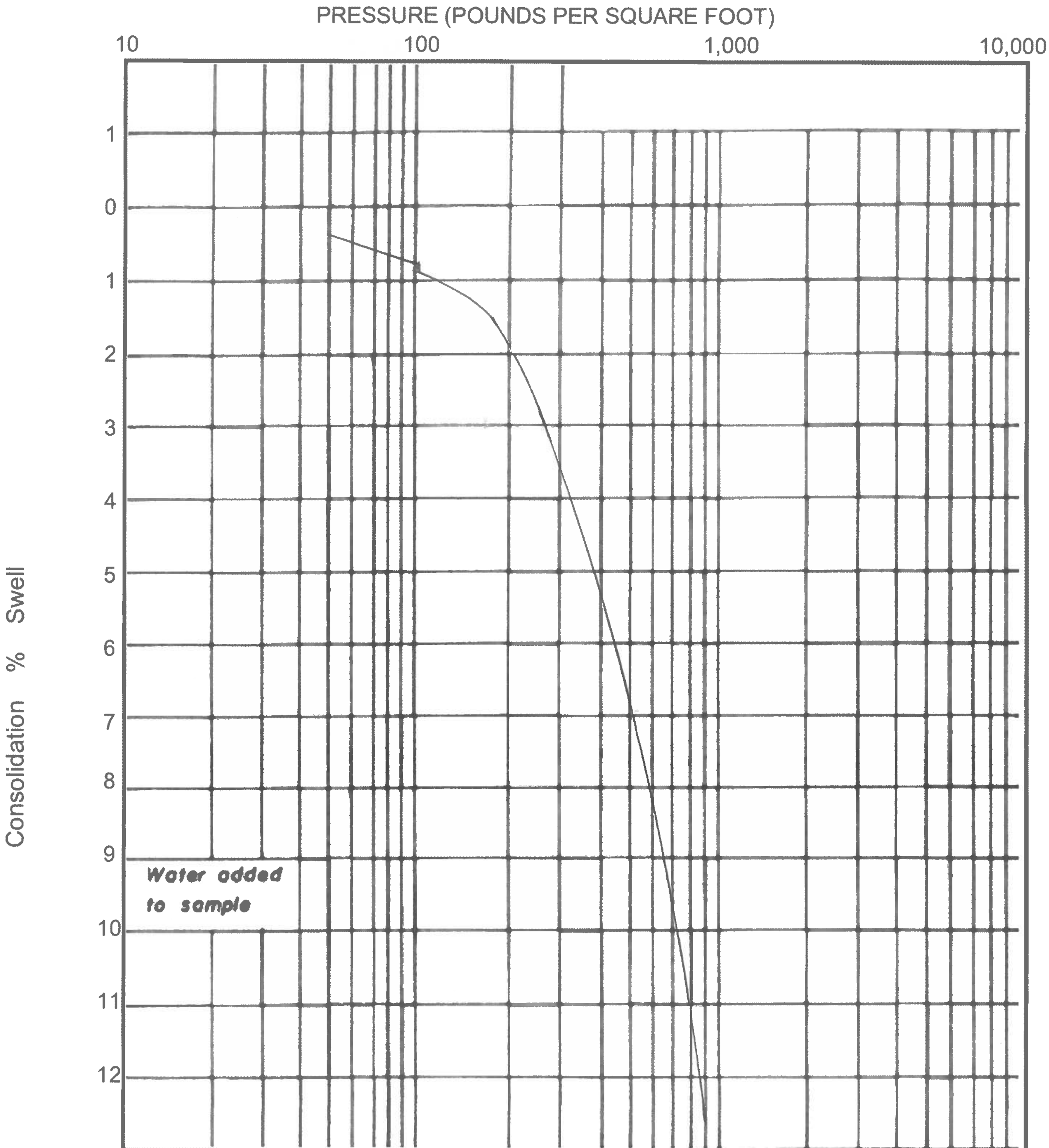


Boring No. 2 Depth 9.5-10 ft	SUMMARY OF TEST RESULTS				
	Moisture Content %	Dry Density PCF	Height in	Diameter In	Swell Pressure PSF
Initial	4.9	119	1.00	1.94	≤ 100
Final	15.1	113	0.960	1.94	
Soil Description	Sand, gravel, clayey, brown				

SWELL-CONSOLIDATION TEST

Lambert and Associates

Project No.	M26010GE
Date:	May 4, 2026
Figure:	B3



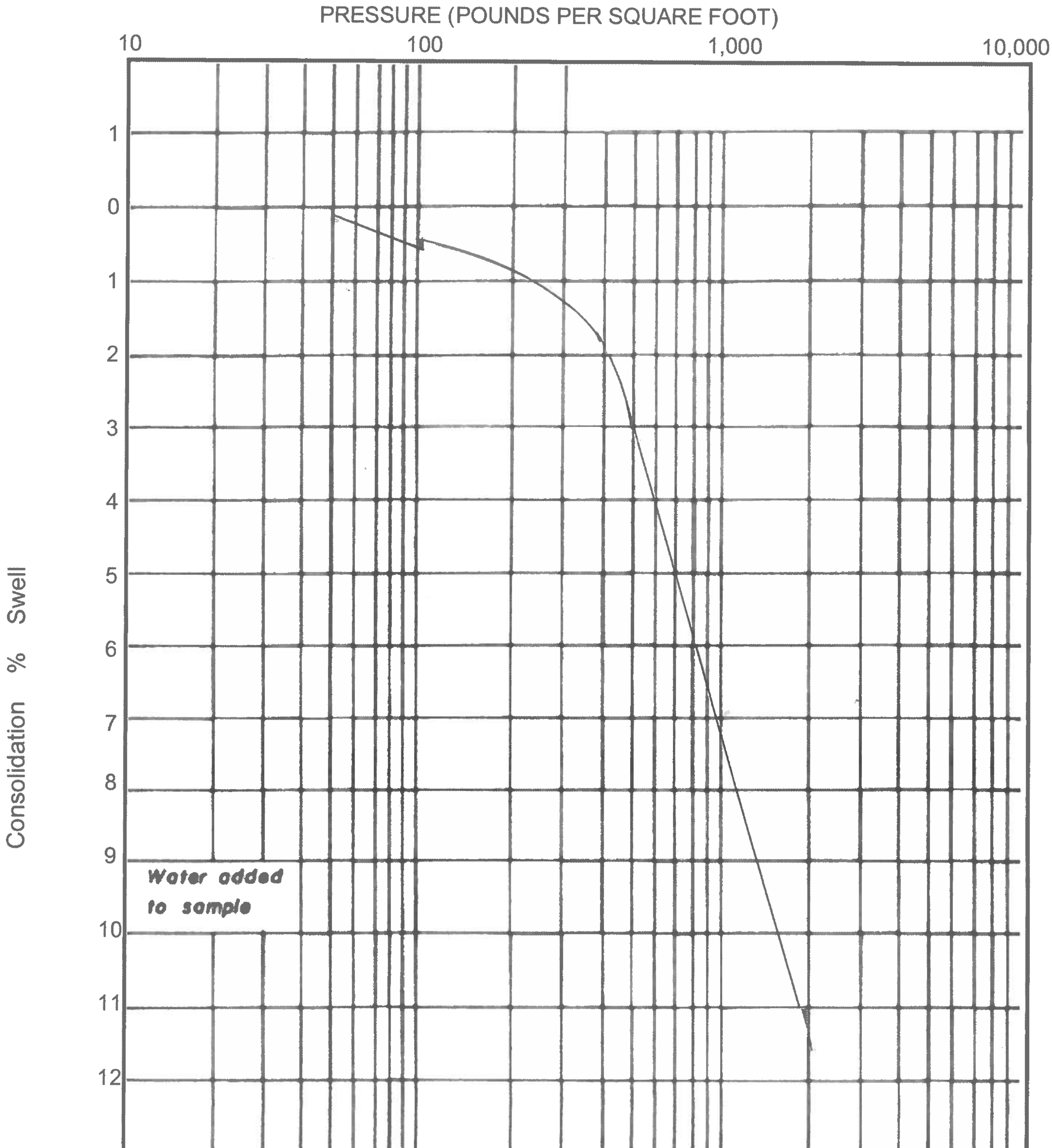
Boring No. 3 Depth 4-5 ft	SUMMARY OF TEST RESULTS				
	Moisture Content %	Dry Density PCF	Height in	Diameter In	Swell Pressure PSF
Initial	8.4	91	1.00	1.94	≤ 100
Final	21.7	120	0.751	1.94	
Soil Description	Clay, silty, sandy, brown				

SWELL-CONSOLIDATION TEST

Lambert and Associates

Project No.	M26010GE
Date:	May 4, 2026
Figure:	B4

ADDENDUM 02, ATTACHMENT 02



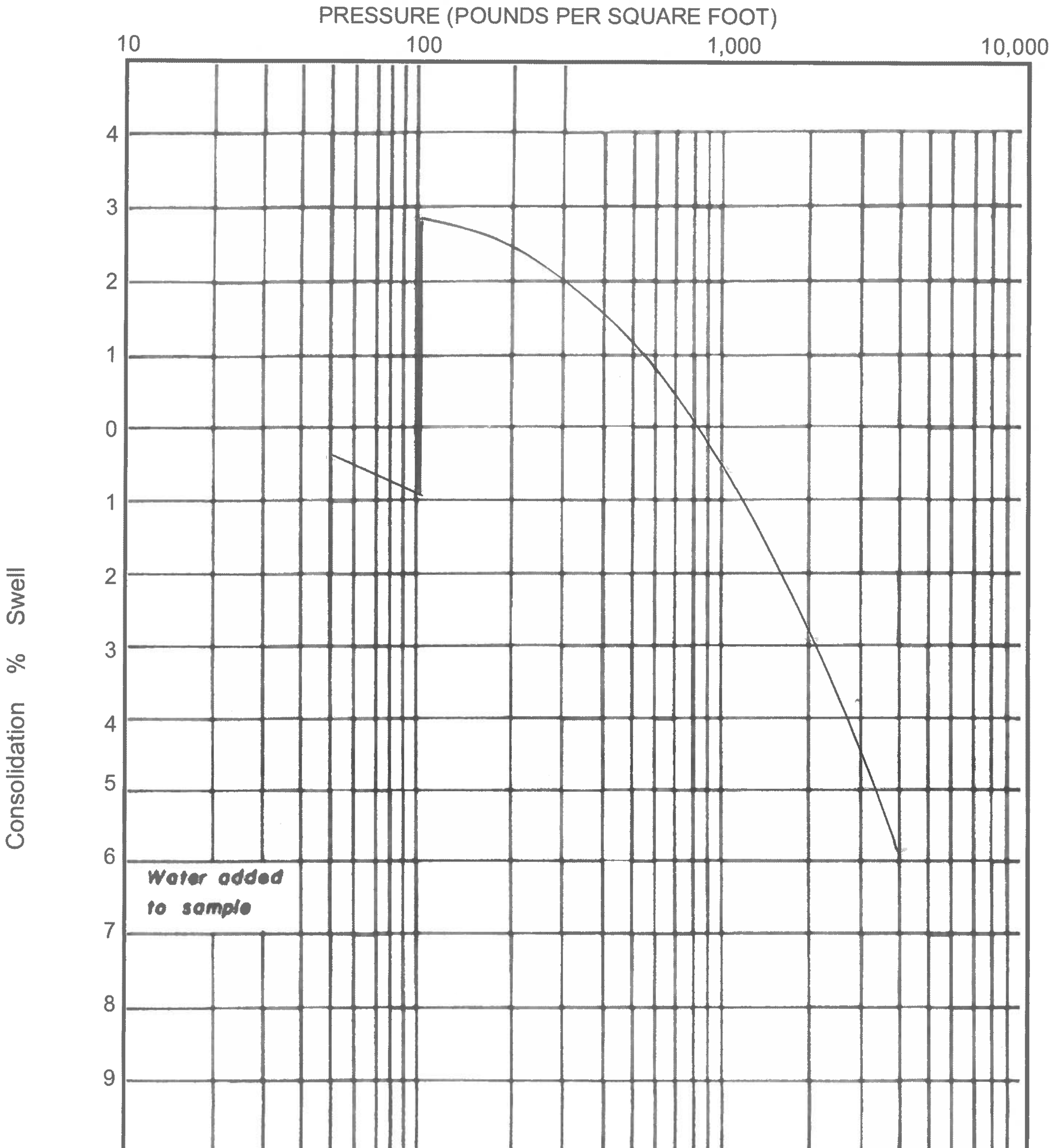
Boring No. 4 Depth 4-5 ft	SUMMARY OF TEST RESULTS				
	Moisture Content %	Dry Density PCF	Height in	Diameter In	Swell Pressure PSF
Initial	6.9	99	1.00	1.94	± 200
Final	21.8	117	0.841	1.94	
Soil Description	Clay, silty, sandy, brown				

SWELL-CONSOLIDATION TEST

Lambert and Associates

Project No.	M26010GE
Date:	May 4, 2026
Figure:	B5

ADDENDUM 02, ATTACHMENT 02



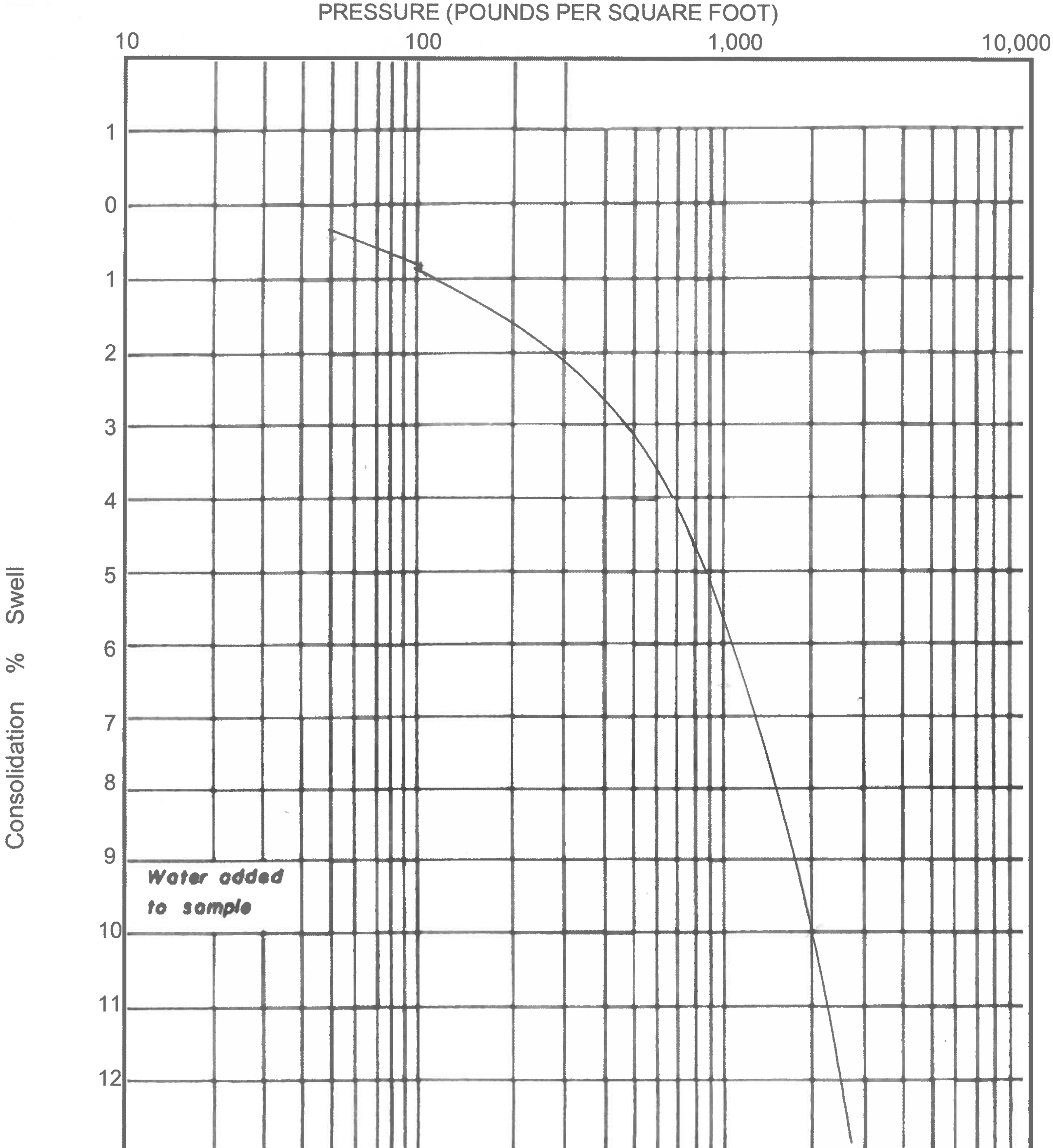
Boring No. 4 Depth 9-10 ft	SUMMARY OF TEST RESULTS				
	Moisture Content %	Dry Density PCF	Height in	Diameter In	Swell Pressure PSF
Initial	15.9	112	1.00	1.94	± 1,350
Final	21.3	120	0.942	1.94	
Soil Description	Formational Shale Material				

SWELL-CONSOLIDATION TEST

Lambert and Associates

Project No.	M26010GE
Date:	May 4, 2026
Figure:	B6

ADDENDUM 02, ATTACHMENT 02



SUMMARY OF TEST RESULTS					
Boring No. 5 Depth 4-5 ft	Moisture Content %	Dry Density PCF	Height in	Diameter In	Swell Pressure PSF
Initial	8.3	99	1.00	1.94	≤ 100
Final	20.8	115	0.860	1.94	
Soil Description	Clay, silty, sandy, brown				

SWELL-CONSOLIDATION TEST

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Project No.	M26010GE
Date:	May 4, 2026
Figure:	B7

ADDENDUM 02, ATTACHMENT 02

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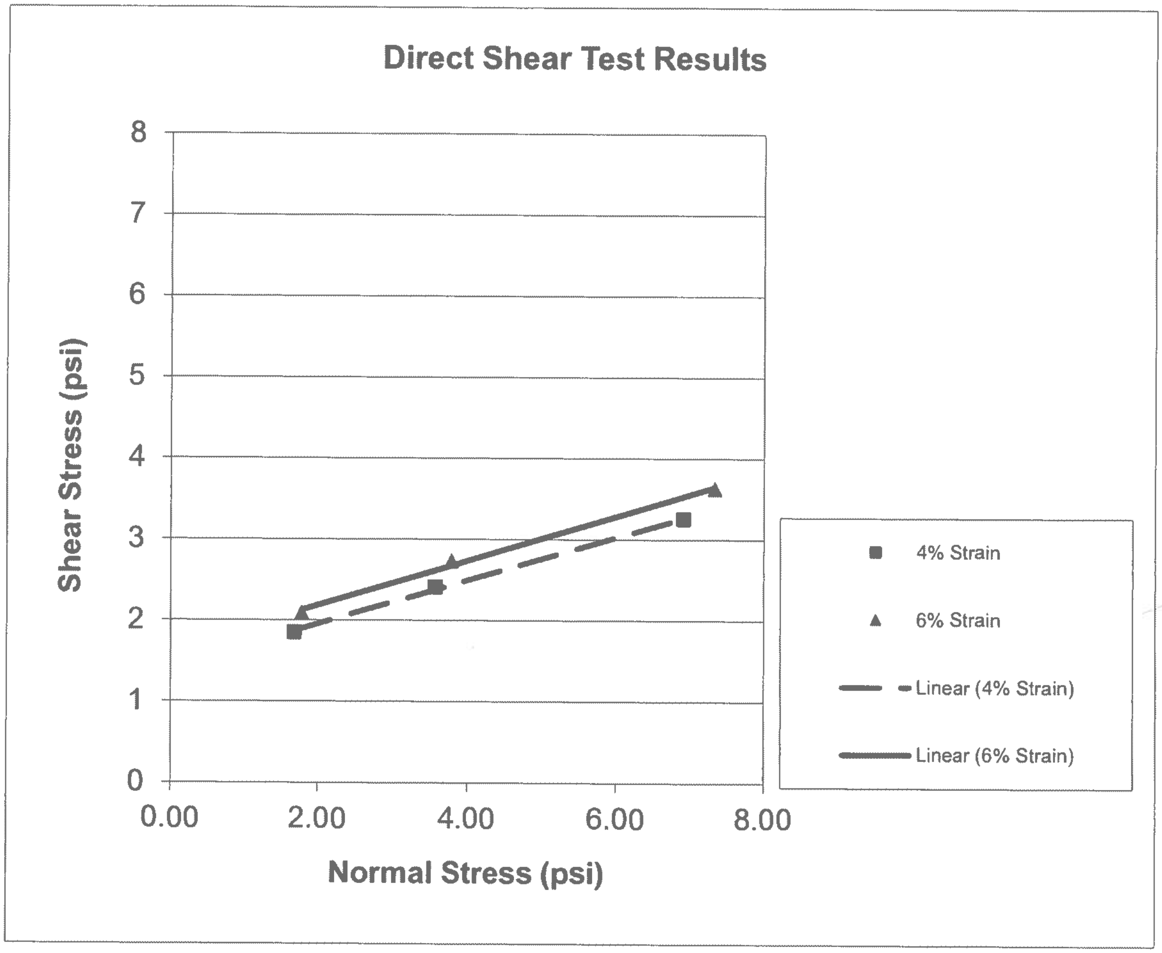
CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

Project:	Fire Station No. 4	**Project Number:**	M26010GE	**Date Sampled:**	3/30/2026
Location:	Montrose, CO	**Sample Source:**	TB 2 @ 4-5 ft	**Lab Sample #:**	5167
Sample Description:	Clay, silty, sandy, brown	**Date Tested:**	4/6/2026	**Tested By:**	AC

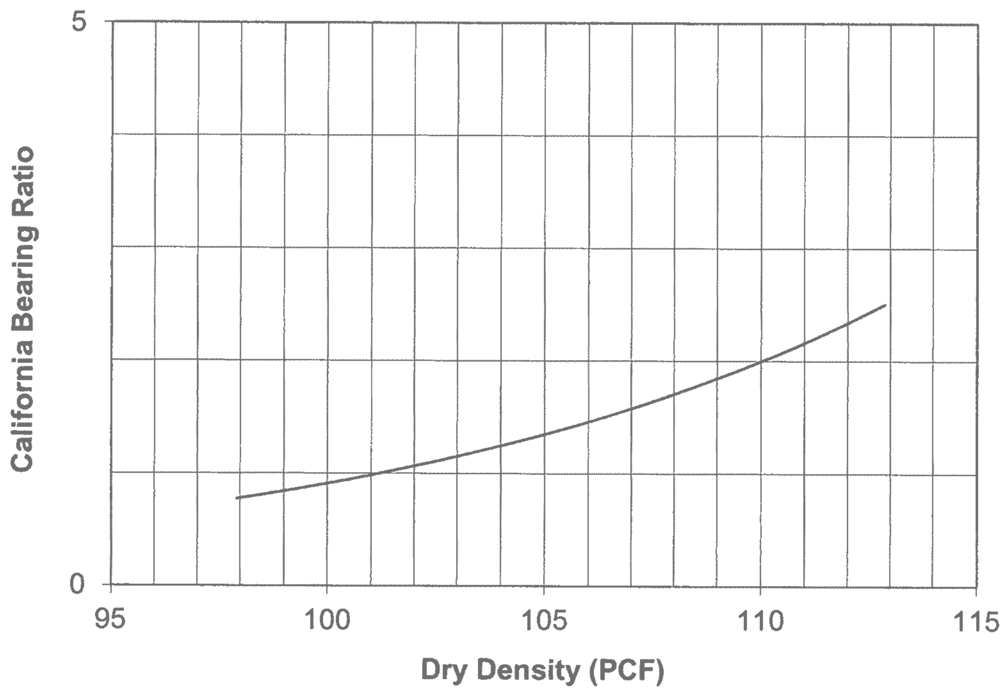


% Strain	Cohesion (psf)	Friction Angle (deg)
4	205	15
6	236	15

Project No.:	M26010GE
Date:	May 4, 2026
Figure:	B8

CALIFORNIA BEARING RATIO TEST RESULTS
ASTM D1883

Project Name :	Fire Station #4	Date:	May 4, 2026
Project Number :	M26010GE	Lab Number :	5167
Proctor Method Used :	D1557	Sample Location :	TB 6, 7, 8 @ 1 - 5 ft
Sample Description :	Clay, sandy, brown	Surcharge Weight :	15



TEST DATA

PRE-SOAK		AFTER 96 HOUR SOAK			
Dry Density (pcf)	Moisture Content (%)	Dry Density (pcf)	Moisture Content Top One (1) Inch (%)	Swell (%)	CBR (%)
97.9	15.7	95.2	23.0	2.8	0.9
107.4	15.7	104.1	19.9	3.2	0.9
112.9	15.7	110.7	17.9	2.0	3.5

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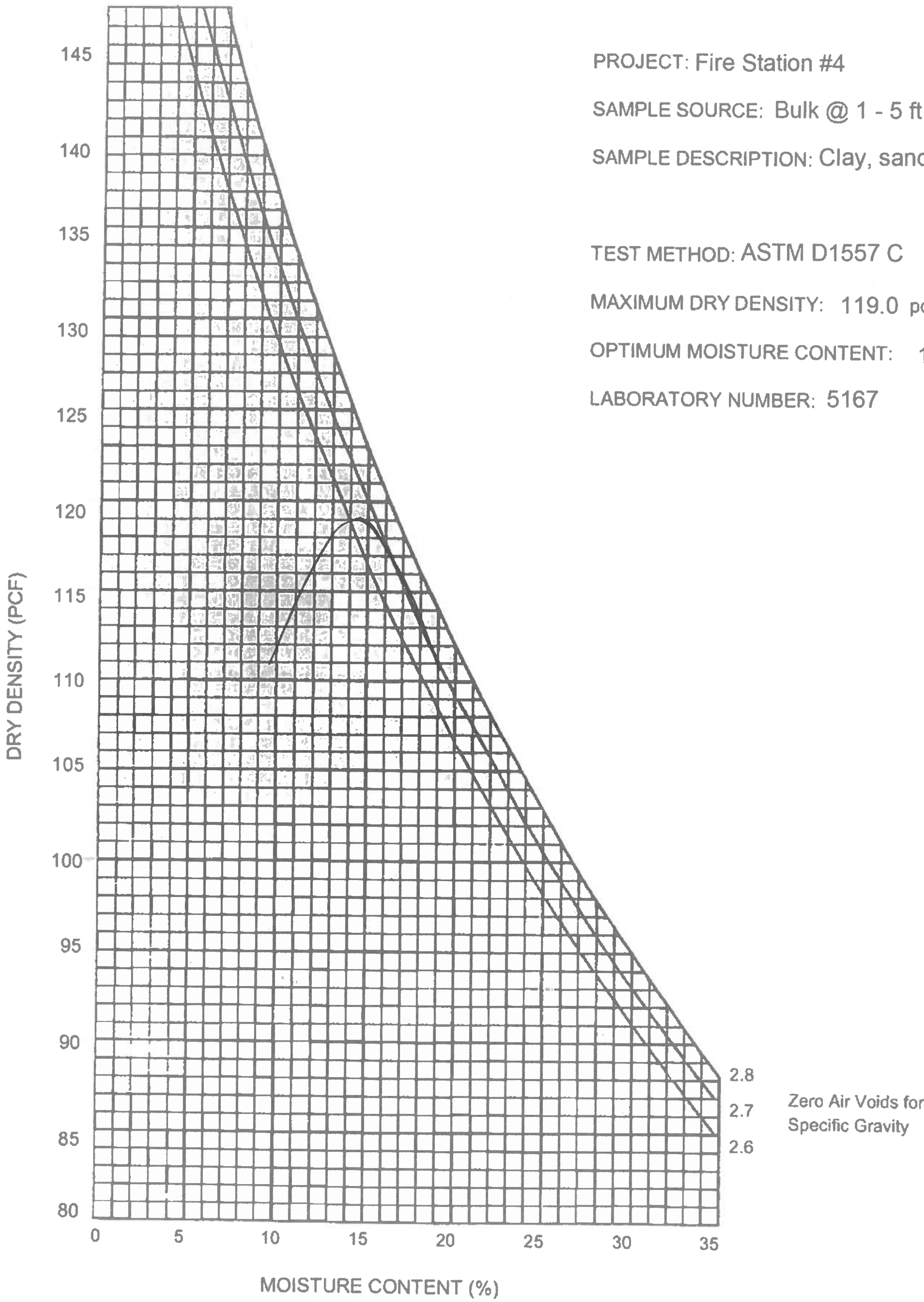
Project No. :	M26010GE
Date:	May 4, 2026
Figure :	B9

ADDENDUM 02, ATTACHMENT 02



PROJECT: Fire Station #4

SAMPLE SOURCE: Bulk @ 1 - 5 ft

SAMPLE DESCRIPTION: Clay, sandy, brown

TEST METHOD: ASTM D1557 C

MAXIMUM DRY DENSITY: 119.0 pcf

OPTIMUM MOISTURE CONTENT: 14.5 %

LABORATORY NUMBER: 5167

Lambert and Associates

Project No.	M26010GE
Date:	May 4, 2026
Figure:	B10

ADDENDUM 02, ATTACHMENT 02

APPENDIX C

GEOLOGY DISCUSSION
SOUTHWEST COLORADO GEOLOGY

Southwest Colorado exhibits many geologic features formed by a multitude of geologic processes. Regional inundation, uplift, volcanism and glaciation are responsible for some of the complex geology of the region. Many theories and speculations concerning the mode of occurrence of the regions's geology have been presented over the years. This cursory discussion of the geology of southwest Colorado presents some theories accepted by the geologic community, but is only intended to introduce the basic concepts and restraints that arise due to geologic activity.

Prior to the formation of the Rocky Mountains southwest Colorado was a primarily a flat lying region with little topographic expression. The North American continent was experiencing many episodes of deposition. The Transcontinental Sea was transgressing and regressing across the continent, these transgressions and regressions are the cause for such diverse rock types. The stratigraphic column in southwestern Colorado expresses rock types from variable depositional environments. Limestones are formed in deeper water, sandstones are formed in beach and tidal flat environments, while arkosic sandstone and conglomerates are formed in alluvial plains and fans. Particle size and mineralogic content in rock units are related to the depositional environment. A sandstone or conglomerate would not be likely to form in a deep sea environment because there would not be enough energy to carry such large particles a great distance from the source lands. As one observes the stratigraphic column of southwest Colorado a siltstone may be overlain by a sandstone which is in turn overlain by a siltstone. This represents a regressional then transgressional sequence. Many such sequences or combinations of other rock units are exhibited throughout southwest Colorado.

The final regression of the sea may have been caused by orogenic activity and uplift. This uplift was not confined to Colorado, it was a regional uplift that occurred in many stages. The uplift is what caused the formation of the ancestral rockies. The Larimide Orogenic episode is responsible for the formation of the San Juan dome. (Note: The San Juan dome theory is not accepted by the entire geologic community. It is used here for descriptive purposes). The San Juan dome was essentially an upwarp of the stratigraphy formed by sedimentation during the Transcontinental Sea. An actual dome probably never existed due to erosion during the uplift. The idea being that a dome of sediments and rock units would have existed had erosion and diastrophism not taken place. The orientation of bedding planes forms a radial pattern around the San Juan region which seems to vindicate this theory.

The stresses need to "upwarp" this large area were obviously tremendous. Locally occurring stresses may not be sufficient to move this quantity of material, global tectonics, directly or indirectly, may have been involved. Compression of the entire North American plate could have occurred. The magnitude of the stresses and the deep seated origin of these stresses also have caused extensive volcanism. Colorado has many large remnants of Calderas that were active during the orogenic activity. The Silverton and Lake City Calderas are the largest in the San Juan region. Activity in the Silverton Caldera has been estimated (radiometrically) to have occurred 22 million years ago. Calderas of this magnitude are believed to have formed by the collapse of epierogenic magma chambers. Volcanic and metamorphic rock bodies are common in the San Juan region, many of these units are related to the orogenic activity in the region.

Faults associated with local orogenic activity are another common geologic feature found in southwestern Colorado. As stated previously, extreme stresses were probably associated with the formation of the San Juan Mountains and may be responsible for deep-seated volcanic and metamorphic processes. These stresses had to be released, the geologic mode for stress release is faulting. Diastrophic activity in the area today is quite low, the lack of seismic activity indicates that stresses are not currently being released. An explanation for the loss of stresses is through faulting.

The last episode of regional geologic activity in the area was glaciation. The most recent period of glacial activity ended approximately 10,000 years ago. Glacial activity is responsible for much of the topographic expression in the area. "U-Shaped" valleys, moraine deposits, tarns, (glacial formed lakes), and rock glaciers are the most prominent features which are found in southwestern Colorado as a result of glacial activity. The valley configurations are a result of the erosional activity of the glaciers. Moraine deposits developed during the glacial activity. Rock glaciers are moving masses of rock which are thought to have an ice core which may be the last remnant of glacial ice. As the surbsurface ice core moves and melts, the overlying mass of rock also moves.

APPENDIX D

GENERAL GEOTECHNICAL ENGINEERING CONSIDERATIONS

D1.0 INTRODUCTION

Appendix D presents general geotechnical engineering considerations for design and construction of structures which will be in contact with soils. The discussion presented in this appendix are referred to in the text of the report and are intended as tutorial and supplemental information to the appropriate sections of the text of the report.

D2.0 FOUNDATION RECOMMENDATIONS

Two criteria for any foundation which must be satisfied for satisfactory foundation performance are:

- contact stresses must be low enough to preclude shear failure of the foundation soils which would result in lateral movement of the soils from beneath the foundation, and
- settlement or heave of the foundation must be within amounts tolerable to the superstructure.

The soils encountered during our field study have varying engineering characteristics that may influence the design and construction considerations of the foundations. The characteristics include swell potential, settlement potential, bearing capacity and the bearing conditions of the soils supporting the foundations. The general discussion below is intended to increase the readers familiarity with characteristics that can influence any structure.

D2.1 Swell Potential

Some of the materials encountered during our field study at the anticipated foundation depth may have swell potential. Swell potential is the tendency of the soil to increase in volume when it becomes wetted. The volume change occurs as moisture is absorbed into the soil and water molecules become attached to or adsorbed by the individual clay platlets. Associated with the process of volume change is swell pressure. The swell pressure is the force the soil applies on its surroundings when moisture is absorbed into the soil. Foundation design considerations concerning swelling soils include structure tolerance to movement and dead load pressures to help restrict uplift. The structure's tolerance to movement should be addressed by the structural engineer and is dependent upon many facets of the design including the overall structural concept and the building material. The uplift forces or pressure due to wetted clay soils can be addressed by designing the foundations with a minimum dead load and/or placing the foundations on a blanket of compacted structural fill. The compacted structural fill blanket will increase the dead load on the swelling foundations soils and will increase the separation of the foundation from the swelling soils. Suggestions and recommendations for design dead load and compacted structural fill blanket are presented below. Compacted structural fill recommendations are presented under COMPACTED STRUCTURAL FILL below.

D2.2 Settlement Potential

Settlement potential of a soil is the tendency for the soil to experience volume change when subjected to a load. Settlement is characterized by downward movement of all or a portion of the supported structure as the soil particles move closer together resulting in decreased soil volume. Settlement potential is a function of;

- foundation loads,
- depth of footing embedment,
- the width of the footing, and
- the settlement potential or compressibility of the influenced soil.

Foundation design considerations concerning settlement potential include the amount of movement tolerable to the structure and the design and construction concepts to help reduce the potential movement. The settlement potential of the foundation can be reduced by reducing foundation pressures and/or by placing the foundations on a blanket of compacted structural fill. The anticipated post construction settlement potential and suggested compacted fill thickness recommendations are based on site specific soil conditions and are presented in the text of the report.

D2.3 Soil Support Characteristics

The soil bearing capacity is a function of;

- the engineering properties of the soil material supporting the foundations,
- the foundation width,
- the depth of embedment of the bottom of the foundation below the
- lowest adjacent grade,
- the influence of the ground water, and
- the amount of settlement tolerable to the structure.

Soil bearing capacity and associated minimum depth of embedment are presented in the text of the report.

The foundation for the structure should be placed on relatively uniform bearing conditions. Varying support characteristics of the soils supporting the foundation may result in nonuniform or differential performance of the foundation. Soils encountered at foundation depths may contain cobbles and boulders. The cobbles and boulders encountered at foundation depths may apply point loads on the foundation resulting in nonuniform bearing conditions. The surface of the formational material may undulate throughout the building site. If this is the case it may result in a portion of the foundation for the structure being placed on the formational material and a portion of the foundation being placed on the overlying soils. Varying support material will result in nonuniform bearing conditions. The influence of nonuniform bearing conditions may be reduced by placing the foundation members on a blanket of compacted structural fill. Suggestions and recommendations for constructing compacted structural fill are presented under COMPACTED STRUCTURAL FILL below and in the text of the report.

D3.0 COMPACTED STRUCTURAL FILL

Compacted structural fill is typically a material which is constructed for direct support of structures or structural components.

There are several material characteristics which should be examined before choosing a material for potential use as compacted structural fill. These characteristics include;

- the size of the larger particles,
- the engineering characteristics of the fine grained portion of material matrix,
- the moisture content that the material will need to be for compaction with respect to the existing initial moisture content,
- the organic content of the material, and
- the items that influence the cost to use the material.

Compacted fill should be a non-expansive material with the maximum aggregate size less than about two (2) inches and less than about twenty five (25) percent coarser than three quarter (3/4) inch size.

The reason for the maximum size is that larger sizes may have too great an influence on the compaction characteristics of the material and may also impose point loads on the footings or floor slabs that are in contact with the material. Frequently pit-run material or crushed aggregate material is used for structural fill material. Pit-run material may be satisfactory, however crushed aggregate material with angular grains is preferable. Angular particles tend to interlock with each other better than rounded particles.

The fine grained portion of the fill material will have a significant influence on the performance of the fill. Material which has a fine grained matrix composed of silt and/or clay which exhibits expansive characteristics should be avoided for use as structural fill. The moisture content of the material should be monitored during construction and maintained near optimum moisture content for compaction of the material.

Soil with an appreciable organic content may not perform adequately for use as structural fill material due to the compressibility of the material and ultimately due to the decay of the organic portion of the material.

D4.0 RADON CONSIDERATIONS

Information presented in "Radon Reduction in New Construction, An Interim Guide: OPA-87-009 by the Environmental Protection Agency dated August 1987 indicates that currently there are no standard soil tests or specific standards for correlating the results of soil tests at a building site with subsequent indoor radon levels. Actual indoor levels can be affected by construction techniques and may vary greatly from soil radon test results. Therefore it is recommended that radon tests be conducted in the structure after construction is complete to verify the actual radon levels in the home.

We suggest that you consider incorporating construction techniques into the development to reduce radon levels in the residential structures and provide for retrofitting equipment for radon gas removal if it becomes necessary.

Measures to reduce radon levels in structures include vented crawl spaces with vapor barrier at the surface of the crawl space to restrict radon gas flow into the structure or a vented gravel layer with a vapor barrier beneath a concrete slab-on-grade floor to allow venting of radon gas collected beneath the floor and to restrict radon gas flow through the slab-on-grade floor into the structure. These concepts are shown on Figure D1.

If you have any questions or would like more information about radon, please contact us or the State Health Department at 303-692-3030.

New Fire Station #4 **Sunnyside Road / 6720 Road Intersection** **Montrose, CO 81401**	**Date: May 12, 2026** Time: 11:00am Project No.: 26004 Location: Northeast corner of Sunnyside Road / 6720 Road Intersection	BG+co. CLIENT FOCUSED DESIGN SERVICE BLYTHE GROUP + co.

MEETING ATTENDANCE

Meeting called by: BG+co.	Type of meeting: Non-Mandatory Site Visit

NAME	COMPANY	PHONE	EMAIL
Will Kiser	Kiser Construction, Inc.	970-314-1008	wkiser@kiserconstructioninc.com
Mark Hendrickson	Shaw Construction	303-653-7434	markhendrickson@shawconstruction.net
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