Appendix 7: Geotechnical Investigation

GEOTECHNICAL EVALUATION
THE HUB FACILITY
NORTHWEST CORNER OF WCR 6 AND WCR 7
WELD COUNTY, COLORADO

PREPARED FOR:

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October 4, 2013
Project No. 500707001

Ninyo & Moore
Geotechnical and Environmental Sciences Consultants

October 4, 2013
Project No. 500707001

Mr. Noah Nemmers, PE
Baseline Engineering
700 12th Street, Suite 220
Golden, Colorado 80401

Subject: Geotechnical Evaluation
The Hub Facility
Northwest Corner of WCR 6 and WCR 7
Weld County, Colorado

Dear Mr. Nemmers:

In accordance with our proposal dated August 21st, 2013 and your authorization Ninyo & Moore has performed a geotechnical evaluation for the above referenced site. The attached report presents our methodology, findings, and conclusions regarding the geotechnical conditions at the project site and provides geotechnical engineering recommendations for the proposed improvements.

We appreciate the opportunity to be of service to you on this project.

Respectfully submitted,
NINYO & MOORE

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TABLE OF CONTENTS

Page

Tables

Figures

Appendices

1. INTRODUCTION

In accordance with your request and authorization, we have performed a geotechnical evaluation for the proposed Hub Facility project located near the northwest corner of WCR 6 and WCR 7 in Weld County, Colorado. The approximate location of the site is depicted on Figure 1.

The purpose of our study was to evaluate the subsurface conditions and to provide design and construction recommendations regarding geotechnical aspects of the proposed project. This report presents the findings of our subsurface exploration program, results of our laboratory testing, conclusions regarding the subsurface conditions at the site, and geotechnical recommendations for design and construction of this project.

2. SCOPE OF SERVICES

The scope of our services for the project generally included:

- Review of readily available aerial photographs and published geologic literature, including maps and reports pertaining to the project site and vicinity.
- Notification to the Utility Notification Center of Colorado of the boring locations prior to drilling.
- Drilling, logging, and sampling six exploratory borings to depths ranging from approximately 20 to 40 feet below ground surface (bgs). The boring logs are presented in Appendix A.
- Performing laboratory tests on selected samples obtained from the borings to evaluate in-situ moisture content and dry density, gradation, 200 wash, Atterberg limits, swell/consolidation, unconfined compressive strength, and soil corrosivity. The results of the laboratory testing are presented on the boring logs and in Appendices B and C.
- Compilation and analysis of the data obtained.
- Preparation of this report presenting our findings, conclusions, and recommendations regarding the design and construction of the project.

3. SITE DESCRIPTION

The project site consists of an approximately 55-acre undeveloped agricultural parcel located near the northwest corner of WCR 6 and WCR 7, in Weld County, Colorado. The site is bordered to the west by undeveloped agricultural pastureland, to the north by a gas well pad and agricultural land, to the east by WCR 7 and to the south by WCR 6. The approximate location of the site is depicted on Figure 1.

The site slopes gently to the east at grades of approximately 1 to 2 percent. Generally, the high point of the site is near the southwest corner at an elevation of approximately 5,235 feet above mean sea level (MSL). The lowest point of the site is near the eastern margin at an elevation of approximately 5,192 feet MSL. A shallow draw bisects the central portion of the site, extending in a roughly east-west direction.

Historical aerial photographs for selected years between 1993 and 2012 provided by Google Earth were reviewed for the site. Based on the historical aerial photograph review, the site has been vacant and likely used as agricultural land since 1993.

4. PROPOSED CONSTRUCTION

Based on conversations with Baseline Engineering (Baseline) personnel and our review of referenced project data, we understand the project will consist of the construction two primary liquid storage tank batteries containing six tanks each, other miscellaneous tanks, process facilities, paved staging and loading pads, a single-story water treatment building, and a single-story office building. Other site improvements include paved access roads, concrete flatwork and associated utilities. Anticipated grading is expected to consist of cuts and fills of approximately 10 feet or less, to establish pad grades and drainage.

5. FIELD EXPLORATION AND LABORATORY TESTING

On September 5 and 6, 2013, Ninyo & Moore conducted a subsurface exploration at the site in order to evaluate the existing subsurface conditions and to collect soil samples for laboratory testing. Our evaluation consisted of the drilling, logging, and sampling of six exploratory borings to evaluate the geologic and subsurface conditions beneath the site. The borings were advanced using a CME-75 track-mounted drill rig equipped with solid-flight augers. Relatively undisturbed soil samples were collected at selected intervals.

Descriptions of the soils encountered are presented on the boring logs in Appendix A. The general locations of the borings are shown on Figure 2.

The soil samples collected from our drilling activities were transported to the Ninyo & Moore laboratory for geotechnical laboratory analyses. The analyses included in-situ moisture content and dry density, No. 200 sieve analyses, gradation analyses, Atterberg limits, swell/consolidation potential, unconfined compressive strength, and corrosivity characteristics (including pH, minimum electrical resistivity, soluble sulfates, and chlorides). The results of the in-situ moisture content and dry density testing are presented on the boring logs in Appendix A. A description of each laboratory test method and the remainder of the test results are presented in Appendices B and C.

6. GEOLOGY AND SUBSURFACE CONDITIONS

The potential geologic hazards at the site are discussed in our previous Geologic Hazards Study, dated September 12, 2013. The geology and subsurface conditions at the site are described in the following sections.

6.1. Geologic Setting

The project site is located approximately 16 miles east of the southern Rocky Mountains, within the Colorado Piedmont section of the Great Plains Physiographic Province. The project site is located near the northern margins of a large north-south trending structural basin called the Denver Basin. The Denver Basin formed during the Laramide Orogeny that uplifted the Rocky Mountains during the late Cretaceous and early Tertiary (Trimble, 1980). Over time, the Denver Basin filled with alluvial sediments and wind-blown eolian deposits. The underlying bedrock is comprised of Tertiary to Cretaceous-age sedimentary units.

The surficial geology of the site is mapped by Colton (1978) as Holocene to late Pleistocene-age Eolian Deposits (wind-blown) including dune sand and loess deposits, which were deposited in the post-glacial period. While geologic maps indicate surficial soils at the site consist of loess deposits, we encountered alluvium underlying topsoil. The underlying formational bedrock unit is mapped as Upper Cretaceous-age Laramie Formation.

6.2. Subsurface Conditions

Our understanding of the subsurface conditions at the project site is based on our field exploration and laboratory testing, and our experience with the general geology of the area. The following sections provide a generalized description of the subsurface materials encountered. More detailed descriptions are presented on the boring logs in Appendix A.

6.2.1. Alluvium

Alluvium was encountered underlying topsoil and extended to depths between approximately 9 and 19.5 feet bgs. Alluvium generally consisted of sandy clay, clayey sand with few gravel, and clayey gravel with sand. Cobbles, and boulders, although not encountered in our borings, may be present within the alluvium. Based on the results of the subsurface exploration and laboratory testing, the alluvium encountered is stiff to dense, exhibits non- to high plasticity, low to moderate consolidation potential (up to 2 percent consolidation), and low swell potential. Selected samples had in-place moisture contents between 9.0 and 24.5 percent and dry densities between 88.8 and 116.5 pounds per cubic foot (pcf).

6.2.2. Laramie Formation Bedrock

Bedrock mapped by Colton (1978) as the Laramie Formation was encountered in each of our borings between approximately 9 and 19.5 feet bgs, and extended to the boring termination depths of approximately 20 to 40 feet bgs. The depth to bedrock is generally deeper in the shallow draw that bisects the site in a general east-west direction. The Laramie Formation was composed of varying shades of brown and gray, moderately to strongly indurated, weathered, claystone with interbeds of moderately to strongly cemented sandstone. Laramie Formation bedrock commonly contains layers and lenses of lignite (black in color). Although lignite layers/lenses were not encountered in the borings drilled, they may be encountered during deep foundation excavations. Based on the subsurface exploration and laboratory test results, the formational bedrock ranged from non-plastic to high plasticity. Selected samples had in-place moisture contents between 12.8 and 24.2 percent and dry densities between 94.7 and 118.3 (pcf). Based on our laboratory test results, tested samples exhibited high swell potential, with percent swells ranging up to 7.5 percent when inundated with water at estimated overburden pressures.

6.3. Groundwater

Groundwater was encountered in Borings B-2 and B-4 through B-6 at depths between 9.5 and 19 feet bgs. This relatively shallow groundwater condition is due to perching atop the relatively impermeable bedrock. The Laramie Fox Hills aquifer is the principal source of groundwater for irrigation, accounting for the majority of groundwater for high capacity wells. The static groundwater table associated with the aquifer is expected to be at a depth of 400 to 700 feet bgs. Recharge to the alluvial aquifer occurs by infiltration of applied irrigation water and precipitation. Seasonal fluctuations in groundwater levels and surface water flow may occur. These fluctuations may be due to variations in ground surface topography, subsurface geologic conditions, precipitation, irrigation, and other factors. Evaluation of factors associated with groundwater fluctuations was beyond the scope of this study.

7. CONCLUSIONS

Based on the results of our subsurface evaluation, laboratory testing, and data analysis, it is our opinion that the proposed construction is feasible from a geotechnical standpoint, provided the recommendations contained in this report are incorporated into design and construction of the proposed project. Geotechnical considerations include the following:

- The site is underlain by approximately 9 to 19.5 feet of alluvium. The laboratory test results indicate that the alluvium is heterogeneous and is non-plastic to high plasticity. Field exploration and laboratory testing indicate the alluvium has low to moderate consolidation potential and low swell potential.

- Formational materials, mapped as the Laramie Formation, were encountered in each of our borings at depths ranging between approximately 9 and 19.5 feet bgs. The depth to bedrock is generally deeper in the shallow draw that bisects the site in a general east-west direction. Based on the field exploration, laboratory test results, and background review, the formational materials have moderate to high swell potential.

- The on-site soils should generally be excavatable to the anticipated removal depths with heavy-duty earthmoving or excavating equipment in good operating condition. We do not anticipate excavations will extend into the Laramie Formation bedrock, with the exception of drilled pier excavations. In the event deeper excavations are planned, the formational bedrock encountered contains lenses of moderately to strongly cemented sandstone. The excavation rate will be very slow within the formational bedrock and the use of more aggressive excavation techniques, such as the use of single-shank rippers or rock breaking equipment, may be needed.

- Groundwater was encountered in Borings B-2 and B-4 through B-6 at approximately 9.5 to 19 bgs. In general, based on our understanding of the proposed site grading groundwater is not expected to be a constraint to construction of the project. Groundwater will be encountered in drilled pier excavations. Yielding subgrade conditions may be encountered in areas where site excavations are within 5 feet of the perched groundwater table. However, groundwater levels may rise due to seasonal variations, precipitation, irrigation, groundwater withdrawal or injection, and other factors.

- Site soils generated from on-site excavation activities consisting of alluvium that are free of deleterious materials, and do not contain particles larger than 3 inches in diameter, can generally be used as engineered fill. Laramie Formation bedrock, if encountered, should not be used as engineered fill.

- The depth to Laramie Formation bedrock on this project site is highly variable. Laramie Formation bedrock has high swell potential and the distance between the finished floor elevation of the building pads and the top of formational bedrock is a major design consideration for this project. If final grades differ by more than plus or minus two feet from the project plans and/or locations of the proposed improvements are modified by more than 5 feet Ninyo & Moore should be notified to re-evaluate the recommendations provided in this report.

- Based on the subsurface conditions encountered, the results of our laboratory testing, and our experience with similar projects, we recommend supporting the proposed Office Control Building on a drilled pier deep foundation system with a structural floor. The proposed Water Treatment Building may be supported on a spread footing shallow foundation system with a slab-on-grade floor founded on 4 or more feet of compacted engineered fill. The proposed liquid storage tanks may be supported on mat foundations founded on 4 or more feet of compacted engineered fill.

- The sulfate content of tested soils presents a negligible risk of sulfate attack to concrete.
- Corrosivity test results indicate the subgrade soils at the site are severely corrosive to ferrous metals. We recommend buried metal piping and components use corrosion resistant materials or a properly designed and installed cathodic protection system.

8. RECOMMENDATIONS

Based on our understanding of the project, the following sections present our geotechnical recommendations for the design and construction of the proposed project. These recommendations were prepared based on preliminary grading plans. If final site grades differ by more than plus or minus 2 feet from preliminary plans, Ninyo & Moore should be notified and given an opportunity to re-evaluate our recommendations prior to bidding the project for construction. It should be noted that we have not been provided structural drawings for the proposed structures, and our recommendations may need to be revised once final plans have been prepared.

8.1. Earthwork

The following sections provide our earthwork recommendations. Our recommendations are based on our evaluation of information obtained from six exploratory borings, our site observations, laboratory test results, and our experience with similar materials.

8.1.1. Excavations

Our evaluation of the excavation characteristics of the on-site materials is based on the results of the subsurface exploration, our site observations, and our experience with similar materials. The on-site surface and near surface soils (alluvium) may generally be excavated with heavy-duty earthmoving or excavation equipment in good operating condition.

Laramie Formation bedrock contains layers and lenses of moderately to strongly cemented sandstone bedrock. We do not anticipate excavations will extend into the Laramie Formation bedrock, with the exception of drilled pier excavations. In the event deeper excavations are planned, the excavation rate will be very slow within the formational bedrock and the use of more aggressive excavation techniques, such as single-shank rippers or rock breaking equipment, may be needed to achieve proposed grades. Laramie Formation bedrock was encountered at depths ranging between approximately 9 and 19.5 feet bgs.

When nearing excavation bottoms, equipment and procedures should be used that do not cause significant disturbance to the excavation bottoms. Excavators with buckets having large claws to loosen the soil should be avoided when excavating the last 6 to 12 inches of excavations. Such equipment may disturb the excavation base. In addition, excavation bottoms may be disturbed or deform excessively under the wheel loads of heavy construction vehicles as the excavations approach the required depths. Construction equipment should be as light as possible to limit development of this condition. The use of track-mounted vehicles is recommended since they exert lower contact pressures.

Groundwater was encountered in Borings B-2 and B-4 through B-6 at depths between approximately 9.5 and 19 feet bgs. Based on the anticipated depths of earthwork and construction, a static groundwater table is not expected to be encountered during construction of the project. Perched water conditions may be encountered on portions of the site in the alluvium. Yielding subgrade conditions may be encountered in areas where site excavations are within 5 feet of the perched groundwater table. Stabilization recommendations are provided in Section 8.1.3 of this report.

8.1.2. Site Grading

Prior to grading, the ground surface in proposed structure and improvement areas should be cleared of any surface obstructions, debris, topsoil, organics (including vegetation), and other deleterious material. Materials generated from clearing operations should be removed from the project site for disposal (e.g. at a legal landfill site). Obstructions that extend below finish grade, if present, should be removed and resulting voids filled with compacted soil or cement slurry, in accordance with the recommendations of the geotechnical consultant. We anticipate a stripping depth of approximately 6 to 8 inches. On-site topsoil should not be incorporated into engineered fill, but may be stockpiled for re-use as landscaping material or other non-structural material.

Prior to placement and compaction of engineered fill, the project's geotechnical consultant should observe excavation bottoms to evaluate the exposed soils and if removals to more competent soils are needed. In areas that will receive engineered fill, the exposed soils should be scarified to a depth of 6 inches, moisture-conditioned to approximately optimum moisture content, and compacted to 95 percent or more relative compaction as evaluated by ASTM D 698.

Based on our subsurface exploration, we anticipate the bearing conditions will be variable across the site. Due to the variability of subsurface conditions across the site and the high potential for post-construction vertical movement, we have developed grading and foundation recommendations specific to the Office Control Building, the Water Treatment Building, and the liquid storage tanks.

In order to reduce the potential for post-construction total and differential vertical movement, we recommend construction of a 5-foot thick fill prism beneath the bottom of the finished floor of the Water Treatment Building and a 4-foot thick fill prism beneath the bottom of the mat foundation for the liquid storage tanks. The existing alluvium below structure footprints should be removed to the required depth and replaced as moisture conditioned and compacted engineered fill. The fill prisms should be constructed to extend 5 or more feet laterally beyond structure footprints.

Pavement and exterior flatwork may be placed on 12 or more inches of moisture conditioned and compacted engineered fill.

The following table summarizes recommended foundation types and overexcavation depths needed to provide an adequate layer of engineered fill beneath proposed project improvements.

Table 1 – Summary of Recommended Foundation Types and Overexcavation Depths

Proposed Improvement	Recommended Foundation Type	Recommended Overexcavation Depth (ft)
Office Control Building	Drilled Piers with Structural Floor	0
Water Treatment Building	Spread Footings with Slab-on-Grade Floor	5
Liquid Storage Tanks	Mat Slabs	4

NOTE:. Overexcavation depth may include approximately 6 inches of scarified, moisture-conditioned, and compacted in-place subsurface soils exposed in the bottom of overexcavations. Any loose, soft, and/or disturbed native soils should be removed from proposed structure and improvement areas. Deeper overexcavation than shown may be needed in some areas.

8.1.3. Subgrade Stabilization

As previously indicated, a static groundwater table is not expected to be encountered during construction. However, perched water conditions may be encountered on portions of the site in the alluvium and pumping conditions may be encountered in excavations near the groundwater table.

Stabilization methods should be provided by the grading contractor, as needed, and may include the use of a geogrid, such as Tensar TX160, BX1100 or a woven geotextile fabric, such as Mirafi 600X, placed on unstable subgrade and overlain by 12 inches of crushed rock or aggregate base. Pushing oversized angular rock, up to approximately 6 inches in nominal diameter, into exposed unstable subgrade soils may also be an appropriate stabilization alternative. The volume of rock needed will vary based upon factors including the moisture content of the native soil, soil type, depth to groundwater, and total affected area. Placement of angular rock should continue until the area exhibits a relatively non-yielding behavior as observed or tested by the geotechnical consultant.

If conditions (e.g. excavations extending below groundwater) are observed that indicate additional stabilization efforts may be needed, a combination of overexcavation, rock fill, and geogrid placement should be considered. Dewatering and use of relatively light or tracked equipment may also be needed. The geotechnical consultant/engineer during construction should evaluate proposed subgrade stabilization methods prior to their implementation.

8.1.4. Fill Placement and Compaction

Based on the laboratory test results and our general observations, it is our opinion the native site soils may be suitable for reuse as engineered fill provided they are processed and moisture conditioned in accordance with the recommendations provided herein. Laramie Formation bedrock should not be used as engineered fill.

Engineered fill soils should not contain expansive soil (swell potential greater than 1 percent under a pressure of 200 psf when remolded at optimum moisture content), organic material, claystone bedrock fragments, or other deleterious material.

Soils used as engineered fill should be moisture-conditioned to moisture contents within 2 percent of optimum moisture content and placed in uniform horizontal lifts. Engineered fill should be compacted to 95 percent, or more, of the maximum proctor density as evaluated by ASTM D 698.

Fill should be compacted by appropriate mechanical methods using vibratory compaction equipment. The optimal lift thickness of fill will depend on the type of soil and compaction equipment used, but should generally not exceed approximately 8 inches in loose thickness. Fill materials should not be placed, worked, or rolled while they are frozen or thawing, and should not be placed during poor/inclement weather conditions.

Earthwork operations should be observed and compaction of engineered fill and backfill materials should be tested by the project's geotechnical consultant. Typically, one field test should be performed, per lift, for each approximately 500 cubic yards of fill placement in structural areas. Additional field tests may also be performed in structural and non-structural areas at the discretion of the geotechnical consultant. Compaction areas should be kept separate, and no lift should be covered by another until relative compaction and moisture content within the recommended ranges are obtained.

8.1.5. Imported Fill Material

Imported material should consist of relatively impervious soil with 30 to 50 percent passing the No. 200 sieve, a low sulfate content (less than 0.1 percent), a low swell potential (approximately 1 percent or less when wetted against a surcharge pressure of 200 psf), and a low plasticity index (approximately 15 or less). Import soil should not contain organic material, clay lumps, bedrock (claystone, sandstone, etc.) fragments, debris, other deleterious matter, or rocks or hard chunks larger than approximately 4 inches nominal diameter.

Imported fill soils should exhibit low corrosion potential. Imported materials placed in contact with ferrous materials should have a saturated soil resistivity of 2,000 ohm-cm or more and a chloride content of 25 parts per million (ppm) or less. Soils in contact with concrete should exhibit a soluble sulfate content less than 0.1 percent.

We further recommend that proposed import material be evaluated by the project's geotechnical consultant at the borrow source for its suitability prior to importation to the project site. Import soil should be moisture-conditioned and placed and compacted in accordance with the recommendations set forth in the previous section.

8.2. Utility Installation

The Contractor should take care to achieve and maintain adequate compaction of the backfill soils around valve risers and other vertical pipeline elements where settlements commonly are observed. Use of "flowable fill," i.e., a lean, sand-cement slurry, or a similar material should be considered in lieu of compacted soil backfill for areas with low tolerances for surface settlements. This would also reduce the permeability of the utility trenches.

Special care should be exercised to avoid damaging pipes or other structures during the compaction of the backfill. In addition, the underside (or haunches) of buried pipes should be supported on bedding material that is compacted as described above. This may need to be performed with placement by hand or small-scale compaction equipment.

8.2.1. Pipe Bedding Materials and Modulus of Soil Reaction (E')

The alluvium encountered will not be suitable for use as free-draining pipe bedding. We recommend pipes be supported on 6 inches or more of graded granular bedding material such as sand and gravel, or crushed rock with a particle size of 3/4-inch or less. To help limit the amount of fines from the excavation sides and bottom washing onto the void spaces of the bedding material after construction, we recommend the bedding material be encapsulated in a non-woven geotextile fabric, such as Mirafi 140N or equivalent.

Pipe bedding materials, placement and compaction should meet pipe manufacturer and applicable municipal standards. Materials proposed for use as pipe bedding should be tested for suitability prior to import.

The modulus of soil reaction (E’) is used to characterize the stiffness of soil backfill placed at the sides of buried pipelines for the purpose of evaluating deflection caused by the weight of the backfill over the pipe. For alluvial backfill soils, we recommend using an E´ value of 1,000 psi.

8.2.2. Water Transfer in Utility Trenches and Pipe Bedding

Bedding materials should be brought up evenly on both sides of pipes to reduce development of unbalanced loads on the pipe. Flooding or jetting of bedding materials should not be permitted.

Development of site grading plans should consider the subsurface transfer of water in utility trenches and the pipe bedding. Pipe bedding materials can function as efficient conduits for re-distribution of natural and applied waters in the subsurface. Cut-off walls in utility trenches or other water-stopping measures, and/or outlet mechanisms for saturated bedding materials should be incorporated to reduce the rates and volumes of water transmitted along utility alignments and toward buildings, pavements and other structures where excessive wetting of the underlying soils will be damaging. These measures also will reduce the risk of loss of fine-grained backfill soils into the bedding material, with resultant surface settlement.

8.3. Temporary Excavations and Shoring

Temporary excavations will be needed for this project to construct foundations and install utilities. Based on the subsurface information obtained from our exploratory borings and our experience with similar projects, we anticipate that site soils may slough or cave during excavation.

The contractor should provide safely sloped excavations or an adequately constructed and braced shoring system, in compliance with Occupational Safety and Health Administration (OSHA) regulations, for employees working in excavations that may expose them to the danger of moving ground. Reducing the inclination of the sidewalls of the excavations, where feasible, may increase the stability of the excavations. If construction or earth material is stored, or equipment is operated near an excavation, flatter slope geometry or shoring should be used during construction.

In our opinion, the native site soils should generally be considered a Type C soil when applying the OSHA guidelines. For these soil conditions, OSHA recommends a temporary slope inclination of 1.5H:1V or flatter for excavations 20 feet or less in depth. Steeper cut slopes may be utilized for excavations less than 4 feet deep depending on the strength, moisture content, and homogeneity of the soils as observed in the field. Appropriate slope inclinations for fill materials and alluvial deposits should be evaluated in the field by an OSHA-qualified “Competent Person” based on the conditions encountered.

8.4. Shallow Foundations

The following sections provide our foundation recommendations for shallow, conventional spread footings and mat foundations bearing on engineered fill compacted in accordance with recommendations set forth in previous sections.

8.4.1. Conventional Spread Footings

Spread footing foundations may be utilized to support the proposed Water Treatment Building provided the remedial grading and over-excavation recommendations presented herein are followed. Spread or continuous footings may be designed using a net allowable bearing capacity of 2,000 pounds per square foot (psf). Interior footings should be placed 18 or more inches below the lowest adjacent finished grade and perimeter footings should extend to 36 inches or more below the lowest exterior finished grade for frost protection. Continuous and isolated footings should have a width of 24 or more inches.

The average footing bearing pressure should not exceed the allowable equivalent uniform bearing pressure tabulated above. However, peak edge stresses may exceed these values as long as the resultant passes through the middle third of the footing base. The allowable soil bearing pressure may be increased by one-third when considering total loads including loads of short duration such as wind or seismic forces. Seismic parameters for design of structures at the site are provided in the referenced Geologic Hazards Report (Ninyo & Moore, 2013).

Positive drainage should be established and maintained around the proposed improvements to direct water away from building foundations. The recommended allowable bearing pressure was based on an assumption of drained conditions. If foundation materials become wet, the effective bearing capacity will be reduced and larger post-construction movements than those estimated below may result.

If the recommendations provided in this report are implemented in design and construction, and positive surface drainage away from the structures is maintained during the life of the project, we estimate total vertical post-construction movement of foundations to be approximately 1-inch. Differential movements for the proposed water treatment building should be on the order of 1/2 to 3/4 of the estimated total vertical movement. These estimates are based on the anticipated loading conditions, the available soil boring information, and our experience with similar soils.

Lateral resistance for footings is presented in Section 8.8. The foundations should preferably be proportioned such that the resultant force from total footing loads, including lateral loads, falls within the kern (i.e., middle one-third of the footing base).

Compacted fill placed against the sides of the footings should be compacted to 95 percent or more, relative compaction in accordance with the recommendations provided in Section 8.1.2.

8.4.2. Mat Foundations

The proposed liquid storage tanks may be supported on mat foundations with frost protected turned down edges bearing on a zone of engineered fill prepared in accordance with the recommendations provided in Section 8.1.2 of this report. The mat foundation may be designed using a net allowable bearing capacity of 2,000 psf. The total and differential settlement corresponding to this allowable bearing load are estimated to be less than approximately 1 inch and 3/4 inch over a horizontal span of 40 feet, respectively.

Mat foundations typically experience some deflection due to loads placed on the mat and the reaction of the soils directly underlying the mat. A design modulus of subgrade reaction (K) of 90 tons per cubic foot (tcf) may be used for the subgrade soils in evaluating such deflections. This value is based on a unit square foot area and should be adjusted for large mats. Adjusted values of the modulus of subgrade reaction, K_v, can be obtained from the following equation for mats of various widths:

$$K_v = K[(B+1)/2B]^2 \quad \text{(tcf)}$$

B in the above equation represents the width of the mat in feet.

8.5. Drilled Pier Foundations

We recommend that the Office Control Building be supported on a drilled pier foundation system with a structural floor. In Boring B-1, which was drilled at the proposed location of the Office Control Building, we encountered shallow Laramie Formation bedrock. The results of our swell/consolidation testing performed on samples obtained in Boring B-1 indicated moderate to high swell potential, see Figures B-5 and B-6. Based on our calculations, if these materials experienced changes in moisture content, the post-construction vertical movement would be substantially greater than what can be tolerated by a conventional spread footing/slab-on-grade foundation system.

The design considerations presented below should be considered during drilled pier foundation system design. The construction details and other considerations presented in this report should also be considered when preparing project documents. If the measures outlined in this report are implemented effectively, the total vertical foundation movement will be less than ½-inch, provided that the drilled pier bearing materials are not significantly disturbed during construction. Differential movements are estimated to be of similar magnitude. This estimate is based on the soil and bedrock conditions between piers disclosed by the borings, anticipated conditions, and our experience with similar geologic materials.

8.5.1. Drilled Pier Design Considerations

Piers bearing in formational bedrock may be designed for a net allowable end bearing pressure of up to 20,000 psf. The portion of the pier penetrating formational bedrock may be designed for an allowable skin friction (in downward axial compression) of up to 2,000 psf. This allowable skin friction value is applicable to provide bearing support and resist uplift. Piers should also be designed for a minimum dead load pressure of 10,000 psf based on pier end area only.

Piers should have a length of 29 feet or more and penetrate 9 feet or more into competent formational bedrock. Both criteria for pier length and bedrock penetration should be met. If the minimum dead load requirement cannot be achieved, and the piers are spaced as far apart as practical, the pier length should be extended above recommended minimum length to make up the dead load deficit. This could be accomplished by assuming skin friction located in the extended zone acts in the direction to resist uplift.

A pier diameter of 18 or more inches or 5 percent of the expected total shaft length, whichever is greater, is recommended to facilitate cleaning and observation of the pier hole. The structural engineer should design the actual length to diameter ratio.

Bedrock penetration in pier holes should be roughened artificially to assist the development of peripheral shear between the pier and bedrock. Artificially roughening of pier holes should consist of installing 3 inches high and 2 inches deep shear rings placed at 18 inches on center within the bedrock penetration zone within the bottom 9 or more feet of each pier. Shear rings should not be installed in the upper 15 feet of the drilled piers.

Piers should be reinforced for their full length to resist the ultimate tensile load created by the on-site swelling materials. Tension may be estimated based on an uplift pressure of 1,200 psf for formational bedrock located within the upper 15 feet of material penetrated by the pier and on the surface area of the pier.

We understand that the lateral load analysis of shafts will be performed by others. The parameters tabulated below may be used for lateral analysis of drilled piers for resistance to lateral loads. The parameters were developed based on the field and laboratory data obtained for the subject site and our experience with similar sites and conditions.

A simplified soil / bedrock profile with approximate unit wet weights (γ, γ_{sub}), angles of internal friction (ϕ), undrained shear strength (c_u), for the earth materials, as well as values for strain at 50 percent of failure stress (ε_{50}) and modulus of horizontal subgrade reaction (k_h) is presented in Table 2. Resistance to lateral loads should be neglected in the upper 3 feet of the existing ground surface. Cased zones (if casing is utilized) should not be included in load calculations and the lengths of individual piers should be increased correspondingly.

Table 2 – Recommended Lateral Load Parameters

Material Type	γ (pci)	γ_{sub} (pci)	ϕ (deg)	c_u (psi)	k_h (pci)	ε_{50}
Engineered Fill	0.060	-	27	10	400	0.007
Alluvium	0.058	.0220	-	10	150	0.010
Formational Bedrock	0.063	.0270	-	28	1,300	0.005

For lateral loading, piers in a group may be considered to act individually when the center-to-center spacing is greater than 3D (where, D is the diameter of the pier) in the direction normal to loading and greater than 5D in the direction parallel to loading. The following table presents the lateral load reduction factors to be applied for various pier spacing for in-line loading. Linear interpolation may be used for spacings that are between 3D and 5D.

Table 3 – Lateral Load Group Reduction Factors

Center-to-Center Pier Spacing for In-Line Loading	Reduction Factor*		
	Row 1	Row 2	Row 3 and higher
3D	0.8	0.40	0.3
5D	1.0	0.85	0.7

Note:
* Based on AASHTO LRFD Bridge Design Specifications, 5th Edition, 2010 Interim Revision

A 12-inch or thicker void form should be placed beneath grade beams and beneath pier caps. The void space that will be created after the void form disintegrates should be protected by a backfill retainer to discourage backfill soils from migrating into the void space on both sides.

8.5.2. Drilled Pier Construction Considerations

Our evaluation of the excavation characteristics of the on-site materials is based on the results of our exploratory borings, site observations, and experience with similar materials. Resistant bedrock was encountered in our borings. Difficult drilling conditions may be encountered during pier hole drilling. The pier-drilling contractor should be prepared to core lenses and beds of highly cemented sandstone bedrock that may be present within the Laramie Formation.

The pier-drilling contractor should mobilize equipment of sufficient size and operating capability to achieve the recommended penetration into the bedrock. The excavation technique chosen by the contractor should not adversely affect the quality or strength of the shaft side or end bearing materials. If refusal is encountered in these materials either during the test program or during actual installation, the geotechnical engineer should be retained to evaluate the conditions to establish that true refusal has been met with adequate drilling equipment.

Groundwater was not encountered in Boring B-1, which was drilled at the proposed location of the Office Control Building. However, groundwater was encountered in Borings B-2 and B-4 through B-6 during the subsurface exploration. Groundwater may be present where not encountered during our subsurface exploration. The contractor should be prepared to advance the piers in the presence of groundwater. Casing may be needed in the pier holes to reduce water infiltration.

The concrete may be placed by the free-fall method into piers that exhibit “dry” conditions (i.e. less than 3 inches of water). This method consists of using a vertical section of concrete chute to divert the concrete flow out of the truck in a vertical stream of concrete with a relatively small diameter. The stream should be diverted to avoid hitting the sides of the drilled shaft or the reinforcing cage, which could cause concrete segregation. In no case, should concrete be placed in more than 3 inches of water, unless placed using a tremie method.

Where water is present in the drilled pier hole, including outside of a casing (if utilized) that will be withdrawn from the hole, the concrete placed for the pier should have sufficient slump and be placed with sufficient head maintained above groundwater levels so that the concrete is not displaced in the body of the pier by water, soil, etc., leading to effective voids in the pier. Concrete utilized in the piers should be a fluid mix with sufficient slump so that it will fill the void between reinforcing steel and the pier hole wall. We recommend the concrete have a slump in the range of 6 inches +/–1-inch.

The contractor should take care to reduce enlargement of the excavation at the tops of piers, which could result in mushrooming of the pier top. Pier holes should be cleaned prior to placement of concrete. Care should be taken to check that the soils at the pier bottom have not been disturbed. The movement associated with mobilizing the end-bearing component should not be beyond tolerable structural limits. The successful advancement of drilled excavations for the construction of drilled shafts will depend largely on the suitability of the drilling equipment and skill of the operator. The drilled foundation contractor should try to reduce the time during which the excavation remains open. The contractor should schedule the sequence of operations so that each excavation can be finished, reinforcing steel placed, and the concrete poured in a rapid and timely manner.

The contractor should not place drilled piers adjacent to each other until the first one is set. The installation of piers should be scheduled to allow the concrete in adjacent shafts to set before drilling the next shaft. Drilled piers spaced closer than about four shaft diameters (clear spacing) should be placed on alternate days and drilled shaft excavations should not be left open over night.

The drilled pier excavations should be evaluated to check that adequate bearing material has been reached and that the bearing surface has been suitably cleaned. In the event lignite (black in color) is encountered within the bedrock penetration or end bearing zone, piers will have to be deepened to adequate bearing material as determined by the geotechnical engineer. This evaluation can typically be done at the surface. Installation should be observed by the Geotechnical Engineer or qualified representative to check that, among other things: 1) subsurface conditions are as anticipated from the borings, 2) the drilled shafts are constructed to the specified size and penetration, 3) drilled shafts are within allowable tolerances for plumbness, and 4) reinforcements are placed per project specifications. These items are fundamental to the installation and behavior of the drilled shafts in accordance with our recommendations. Furthermore, we recommend the following for the installation of drilled shafts.

- The clear spacing between bars or behind the rebar cage should be more than three times the maximum size of the coarse aggregate used in the concrete.
- Centralizers on the rebar cage should be installed to keep the cage positioned per project specifications.
- Cross bracing of a reinforcing cage may be used when fabricating, transporting, and lifting. However, experience has shown that cross bracing can contribute to the development of voids in a concrete shaft. Therefore, we recommend removing the cross bracing prior to lowering the reinforcing cage into the open excavation.
- If casing is used, a sufficient head of concrete that fills the casing should be used before pulling the casing.
- Concrete tremied into a shaft with slurry (if utilized) should maintain a hydrostatic pressure in excess of either the surrounding water table or slurry in the excavation.

We should be given an opportunity to review the proposed specifications and the contractor’s installation procedures prior to construction.

8.6. Structural Floors

We recommend the Office Control Building supported on a drilled pier foundation system be provided with a structural floor. Structural floors should be supported on grade beams and straight shaft drilled piers. Requirements for the number and position of additional piers to support the structural floors will depend upon the span, design load, and structural design, and should be developed by the Structural Engineer. Geotechnical recommendations for design and installation of drilled piers are provided in the previous section.

Structural floors should be constructed to span above a well-ventilated crawl space 3 or more feet in height to allow access for maintenance of under floor utility piping. Interior utility lines should be suspended from the bottom of the structural floor and should be placed 12 or more inches above the site soils. In areas where utility piping supported by site soils enter the structural floor, positive bond breaks that allow 3 inches of differential movement should be used. Design and installation of associated fixtures should also accommodate this potential movement. Plumbing lines should be carefully tested before operation.

If a wooden structural floor system is used in the buildings, particular care should be taken to design and maintain the under-floor ventilation system in order to reduce potential deterioration of the wooden structural members.

A vapor barrier meeting ASTM E-1745 (Class "A") should be considered for installation below the structurally supported floor and should be attached/sealed to foundation walls/drilled piers above the void material. The sheet material should not be attached to horizontal surfaces such that condensate might drain to wood or corrodible metal surfaces.

New buildings generally lack ventilation due primarily to systematic efforts to construct airtight, energy-efficient structures. Therefore, areas such as crawl spaces beneath structural floors are typically areas of elevated humidity. Persistently warm, humid conditions in the presence of cellulose, which is the base material found in many typical construction products, creates an ideal environment for the growth of fungi, molds, and mildew. Published data suggest links between molds and negative health affects. Therefore, we recommend that the crawl space beneath the structural floor be provided with adequate, positive active ventilation system or other active mechanisms such as specially designed HVAC systems to reduce the potential for mold, fungus and mildew growth. The owner should understand the risks of potential mold, fungus, and mildew growth when utilizing a structural floor system. Crawl spaces should be inspected periodically so that remedial measures can be taken in a timely manner.

8.7. Slab-on-Grade Floors

The proposed Water Treatment Building may be designed for a slab-on-grade floor. The design of the floor slabs (including jointing and reinforcement) is the responsibility of the structural engineer. Joints should be constructed at intervals designed by the structural engineer to help reduce random cracking of the slab. However, from a geotechnical standpoint, we recommend that floor slabs have a thickness of 5 or more inches and be reinforced with steel as designed by a structural engineer. Placement of the reinforcement in the slab is vital for satisfactory performance. Soils underlying the slabs should be improved in accordance with the recommendations provided in Section 8.1.2.

Floor slabs should be separated from bearing walls and columns with expansion joints, which allow unrestrained vertical movement. Joints should be observed periodically, particularly during the first several years after construction. Slab movement can cause previously free-slipping joints to bind. Measures should be taken so that slab isolation is maintained in order to reduce the likelihood of damage to walls and other interior improvements. If post-construction vertical slab movement of about 1-inch cannot be tolerated or desired, than we recommend utilizing a structural floor system spanning over a void or a crawl space.

Interior partitions resting on floor slabs should be provided with slip joints so that if the slabs move, the movement cannot be transmitted to the upper structure, including wallboards and door frames. A slip joint that allows 2 or more inches of vertical movement is recommended for placement at the bottoms of the interior partitions. If slip joints are placed at the tops of walls, in the event that the floor slabs move, it is expected that the wall will show signs of distress, especially where the floors meet the exterior wall. Plumbing lines should be carefully tested before operation. Where plumbing lines enter through the floor, a positive bond break should be provided. Flexible connections allowing 2 or more inches of vertical movement should be provided for slab-bearing mechanical equipment.

The slab should be underlain by 4 or more inches of moist clean sand and/or gravel. The need for a moisture-retarding system should be considered by the structural engineer or architect based on the moisture sensitivity of the anticipated flooring.

8.8. Lateral Earth Pressures

Walls that are not restrained from movement at the top and have a level backfill behind the wall may be designed using an “active” equivalent fluid unit weight of 45 pounds per cubic foot (pcf). This value assumes compaction within about 5 feet of the wall will be accomplished with relatively light compaction equipment, and that backfill meeting engineered fill requirements will be placed behind the wall to a distance of half or more of the wall height. Unrestrained retaining walls and below-grade pit walls should also be designed to resist a surcharge pressure of 0.35q. The value for “q” represents the pressure induced by adjacent light loads, slab, or traffic loads plus any adjacent footing loads.

The “at-rest” earth pressure against walls that are restrained at the top or braced so that they cannot yield, and with level backfill meeting the above stated criteria, may be taken as equivalent to the pressure exerted by a fluid weighing 67 pcf. Restrained retaining walls should also be designed to resist a horizontal earth pressure of 0.52q. The value of q represents the vertical surcharge pressure induced by adjacent light loads, slab, or traffic loads plus any adjacent footing loads.

For “passive” resistance to lateral loads, we recommend that an equivalent fluid weight of 250 pcf be used up to value of 2,500 psf. This value assumes that the ground is horizontal for a distance of 10 feet or more behind the wall or three times the height generating the passive pressure, whichever is more. We recommend that the upper 24 inches of soil not protected by pavement or a concrete slab be neglected when calculating passive resistance. For frictional resistance to lateral loads, we recommend that a coefficient of friction of 0.34 be used between soil and concrete. If passive and frictional resistances are to be used in combination, we recommend that the passive resistance be limited to one-half of the ultimate lateral resistance. The passive resistance values may be increased by one-third when considering loads of short duration such as wind or seismic forces.

Retaining walls should be backfilled and provided with a drain. Drainpipes should outlet away from structures, and retaining walls should be waterproofed in accordance with the recommendations of the project civil engineer or architect. To reduce the potential for water- and sulfate/salt-related damage to the retaining walls, particular care should be taken in selection of the appropriate type of waterproofing material to be utilized and in the application of this material. For exterior retaining walls, weepholes may be used in lieu of drainage pipes.

8.9. Pavements

We understand project pavements will be privately maintained. Pavement section alternatives for the paved surfaces including standard duty (i.e. automobile access and parking) and heavy duty (i.e. truck access, drive lanes, and staging areas) areas were developed in general accordance with the guidelines and procedures of the American Association of State Highway and Transportation Officials (AASHTO), the Colorado Department of Transportation (CDOT), and Weld County.

8.9.1. Pavement Subgrade Support

The subgrade soils encountered during the subsurface exploration typically consisted of sandy clay and clayey sand materials that classify as A-6 to A-7-5 soils in accordance with the AASHTO (2011) classification system.

A resistance value (R-Value) of 3 was established from classifications of composite soil samples representative of subsurface soils at the site. The R-Value of 3 correlates to a subgrade resilient modulus (M_R) of 2,834 pounds per square inch (psi) using equations developed by CDOT (2013). If during construction the subgrade is found to vary from the expected soil conditions, we should be contacted so we may re-evaluate our recommended resilient modulus value.

8.9.2. Design Traffic

Specific traffic loadings for the project were not available at the time of this report preparation. Equivalent 18-kip single axle load applications (ESAL's) of 36,500 and 500,000 for 20-year design lives were assumed for standard duty pavements (i.e. automobile access and parking) and heavy duty pavements (i.e. truck access, drive lanes, and staging areas), respectively. If design traffic loadings differ significantly from these assumed values, we should be notified to re-evaluate the pavement recommendations below.

8.9.3. Pavement Design

Pavement designs for the site were based on the "Guide for Design of Pavement Structures" by the American Association of State Highway and Transportation Officials (AASHTO, 1993).

The design of flexible pavements was based on the following input parameters:

Initial Serviceability:	4.5
Terminal Serviceability:	2.0
Reliability	80 %
Overall Standard Deviation:	0.44
Resilient Modulus:	2,834 psi
Stage Construction:	1

The design of rigid pavements was based on the following input parameters:

Initial Serviceability:	4.5
Terminal Serviceability:	2.0
28-Day Mean PCC Modulus Rupture:	650 psi
28-Day Mean Elastic Modulus of Slab:	3.6×10^6 psi
Mean Effective k value:	150 psi/in
Reliability:	90%
Overall Standard Deviation:	0.35
Load Transfer Coefficient:	4.2
Overall Drainage Coefficient:	1

8.9.4. Pavement Section Recommendations

Based on the above-mentioned design traffic and input parameters, and following the AASHTO method of pavement design (AASHTO, 1993), the following structural sections were calculated for standard duty and heavy duty pavements. Strength coefficients used in the design of the pavement sections were provided by CDOT (2013). Table 4 summarizes the recommended pavement sections.

Table 4 – Recommended New Pavement Sections

Traffic Type	AC / ABC (inches)	PCC / ABC (inches)
Standard Duty	5 / 6	-
Heavy Duty	7 / 10	6.5 / 6

8.9.5. Pavement Materials

The asphalt pavement shall consist of a bituminous plant mix composed of a mixture of high quality aggregate and bituminous material, which meets the requirements of a job-mix formula established by a qualified engineer. Grading S should be used for the lower lift(s) and grading SX should be used for the surface course. Pavement layer thickness should be 2 to 3 inches for the lower lift(s) and 1.5 to 2.5 inches for the surface course with tack coat layer(s) in between. The geotechnical engineer should be retained to review the proposed pavement mix designs, grading, and lift thicknesses prior to construction.

Concrete pavements should consist of a plant mix composed of a mixture of aggregate, Portland cement and appropriate admixtures meeting the requirements of Weld County. Concrete should have a modulus of rupture of third point loading of 650 psi or more. The concrete should be air-entrained with approximately 6 percent air and should have a cement content of 6 or more sacks per cubic yard. Allowable slump should be 4 inches.

Thickened edges should be used along outside edges of concrete pavements. Edge thickness should be 2 inches or more than the concrete pavement thickness and taper to the actual concrete pavement thickness 36 inches inward from the edge. Integral curbs may be used in lieu of thickened edges.

Concrete pavements should have longitudinal and transverse joints that meet the applicable requirements of Weld County. In areas of repeated turning stresses, we recommend that the concrete pavement joints be fully tied and doweled. We suggest that civil design consider joint layout in accordance with CDOT's M Standards.

The aggregate base material placed beneath pavements should meet the criteria of CDOT Class 6 aggregate base. Requirements for CDOT Class 6 aggregate base can be found in Section 703 of the current CDOT Standards and Specifications for Road and Bridge Construction.

8.9.6. Pavement Subgrade Preparation

For both the flexible and rigid pavement sections recommended above, we recommend the underlying subgrade soils be prepared as described in Section 8.1 of this report.

The contractor should be prepared either to dry the subgrade materials or moisten them, as needed, prior to compaction. Some site soils may pump or deflect during compaction if moisture levels are not carefully monitored. The contractor should be prepared to process and compact such soils to establish a stable platform for paving, including use of chemical stabilization or geotextiles, where needed.

The prepared subgrade should be protected from the elements prior to pavement placement. Subgrades that are exposed to the elements may need additional moisture conditioning and compaction, prior to pavement placements.

Immediately prior to paving, the pavement subgrade should be proof rolled with a heavily loaded, pneumatic tired vehicle, and checked for moisture content. Areas that exhibit excessive deflection (as evaluated by the geotechnical engineer) during proof rolling should be excavated and replaced and/or stabilized.

8.9.7. Pavement Maintenance

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

As noted above, the standard care of practice in pavement design describes the recommended flexible pavement section as a “20-year” design pavement; however, many pavements will not remain in satisfactory condition without routine, preventive maintenance and rehabilitation procedures performed during the life of the pavement. Preventive pavement treatments are surface rehabilitation and operations applied to improve or extend the functional life of a pavement. These treatments preserve, rather than improve, the structural capacity of the pavement structure. In the event the existing pavement is not structurally sound, the preventive maintenance will have no long-lasting effect. Therefore, a routine maintenance program to seal joints and cracks, and repair distressed areas is recommended.

8.10. Concrete Flatwork

It should be noted that ground-supported flatwork such as walkways will be subject to soil-related movements resulting from heave/settlement, frost, etc. Thus, where these types of elements abut rigid building foundations or isolated/suspended structures, differential movements should be anticipated. We recommend that flexible joints be provided where such elements abut the main structure to allow for differential movement at these locations.

We recommend that exterior concrete flatwork be supported on improved subgrade as described in Section 8.1.2 of this report. Positive drainage should be established and maintained adjacent to flatwork. Water should not be allowed to pond on or adjacent to flatwork.

In no case should exterior flatwork extend to under any portion of the building where there is less than 3 inches of clearance between the flatwork and any element of the building. Exterior flatwork in contact with brick, rock facades, or any other element of the building can cause damage to the structure if the flatwork experiences movements.

Prior to placement of flatwork, a proof roll should be performed to evaluate areas that exhibit instability and deflection. The soils in these areas should be removed and replaced with engineered fill or stabilized.

8.11. Site Drainage

Surface drainage should be provided to divert water away from the proposed structures and off of paved surfaces. Surface water should not be permitted to drain toward the structures or to pond adjacent to foundation walls or on paved surfaces. Positive drainage is defined as a slope of 2 or more percent for a distance of 10 or more feet away from the structures. Roof gutters should be installed on structures. Downspouts should discharge to drainage systems away from structures, pavements, and flatwork.

Vegetation that may need irrigation should not be located within 5 feet of structure foundation perimeters. Irrigation sprinkler heads should be deployed so that applied water is not introduced within 5 feet of the foundation perimeters. Landscape irrigation outside the 5-foot limit should be limited to sustain healthy plant growth.

8.12. Corrosivity

The corrosion potential of the on-site materials was analyzed to evaluate its potential effect on the foundations and structures. Corrosion potential was evaluated using the results of laboratory testing of a sample obtained during our subsurface evaluation that was considered representative of soils at the subject site.

Laboratory testing consisted of pH, minimum electrical resistivity, chloride, and soluble sulfate contents. Soil pH and minimum resistivity tests were performed on a representative sample in general accordance with ASTM D 4972. The chloride content of a selected sample was evaluated in general accordance with CDOT Laboratory Procedure 2104. The sulfate content of a selected sample was evaluated in general accordance with CDOT Laboratory Procedure 2103. The results of the corrosivity tests are presented in Appendix B.

Based on the values obtained for the soil parameters, the site soils are considered severely corrosive to ferrous metals. Corrosive conditions can be addressed by use of materials not vulnerable to corrosion, heavier gauge materials (increased pipe wall/metal thickness) with longer design lives, polyethylene encasement, or cathodic protection systems. A corrosion specialist should be consulted for further recommendations.

8.13. Water Soluble Sulfates and Concrete

Laboratory chemical tests performed on an on-site soil sample indicated a water soluble sulfate contents of up to 0.01 percent by weight. Based on review of the referenced International Building Code (ICC, 2009) and American Concrete Institute (ACI, 2005) the tested soil is considered to have a negligible sulfate exposure to concrete. Notwithstanding the sulfate test results and due to the limited number of chemical tests performed, as well as our experience with similar soil conditions and local practice, we recommend the use of "Type II" cement for construction of concrete structures at this site.

The concrete should have a water-cementitious materials ratio of no more than 0.50 by weight for normal weight aggregate concrete. The structural engineer should ultimately select the concrete design strength based on the project specific loading conditions. However, higher strength concrete may be selected for increased durability, resistance to slab curling and shrinkage cracking. We recommend the use of concrete with a design 28-day compressive strength of 4000 psi or more, for concrete grade slabs at this site. Concrete exposed to the elements should be air-entrained.

8.14. Construction in Cold or Wet Weather

Given the soil conditions, it is important to avoid ponding of water in excavations. Water that accumulates in excavations should be promptly pumped out or otherwise removed and these areas should be allowed to dry out before resuming construction.

Earthwork activities undertaken during the cold weather season may be difficult and should be done by an experienced contractor. Fill should not be placed on top of frozen soils. The frozen soils should be removed prior to the placement of new engineered fill or other construction material. Frozen soil should not be used as structural fill or backfill. The frozen soil may be reused (provided it meets the selection criteria) once it has thawed completely. In addition, compaction of the soils may be more difficult due to the viscosity change in water at lower temperatures.

If construction proceeds during cold weather, foundations, or other concrete elements should not be placed on frozen subgrade soil. Frozen soil should either be removed from beneath concrete elements, or thawed and recompacted. To limit the potential for soil freezing, the time passing between excavation and construction should be minimized. Blankets, straw, soil cover, or heating could be used to discourage the soil from freezing.

8.15. Pre-Construction Conference

We recommend that a pre-construction conference be held. Representatives of the owner, civil engineer, the geotechnical consultant, and the contractor should be in attendance to discuss the project plans and schedule. Our office should be notified if the project description included herein is incorrect, or if the project characteristics are significantly changed.

8.16. Construction Observation and Testing

During construction operations, we recommend that a qualified geotechnical consultant perform observation and testing services for the project. These services should be performed to evaluate exposed subgrade conditions, including the extent and depth of overexcavation, to evaluate the suitability of proposed borrow materials for use as fill, to evaluate the stability of open temporary excavations, to evaluate the results of any dewatering activities, and to observe placement and test compaction of fill soils. If another geotechnical consultant is selected to perform observation and testing services for the project, we request that the selected consultant provide a letter to the owner, with a copy to Ninyo & Moore, indicating that they fully understand our recommendations and that they are in full agreement with the recommendations contained in this report. Qualified subcontractors utilizing appropriate techniques and construction materials should perform construction of the proposed improvements.

9. LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

10. SELECTED REFERENCES

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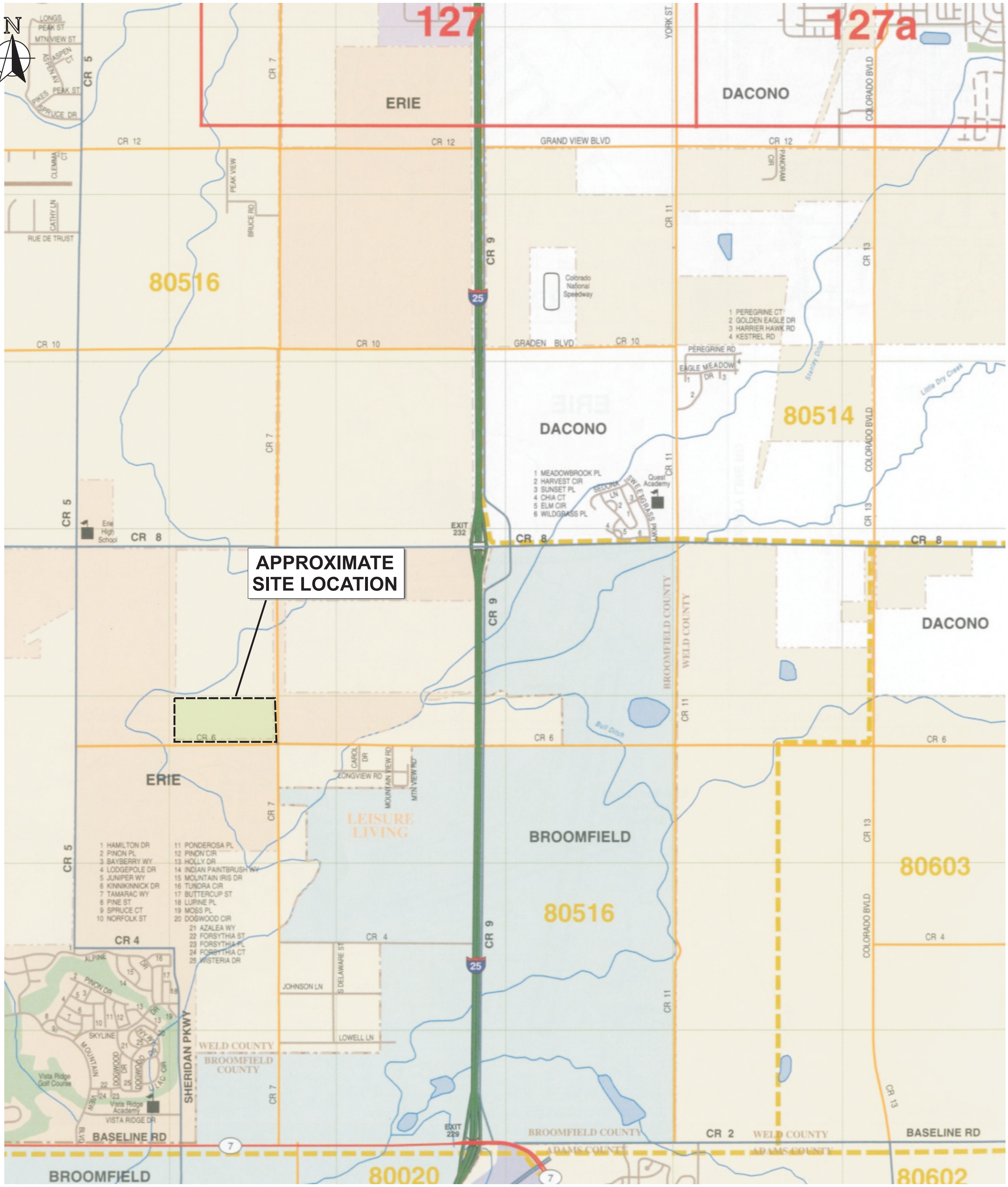
Aerial Photograph References

Source	Dates
Google Earth	June 1993; October 1999; August 2004; March 2008, October 2012



Source: Macvan Map Company, Denver Metro Edition, Colorado, 2010.

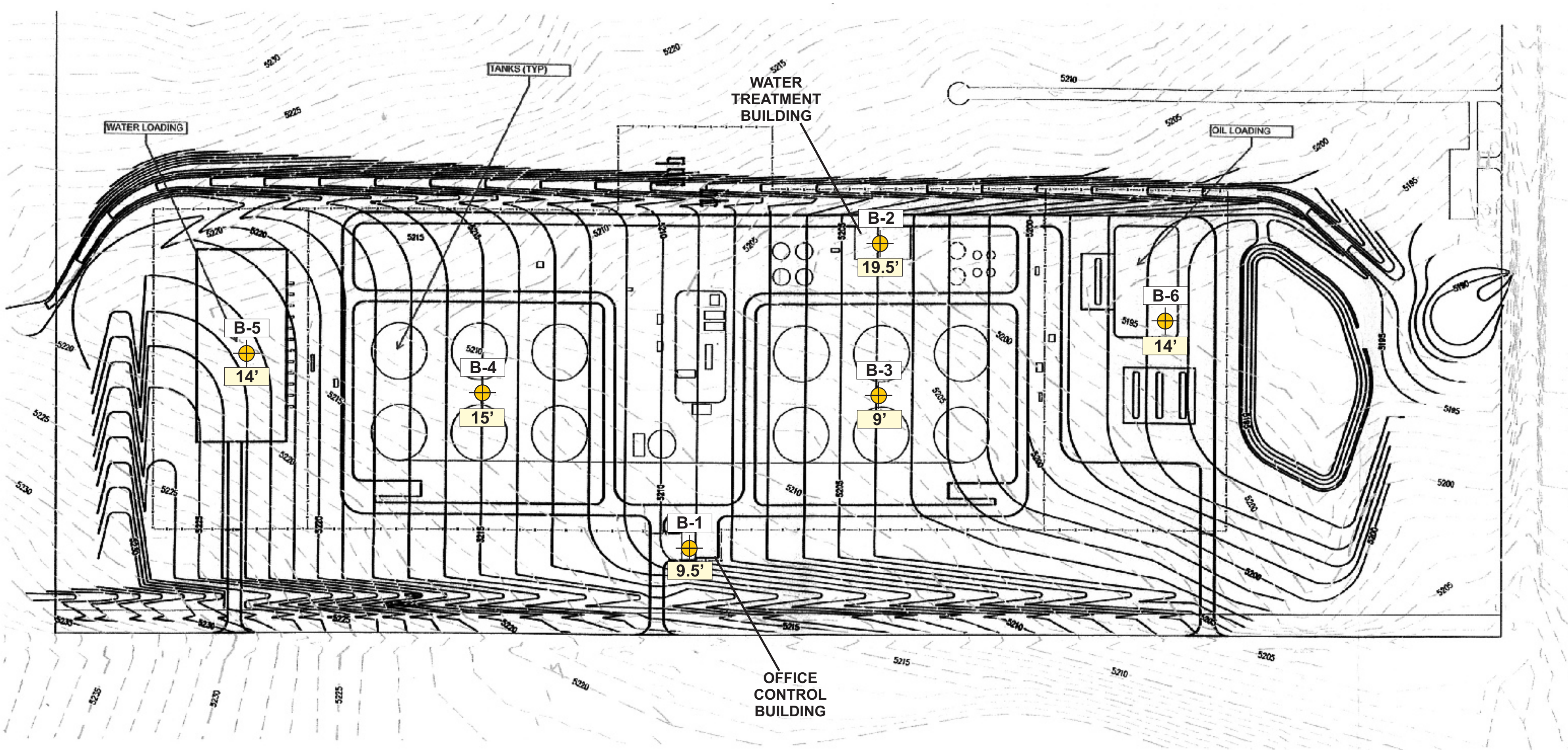
0 1900

Approximate Scale: 1 inch = 1900 feet

Note: Dimensions, directions, and locations are approximate.

Ninyo & Moore	SITE LOCATION	FIGURE
PROJECT NO: 500707001	DATE: 10/13	THE HUB FACILITY WELD COUNTY ROAD 6 AND COUNTY ROAD 7 WELD COUNTY, COLORADO	1

file no: 0707vmap0913



LEGEND

B-6	Boring Location
19.5'	Depth to Bedrock

N

NOT TO SCALE

Note: Dimensions, directions, and locations are approximate.

file no: 0707blm0913

Ninyo & Moore

BORING LOCATIONS

PROJECT NO: 500707001

DATE: 10/13

THE HUB FACILITY
WELD COUNTY ROAD 6 AND COUNTY ROAD 7
WELD COUNTY, COLORADO

FIGURE 2

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples
Disturbed soil samples were obtained in the field using the following methods.

Bulk Samples
Bulk samples of representative earth materials were obtained from the exploratory borings. The samples were bagged and transported to the laboratory for testing.

The Standard Penetration Test (SPT) Spoon
Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test spoon sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of $1^{3}/_{8}$ inches. The spoon was driven into the ground 12 to 18 inches with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D 1586-99. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the spoon, bagged, sealed and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples
Relatively undisturbed soil samples were obtained in the field using the following methods.

The Modified California Split-Barrel Drive Sampler
The sampler, with an external diameter of 3.0 inches, was lined with thin brass rings with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer or bar, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

The California Drive Sampler
The sampler, with an external diameter of 2.4 inches, was lined with four 4-inch long, thin brass rings with inside diameters of approximately 1.9 inches. The sample barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass liners, sealed, and transported to the laboratory for testing.

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0								Bulk sample.
								Modified split-barrel drive sampler.
								No recovery with modified split-barrel drive sampler.
								Sample retained by others.
								Standard Penetration Test (SPT).
5								No recovery with a SPT.
			XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
								No recovery with Shelby tube sampler.
								Continuous Push Sample.
								Seepage.
10								Groundwater encountered during drilling.
								Groundwater measured after drilling.
							SM	ALLUVIUM: Solid line denotes unit change.
								Dashed line denotes material change.
								Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Sheared Bedding Surface
15								
20								The total depth line is a solid line that is drawn at the bottom of the boring.

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BORING LOG		
EXPLANATION OF BORING LOG SYMBOLS		
PROJECT NO.	DATE Rev. 01/03	FIGURE

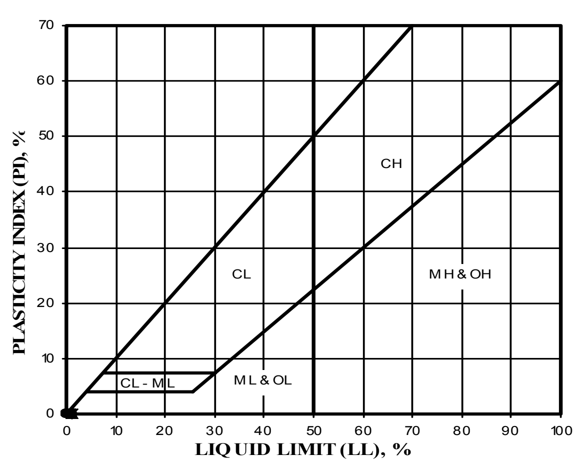
U.S.C.S. METHOD OF SOIL CLASSIFICATION

MAJOR DIVISIONS		SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS (More than 1/2 of soil >No. 200 sieve size)	**GRAVELS (More than 1/2 of coarse fraction > No. 4 sieve size)**	GW	Well graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS (More than 1/2 of coarse fraction <No. 4 sieve size)	SW	Well graded sands or gravelly sands, little or no fines
		SP	Poorly graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (More than 1/2 of soil <No. 200 sieve size)	**SILTS & CLAYS Liquid Limit <50**	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean
		OL	Organic silts and organic silty clays of low plasticity
	SILTS & CLAYS Liquid Limit >50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity, organic silty clays, organic silts
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils

GRAIN SIZE CHART

CLASSIFICATION	RANGE OF GRAIN SIZE	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS	Above 12"	Above 305
COBBLES	12" to 3"	305 to 76.2
GRAVEL **Coarse** **Fine**	3" to No. 4 3" to 3/4" 3/4" to No. 4	76.2 to 4.76 76.2 to 19.1 19.1 to 4.76
SAND **Coarse** **Medium** **Fine**	No. 4 to No. 200 No. 4 to No. 10 No. 10 to No. 40 No. 40 to No. 200	4.76 to 0.075 4.76 to 2.00 2.00 to 0.420 0.420 to 0.075
SILT & CLAY	Below No. 200	Below 0.075

PLASTICITY CHART



Ninyo & Moore | U.S.C.S. METHOD OF SOIL CLASSIFICATION

DATE DRILLED	9/06/13
BORING NO.	B-1
GROUND ELEVATION	5,212' ± (MSL)
SHEET	1 OF 1
METHOD OF DRILLING	CME-75, 4" Diameter Solid-Stem Auger (Precision Drilling)
DRIVE WEIGHT	140 lbs. (Auto Hammer)
DROP	30"
SAMPLED BY	DLH
LOGGED BY	DLH
REVIEWED BY	JMJ

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								TOPSOIL: Approximately 8 inches thick.
							CH	ALLUVIUM: Brown, damp, firm to very stiff, sandy CLAY with few calcium carbonate mineralization.
		■	20	9.0	88.8			
10		■	41	19.8	103.8			LARAMIE FORMATION: Brown to gray, damp to moist, moderately indurated, CLAYSTONE; iron staining and gypsum mineralization; weathered.
		⧗	24	18.5	118.3			
								Light brown, dry, moderately cemented, silty, fine-grained SANDSTONE; weathered.
20		⧗	50					Total Depth = 20 feet. Groundwater not encountered during drilling. Backfilled on 9/06/13 shortly after completion of drilling. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
30								
40								

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BORING LOG

THE HUB FACILITY

WCR 6 AND WCR 7, WELD COUNTY, COLORADO

PROJECT NO.	DATE	FIGURE
500707001	10/13	A-1

DATE DRILLED	9/06/13	BORING NO.	B-2
GROUND ELEVATION	5,201' ± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING	CME-75, 4" Diameter Solid-Stem Auger (Precision Drilling)		
DRIVE WEIGHT	140 lbs. (Auto Hammer)	DROP	30"
SAMPLED BY	DLH	LOGGED BY DLH	REVIEWED BY JMJ

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								TOPSOIL: Approximately 8 inches thick.
							SC	ALLUVIUM: Reddish brown, moist, medium dense, clayey SAND with few gravel.
		Driven	21	13.6	116.1			
							CL	Reddish brown, moist, very stiff, sandy CLAY.
10		Driven	15	19.1	108.1			
		Driven	34	16.3	116.5		GC	Reddish brown and brown, moist, dense, clayey GRAVEL with sand.
				▽				@19': Groundwater encountered during drilling.
20		Driven	41					LARAMIE FORMATION: Purplish gray, moist to saturated, moderately indurated, CLAYSTONE; weathered.
								Total Depth = 20.5 feet. Groundwater was measured at a depth of approximately 19 feet in borehole during drilling. Backfilled on 9/06/13 shortly after completion of drilling. Note: Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report.
30								
40								

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BORING LOG

THE HUB FACILITY
WCR 6 AND WCR 7, WELD COUNTY, COLORADO

PROJECT NO.	DATE	FIGURE
500707001	10/13	A-2

DATE DRILLED	9/05/13	BORING NO.	B-3
GROUND ELEVATION	5,207' ± (MSL)	SHEET 1 OF 2	
METHOD OF DRILLING	CME-75, 4" Diameter Solid-Stem Auger (Precision Drilling)		
DRIVE WEIGHT	140 lbs. (Auto Hammer)	DROP	30"
SAMPLED BY	DLH	LOGGED BY DLH	REVIEWED BY JMJ

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								TOPSOIL: Approximately 8 inches thick.
							CL	ALLUVIUM: Light brown to brown, damp, very stiff, sandy lean CLAY with few calcium carbonate mineralization.
			20					
10			33					LARAMIE FORMATION: Light brown to brown, damp, very stiff, moderately indurated; CLAYSTONE; trace iron staining.
			84	12.8	115.0			Few iron staining.
20			41	16.9	116.8			
			42	22.7	104.4			
30			76/11"					Gray.
			81/11"					Olive brown; few black carbonaceous mineralization.
40			57					

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BORING LOG

THE HUB FACILITY
WCR 6 AND WCR 7, WELD COUNTY, COLORADO

PROJECT NO.	DATE	FIGURE
500707001	10/13	A-3

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40								Total Depth = 40 feet. Groundwater not encountered during drilling. Backfilled on 9/05/13 shortly after completion of drilling. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
50								
60								
70								
80								

DATE DRILLED 9/05/13 BORING NO. B-3

GROUND ELEVATION 5,207' ± (MSL) SHEET 2 OF 2

METHOD OF DRILLING CME-75, 4" Diameter Solid-Stem Auger (Precision Drilling)

DRIVE WEIGHT 140 lbs. (Auto Hammer) DROP 30"

SAMPLED BY DLH LOGGED BY DLH REVIEWED BY JMJ

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BORING LOG

THE HUB FACILITY
WCR 6 AND WCR 7, WELD COUNTY, COLORADO

PROJECT NO.	DATE	FIGURE
500707001	10/13	A-4

DATE DRILLED 9/05/13 BORING NO. B-4

GROUND ELEVATION 5,212' ± (MSL) SHEET 1 OF 2

METHOD OF DRILLING CME-75, 4" Diameter Solid-Stem Auger (Precision Drilling)

DRIVE WEIGHT 140 lbs. (Auto Hammer) DROP 30"

SAMPLED BY DLH LOGGED BY DLH REVIEWED BY JMJ

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							TOPSOIL: Approximately 6 inches thick.
						CL	ALLUVIUM: Brown, moist, very stiff, sandy CLAY with little calcium carbonate mineralization.
	Driven	16					
10	Driven	18	13.9	115.0		GC	Reddish brown, saturated, medium dense, clayey GRAVEL with sand. @9.5': Groundwater encountered during drilling.
	Driven	32					Gray.
							LARAMIE FORMATION: Gray to brownish gray, moist to saturated, moderately indurated, CLAYSTONE with little black carbonaceous mineralization; trace iron staining; weathered.
20	Driven	34					
	Driven	23	24.2	94.7			Purplish gray; some iron staining.
30	Driven	34					Gray to dark gray; moist; black carbonaceous mineralization; little iron staining.
	Driven	86/11"					Light gray and brown; strongly indurated; trace black carbonaceous laminations.
40	Driven	52					

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BORING LOG

THE HUB FACILITY
WCR 6 AND WCR 7, WELD COUNTY, COLORADO

PROJECT NO.	DATE	FIGURE
500707001	10/13	A-5

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 9/05/13 BORING NO. B-4 GROUND ELEVATION 5,212' ± (MSL) SHEET 2 OF 2 METHOD OF DRILLING CME-75, 4" Diameter Solid-Stem Auger (Precision Drilling) DRIVE WEIGHT 140 lbs. (Auto Hammer) DROP 30" SAMPLED BY DLH LOGGED BY DLH REVIEWED BY JMJ **DESCRIPTION/INTERPRETATION**
40								Total Depth = 40 feet. Groundwater was measured at a depth of approximately 9.5 feet in borehole during drilling. Backfilled on 9/05/13 shortly after completion of drilling. <u>Note</u>: Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report.
50								
60								
70								
80								

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BORING LOG

THE HUB FACILITY
WCR 6 AND WCR 7, WELD COUNTY, COLORADO

PROJECT NO.	DATE	FIGURE
500707001	10/13	A-6

DATE DRILLED 9/06/13 BORING NO. B-5

GROUND ELEVATION 5,215' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING CME-75, 4" Diameter Solid-Stem Auger (Precision Drilling)

DRIVE WEIGHT 140 lbs. (Auto Hammer) DROP 30"

SAMPLED BY DLH LOGGED BY DLH REVIEWED BY JMJ

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								TOPSOIL: Approximately 6 inches thick.
	X						CL	ALLUVIUM: Brown, moist, stiff, sandy CLAY with trace calcium carbonate mineralization.
		X	13	20.0	101.5			
10		X	12	24.5	99.5			Wet.
							GC	Brown, saturated, very dense, clayey GRAVEL with sand. @11': Groundwater encountered during drilling.
		X	42	14.9				LARAMIE FORMATION: Gray and brown, moist to saturated, moderately indurated, CLAYSTONE interbedded with moderately to strongly cemented, silty fine-grained SANDSTONE; weathered.
20		X	35					Few black carbonaceous mineralization and iron staining.
								Total Depth = 20.5 feet. Groundwater was measured at a depth of approximately 11 feet in borehole during drilling. Backfilled on 9/06/13 shortly after completion of drilling. Note: Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report.
30								
40								

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BORING LOG

THE HUB FACILITY

WCR 6 AND WCR 7, WELD COUNTY, COLORADO

PROJECT NO.	DATE	FIGURE
500707001	10/13	A-7

DATE DRILLED 9/06/13		BORING NO. B-6	
GROUND ELEVATION 5,194' ± (MSL)		SHEET 1	OF 1
METHOD OF DRILLING CME-75, 4" Diameter Solid-Stem Auger (Precision Drilling)			
DRIVE WEIGHT 140 lbs. (Auto Hammer)		DROP 30"	
SAMPLED BY DLH	LOGGED BY DLH	REVIEWED BY JMJ	

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								TOPSOIL: Approximately 8 inches thick.
							CL	ALLUVIUM: Brown, moist to wet, stiff, sandy CLAY with clayey gravel interlayers.
			7					
10			12	20.3				@9.5': Groundwater encountered during drilling. Saturated.
			20	23.1				LARAMIE FORMATION: Gray, moist to saturated, moderately indurated, CLAYSTONE interbedded with moderately cemented, silty, fine-grained SANDSTONE with iron staining; weathered.
20			55					Dark gray and brownish gray; strongly indurated.
								Total Depth = 20.5 feet. Groundwater was measured at a depth of approximately 9.5 feet in borehole during drilling. Backfilled on 9/06/13 shortly after completion of drilling. Note: Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report.
30								
40								

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BORING LOG

THE HUB FACILITY
WCR 6 AND WCR 7, WELD COUNTY, COLORADO

PROJECT NO.	DATE	FIGURE
500707001	10/13	A-8

APPENDIX B

LABORATORY TESTING

Classification

Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

In-Place Moisture and Density Tests

The moisture content and dry density of relatively undisturbed samples obtained from the exploratory excavations were evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory excavations in Appendix A.

Atterberg Limits

Tests were performed on selected representative soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System. The test results are summarized on Figure B-1.

200 Wash

An evaluation of the percentage of particles finer than the No. 200 sieve in a selected soil sample was performed in general accordance with ASTM D 1140. The results of the tests are presented on Figure B-2.

Gradation Analysis

A Gradation analysis test was performed on a selected representative soil sample in general accordance with ASTM D 422. The grain-size distribution curve is shown on Figure B-3. The test results were utilized in evaluating the soil classifications in accordance with the Unified Soil Classification System.

Consolidation/Swell Tests

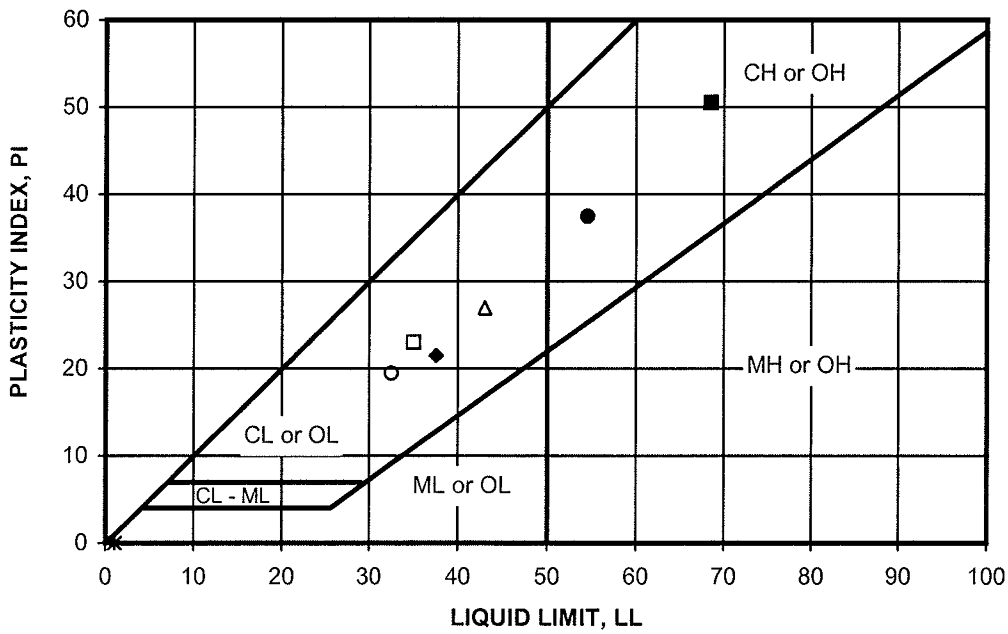
The consolidation and/or swell potential of selected materials were evaluated in general accordance with ASTM D 4546. Relatively undisturbed and remolded specimens were loaded with a specified surcharge before inundation with water. Readings of volumetric swell were recorded until completion of primary swell. After the completion of primary swell, surcharge loads were increased incrementally to evaluate swell pressure. The results of the swell tests performed on relatively undisturbed samples are presented on Figures B-4 through B-9.

Soil Corrosivity Tests

Soil pH and minimum resistivity tests were performed on a representative sample in general accordance with ASTM D 4972. The chloride content of a selected sample was evaluated in general accordance with CDOT Laboratory Procedure 2104. The sulfate content of a selected sample was evaluated in general accordance with CDOT Laboratory Procedure 2103. The test results are presented on Figure B-10.

SYMBOL	LOCATION	DEPTH (FT)	LIQUID LIMIT, LL	PLASTIC LIMIT, PL	PLASTICITY INDEX, PI	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS (Entire Sample)
●	B-1	4	55	17	38	CH	CH
■	B-1	14	69	18	51	CH	-
◆	B-2	4	38	16	22	CL	SC
○	B-2	9	33	13	20	CL	CL
□	B-3	9	35	12	23	CL	-
Δ	B-5	4	43	16	27	CL	CL

NP - INDICATES NON-PLASTIC



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

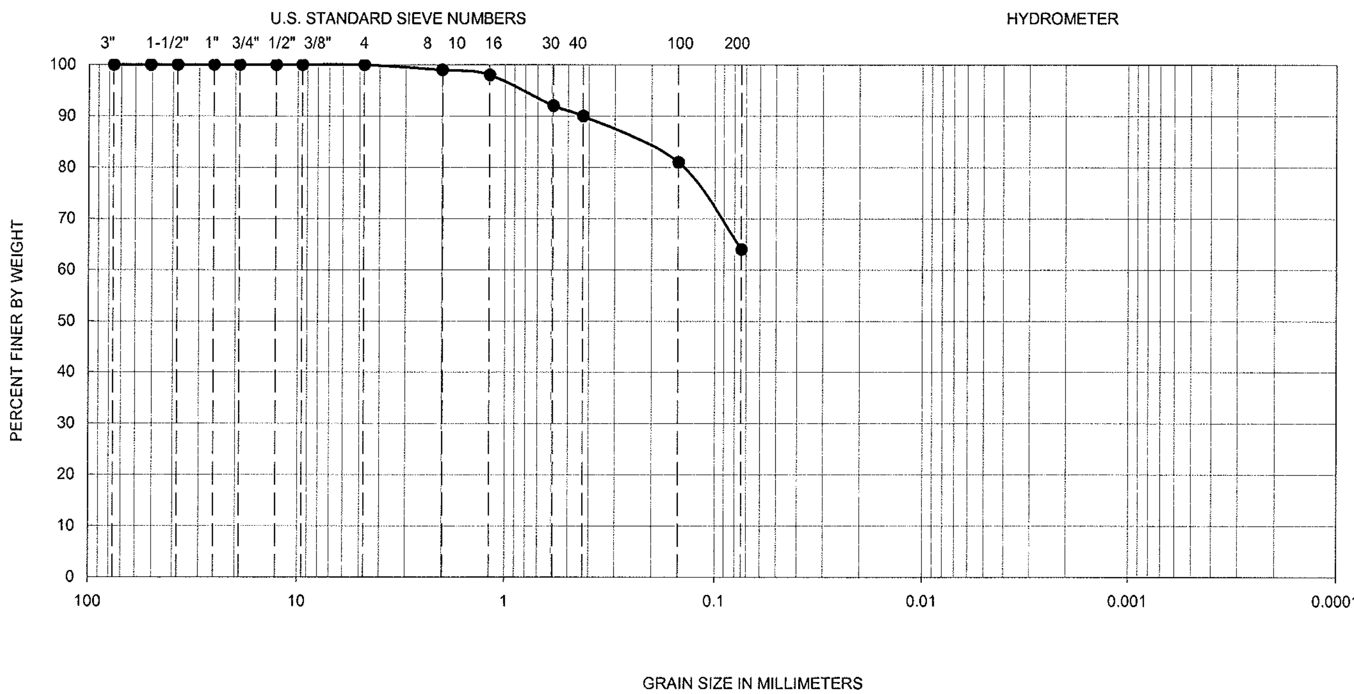
Ninyo & Moore	ATTERBERG LIMITS TEST RESULTS	FIGURE
PROJECT NO.: 500707001	THE HUB FACILITY WELD COUNTY ROAD 6 AND WELD COUNTY ROAD 7 WELD COUNTY, COLORADO	B-1
DATE: 10/13		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	EQUIVALENT USCS
B-1	4	Brown, sandy CLAY	100	73	CH
B-1	9	Brown, CLAYSTONE	100	97	CH
B-1	14	Brown, CLAYSTONE	100	72	CH
B-2	4	Reddish Brown, clayey SAND	100	46	SC
B-2	9	Reddish Brown, sandy CLAY	100	72	CL
B-3	9	Light Brown, CLAYSTONE	100	95	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

Ninyo & Moore		NO. 200 SIEVE ANALYSIS	FIGURE
PROJECT NO.	DATE	THE HUB FACILITY WELD COUNTY ROAD 6 AND WELD COUNTY ROAD 7 WELD COUNTY, COLORADO	B-2
500707001	10/13		

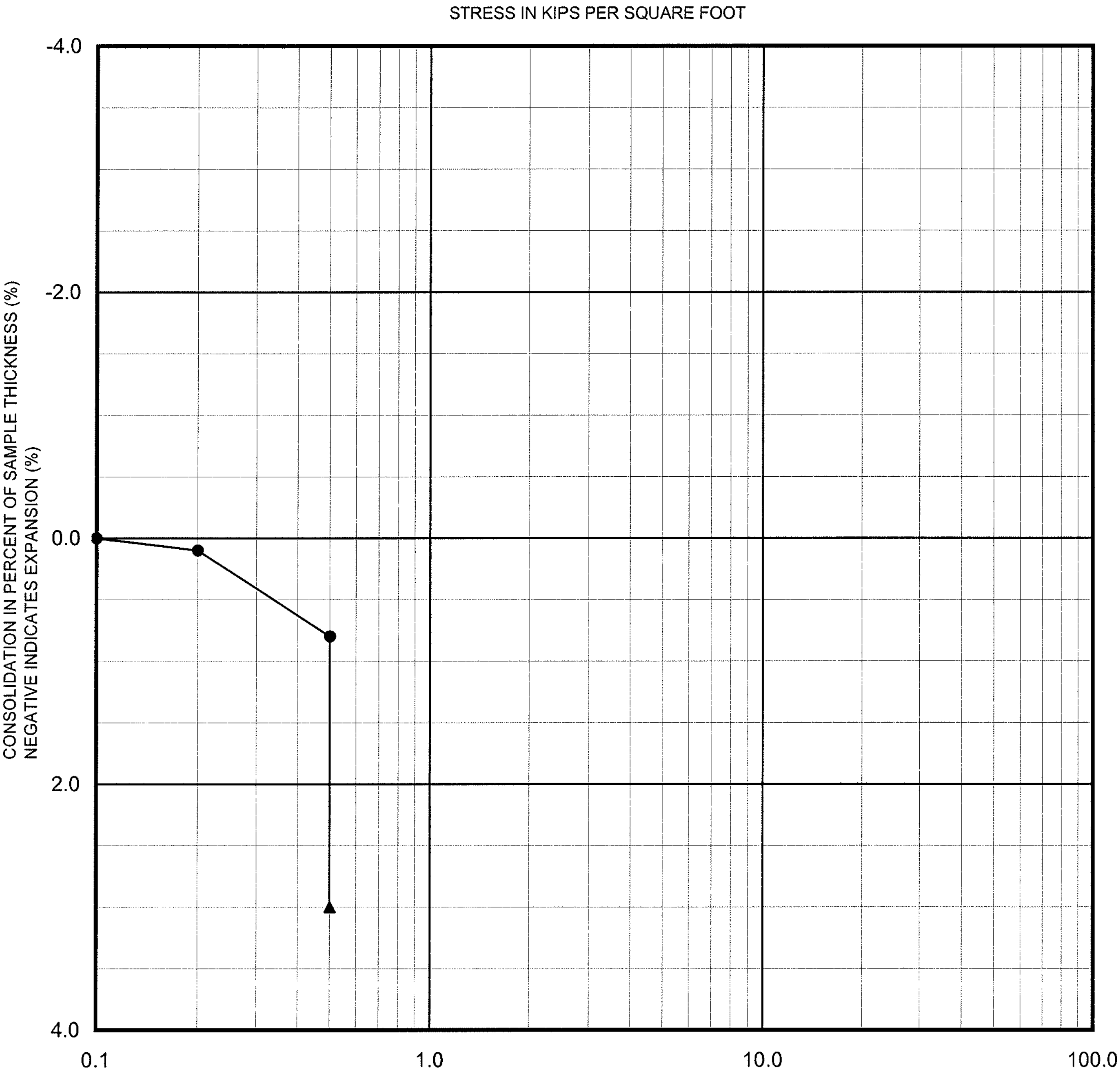
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	U.S.C.S
●	B-5	4	43	16	27	--	--	--	--	--	64	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore		GRADATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	THE HUB FACILITY WELD COUNTY ROAD 6 AND WELD COUNTY ROAD 7 WELD COUNTY, COLORADO	B-3
500707001	10/13		

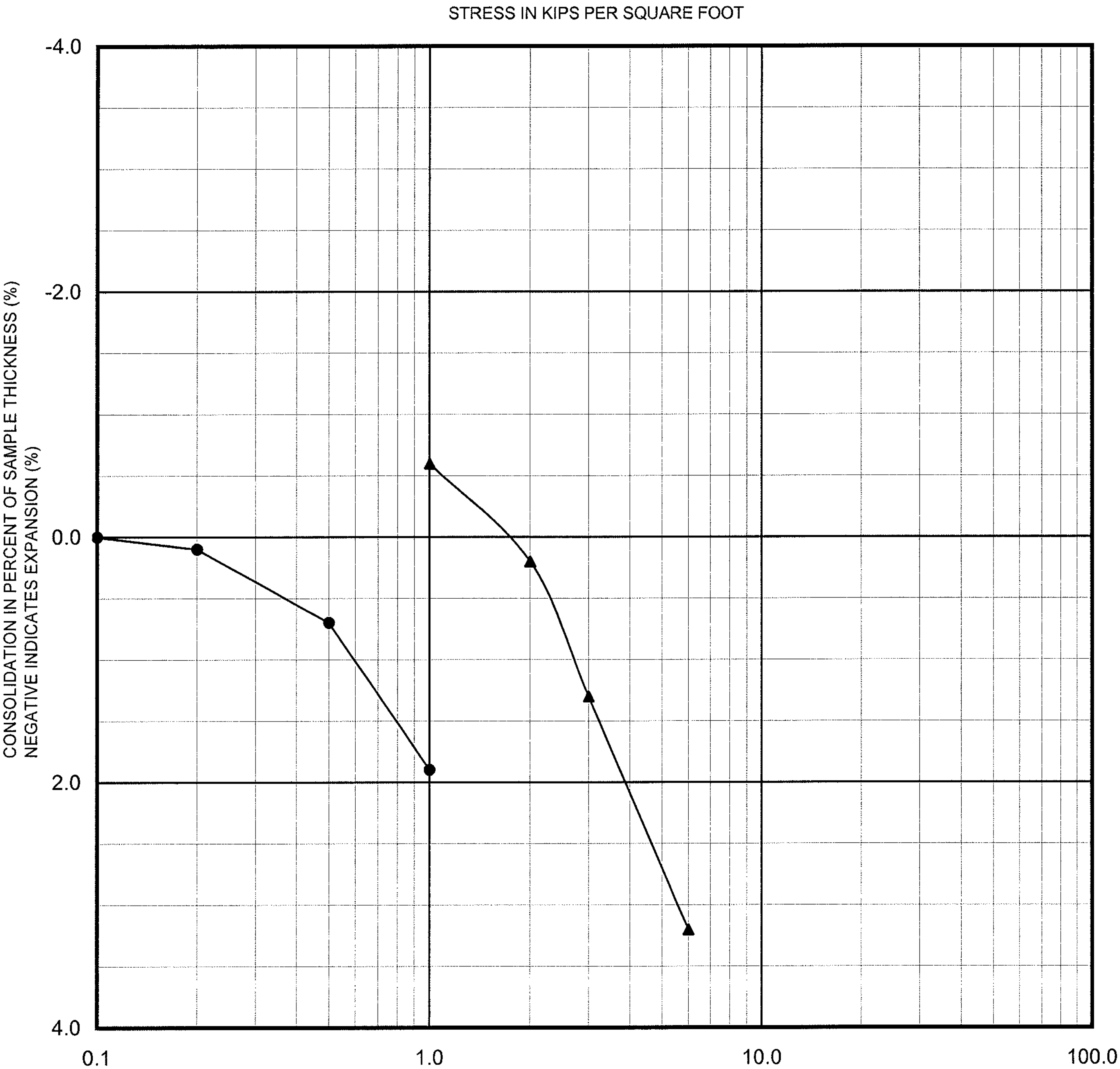


Seating Cycle		Sample Location	B-1
Loading Prior to Inundation		Depth (ft.)	4
Loading After Inundation		Soil Type	CH
Rebound Cycle			

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

Ninyo & Moore		CONSOLIDATION/SWELL TEST RESULTS	FIGURE
PROJECT NO.	DATE	THE HUB FACILITY WELD COUNTY ROAD 6 AND WELD COUNTY ROAD 7 WELD COUNTY, COLORADO	B-4
500707001	10/13		

FIG B-4 CONSOL B-1 @ 4.xls

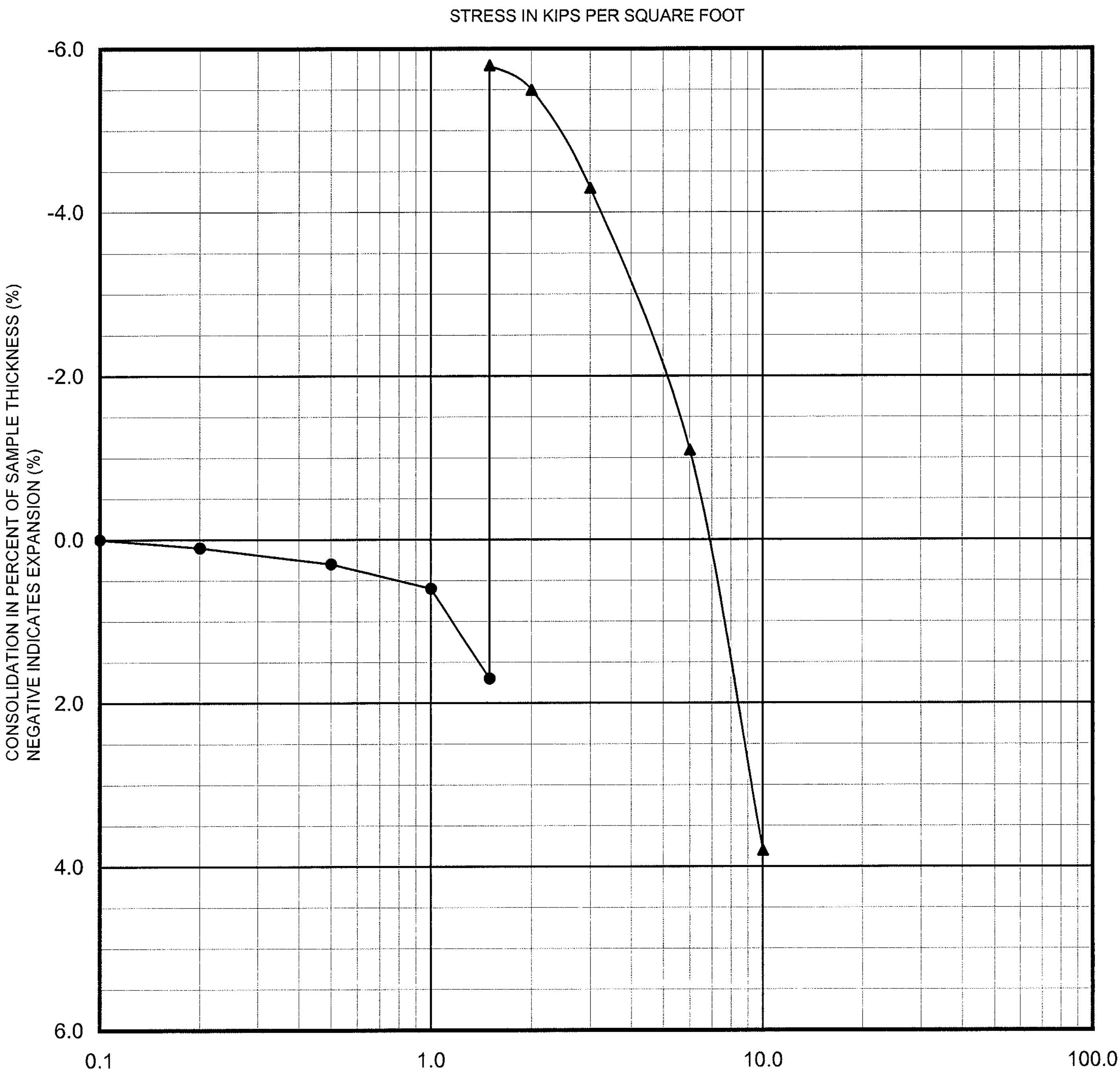


Seating Cycle		Sample Location	B-1
Loading Prior to Inundation		Depth (ft.)	9
Loading After Inundation		Soil Type	Laramie Formation
Rebound Cycle			

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

Ninyo & Moore		CONSOLIDATION/SWELL TEST RESULTS	FIGURE
PROJECT NO.	DATE	THE HUB FACILITY WELD COUNTY ROAD 6 AND WELD COUNTY ROAD 7 WELD COUNTY, COLORADO	B-5
500707001	10/13		

FIG B-5 CONSOL B-1 @ 9.xls



---●---	Seating Cycle	Sample Location	B-1
—●—	Loading Prior to Inundation	Depth (ft.)	14
—▲—	Loading After Inundation	Soil Type	Laramie Formation
--▲--	Rebound Cycle		

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

Ninyo & Moore

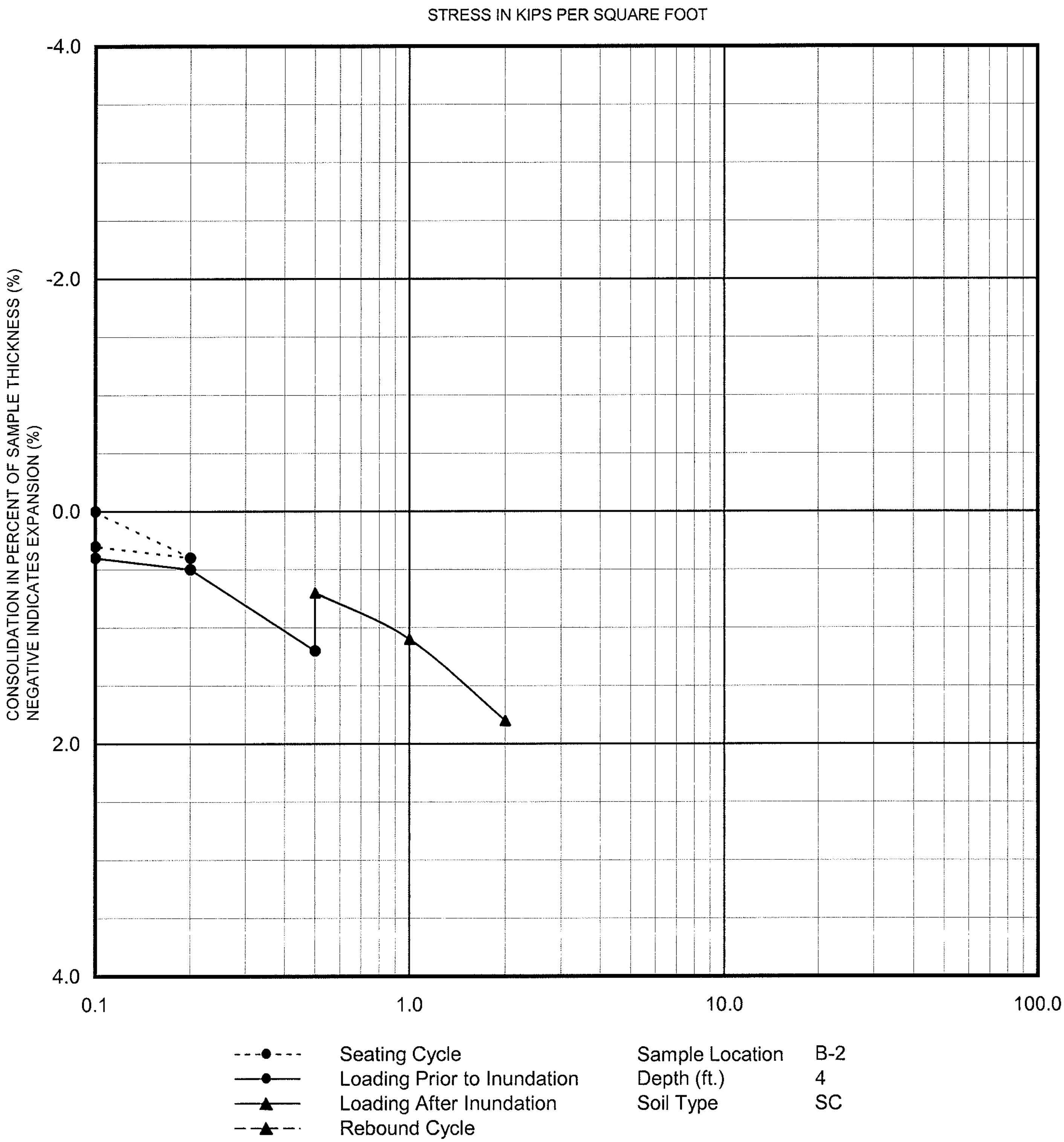
CONSOLIDATION/SWELL TEST RESULTS

FIGURE **B-6**

PROJECT NO.	DATE
500707001	10/13

THE HUB FACILITY
WELD COUNTY ROAD 6 AND WELD COUNTY ROAD 7
WELD COUNTY, COLORADO

FIG B-6 CONSOL B-1 @ 14.xls



Sample Location	B-2
Depth (ft.)	4
Soil Type	SC

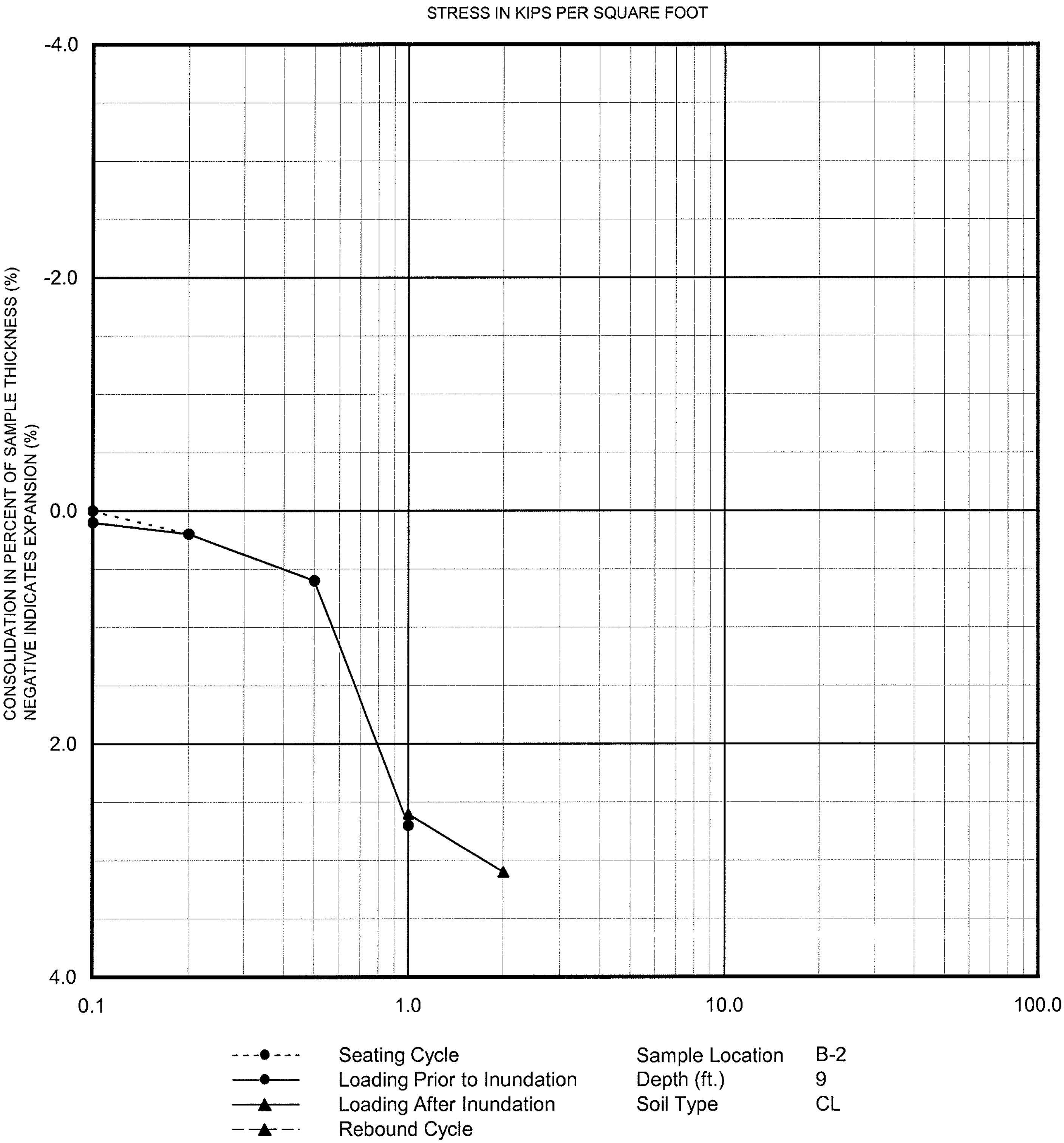
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

Ninyo & Moore

CONSOLIDATION/SWELL TEST RESULTS

PROJECT NO.	DATE	THE HUB FACILITY WELD COUNTY ROAD 6 AND WELD COUNTY ROAD 7 WELD COUNTY, COLORADO	FIGURE
500707001	10/13		B-7

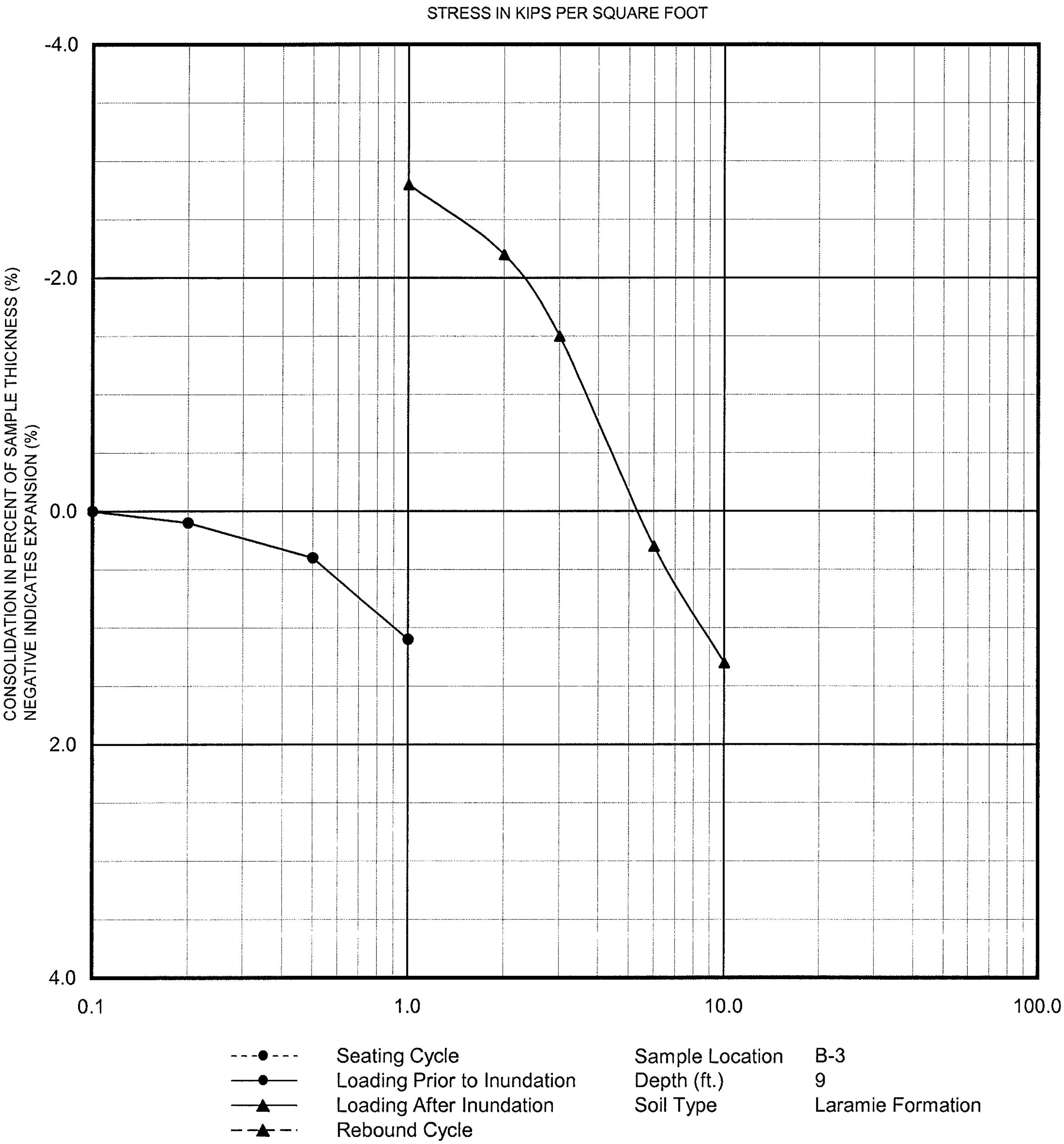
FIG B-7 CONSOL B-2 @ 4.xls



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

Ninyo & Moore		CONSOLIDATION/SWELL TEST RESULTS	FIGURE
PROJECT NO.	DATE	THE HUB FACILITY WELD COUNTY ROAD 6 AND WELD COUNTY ROAD 7 WELD COUNTY, COLORADO	B-8
500707001	10/13		

FIG B-8 CONSOL B-2 @ 9.xls



Sample Location	B-3
Depth (ft.)	9
Soil Type	Laramie Formation

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

Ninyo & Moore		CONSOLIDATION/SWELL TEST RESULTS	FIGURE
PROJECT NO.	DATE	THE HUB FACILITY WELD COUNTY ROAD 6 AND WELD COUNTY ROAD 7 WELD COUNTY, COLORADO	B-9
500707001	10/13		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH [1]	RESISTIVITY [2] (Ohm-cm)	SULFATE CONTENT [3]		CHLORIDE CONTENT [4] (ppm)
				(ppm)	(%)	
B-6	1-4	8.2	889	100	0.010	18.8

1 PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4972

2 PERFORMED IN GENERAL ACCORDANCE WITH AASHTO T288

3 PERFORMED IN GENERAL ACCORDANCE WITH CDOT TEST METHOD CP-L 2103

4 PERFORMED IN GENERAL ACCORDANCE WITH CDOT TEST METHOD CP-L 2104

Ninyo & Moore		CORROSIVITY TEST RESULTS	FIGURE
PROJECT NO.	DATE	WELD COUNTY HUB FACILITY WELD COUNTY ROAD 6 AND WELD COUNTY ROAD 7 ERIE, COLORADO	B-10
500707001	10/13		

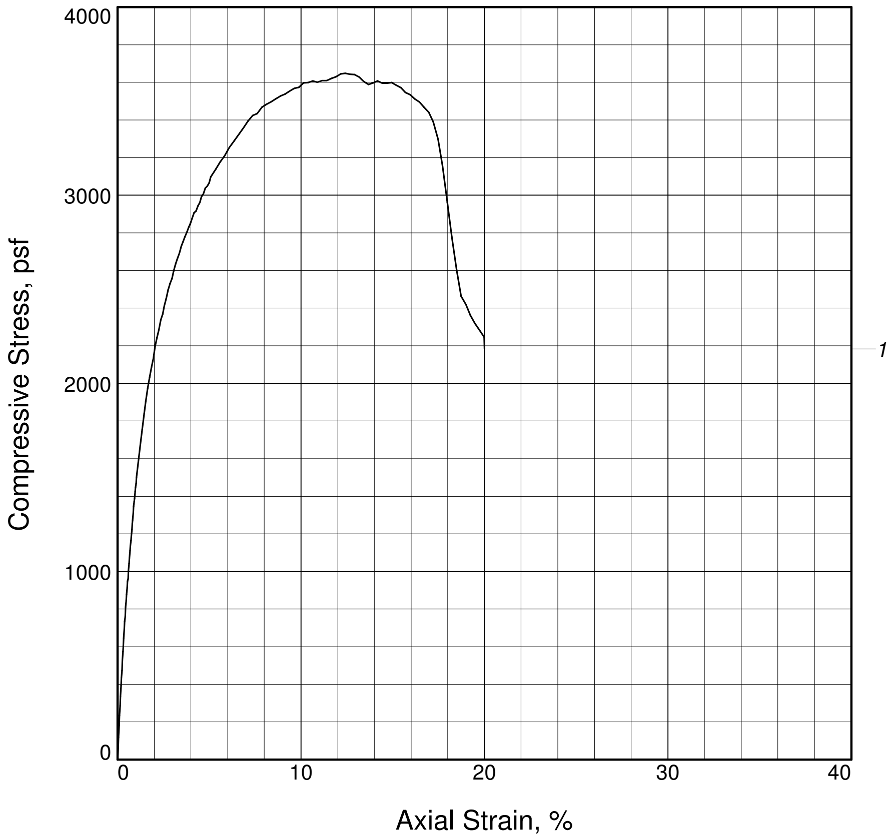
FIG B-10 CORROSIVITY.xls

APPENDIX C

UNCONFINED COMPRESSION TEST RESULTS

Cursory interpretations provided require review by a professional engineer. Knight Piesold accepts no responsibility in subsequent analyses.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	3648			
Undrained shear strength, psf	1824			
Failure strain, %	12.4			
Strain rate, %/min.	1.00			
Water content, %	17.9			
Wet density, pcf	130.7			
Dry density, pcf	110.8			
Saturation, %	92.8			
Void ratio	0.5213			
Specimen diameter, in.	2.42			
Specimen height, in.	4.98			
Height/diameter ratio	2.06			

Description:

LL =	**PL =**	**PI =**	**Assumed GS=** 2.7	**Type:** Undisturbed

Project No.: DV108-298-02-400

Date Sampled: 9/19/13

Remarks:

Figure _______

Client: Niyo & Moore

Project: Encana Hub Facility
N&M #500707001

Sample Number: B-4 **Depth:** 4'

Knight Piésold CONSULTING

Tested By: JHK **Checked By:** JDB

UNCONFINED COMPRESSION TEST

9/23/2013

Date: 9/19/13
Client: Niyo & Moore
Project: Encana Hub Facility
N&M #500707001
Project No.: DV108-298-02-400
Depth: 4' **Sample Number:** B-4
Description:
Remarks:
Type of Sample: Undisturbed
Assumed Specific Gravity=2.7 **LL=** **PL=** **PI=**

Parameters for Specimen No. 1

Specimen Parameter	Initial
Moisture content: Moist soil+tare, gms.	967.100
Moisture content: Dry soil+tare, gms.	847.900
Moisture content: Tare, gms.	182.850
Moisture, %	17.9
Moist specimen weight, gms.	786.9
Diameter, in.	2.42
Area, in.²	4.61
Height, in.	4.98
Wet density, pcf	130.7
Dry density, pcf	110.8
Void ratio	0.5213
Saturation, %	92.8

Test Readings for Specimen No. 1

Strain rate, %/min. = 1.00
Unconfined compressive strength = 3648 psf **at reading no.** 109

No.	Def. Dial in.	Load Dial	Load lbs.	Strain %	Deviator Stress psf
0	-1.8004	9.477	0.0	0.0	0
1	-1.7990	11.373	1.9	0.0	59
2	-1.7977	13.268	3.8	0.1	118
3	-1.7964	15.326	5.8	0.1	183
4	-1.7952	17.007	7.5	0.1	235
5	-1.7939	18.632	9.2	0.1	286
6	-1.7926	20.272	10.8	0.2	337
7	-1.7913	22.086	12.6	0.2	393
8	-1.7900	23.449	14.0	0.2	436
9	-1.7887	24.714	15.2	0.2	475
10	-1.7874	26.449	17.0	0.3	529
11	-1.7862	27.910	18.4	0.3	574
12	-1.7849	28.966	19.5	0.3	607
13	-1.7836	30.202	20.7	0.3	646
14	-1.7823	31.794	22.3	0.4	695
15	-1.7810	33.083	23.6	0.4	735
16	-1.7797	33.895	24.4	0.4	760

Test Readings for Specimen No. 1

No.	Def. Dial in.	Load Dial	Load lbs.	Strain %	Deviator Stress psf
17	-1.7784	35.515	26.0	0.4	810
18	-1.7772	36.652	27.2	0.5	845
19	-1.7759	37.694	28.2	0.5	878
20	-1.7746	38.823	29.3	0.5	912
21	-1.7733	39.979	30.5	0.5	948
22	-1.7721	40.370	30.9	0.6	960
23	-1.7708	41.868	32.4	0.6	1006
24	-1.7695	43.006	33.5	0.6	1041
25	-1.7682	43.980	34.5	0.6	1071
26	-1.7669	45.225	35.7	0.7	1110
27	-1.7656	46.106	36.6	0.7	1137
28	-1.7644	47.023	37.5	0.7	1165
29	-1.7631	47.760	38.3	0.7	1188
30	-1.7618	48.963	39.5	0.8	1225
31	-1.7605	49.949	40.5	0.8	1255
32	-1.7592	50.581	41.1	0.8	1274
33	-1.7579	51.797	42.3	0.9	1311
34	-1.7567	52.925	43.4	0.9	1346
35	-1.7554	53.782	44.3	0.9	1372
36	-1.7541	54.265	44.8	0.9	1387
37	-1.7528	55.491	46.0	1.0	1424
38	-1.7515	56.238	46.8	1.0	1447
39	-1.7503	56.889	47.4	1.0	1467
40	-1.7490	57.937	48.5	1.0	1499
41	-1.7439	60.822	51.3	1.1	1587
42	-1.7389	63.658	54.2	1.2	1673
43	-1.7338	66.408	56.9	1.3	1756
44	-1.7288	69.141	59.7	1.4	1838
45	-1.7237	71.366	61.9	1.5	1905
46	-1.7187	73.664	64.2	1.6	1973
47	-1.7137	75.646	66.2	1.7	2032
48	-1.7086	77.451	68.0	1.8	2085
49	-1.7036	79.068	69.6	1.9	2133
50	-1.6985	81.173	71.7	2.0	2195
51	-1.6935	82.790	73.3	2.1	2242
52	-1.6884	84.242	74.8	2.2	2284
53	-1.6834	86.042	76.6	2.3	2337
54	-1.6783	87.157	77.7	2.5	2368
55	-1.6733	88.703	79.2	2.6	2413
56	-1.6683	90.049	80.6	2.7	2451
57	-1.6632	91.663	82.2	2.8	2498
58	-1.6582	92.843	83.4	2.9	2531
59	-1.6531	93.782	84.3	3.0	2557
60	-1.6480	95.130	85.7	3.1	2595
61	-1.6430	96.494	87.0	3.2	2634
62	-1.6380	97.579	88.1	3.3	2664
63	-1.6329	98.595	89.1	3.4	2692

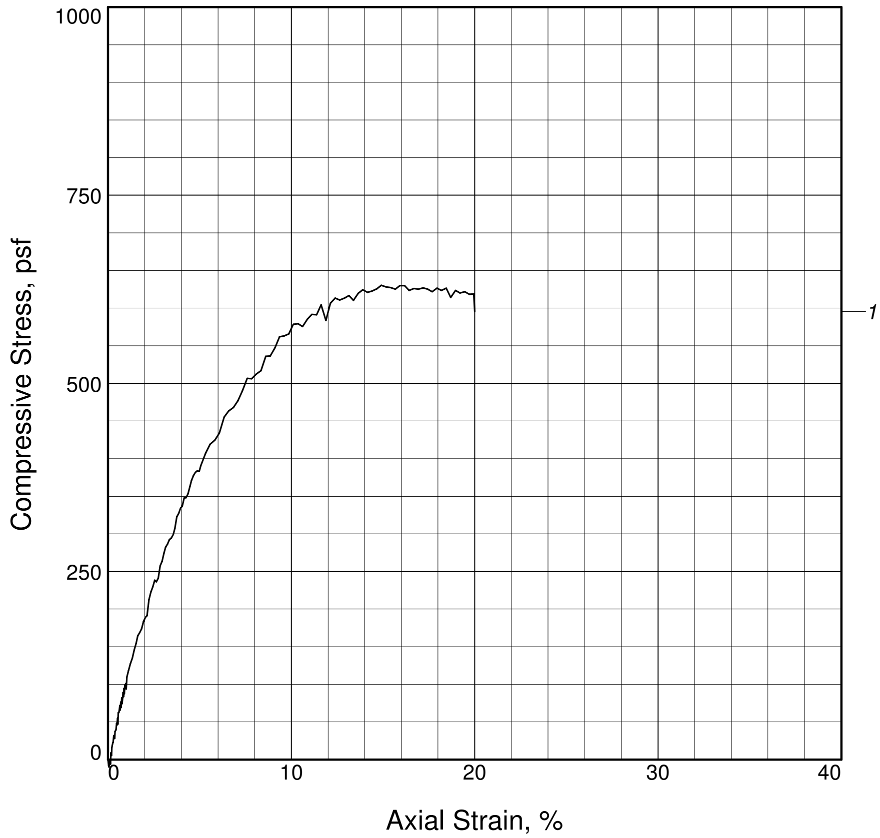
Test Readings for Specimen No. 1

No.	Def. Dial in.	Load Dial	Load lbs.	Strain %	Deviator Stress psf
64	-1.6279	99.895	90.4	3.5	2728
65	-1.6228	100.720	91.2	3.6	2750
66	-1.6178	101.743	92.3	3.7	2778
67	-1.6128	102.643	93.2	3.8	2802
68	-1.6078	103.632	94.2	3.9	2829
69	-1.6027	104.493	95.0	4.0	2852
70	-1.5976	105.468	96.0	4.1	2878
71	-1.5926	106.527	97.0	4.2	2907
72	-1.5875	106.906	97.4	4.3	2915
73	-1.5825	107.940	98.5	4.4	2943
74	-1.5775	108.635	99.2	4.5	2960
75	-1.5724	109.769	100.3	4.6	2991
76	-1.5674	110.415	100.9	4.7	3007
77	-1.5623	111.542	102.1	4.8	3038
78	-1.5573	112.018	102.5	4.9	3048
79	-1.5522	112.709	103.2	5.0	3066
80	-1.5471	113.858	104.4	5.1	3097
81	-1.5347	115.504	106.0	5.3	3137
82	-1.5220	117.095	107.6	5.6	3176
83	-1.5094	118.566	109.1	5.8	3210
84	-1.4969	120.313	110.8	6.1	3253
85	-1.4845	121.757	112.3	6.3	3287
86	-1.4718	123.240	113.8	6.6	3321
87	-1.4592	124.805	115.3	6.9	3358
88	-1.4465	126.340	116.9	7.1	3393
89	-1.4340	127.719	118.2	7.4	3424
90	-1.4214	128.420	118.9	7.6	3435
91	-1.4088	129.870	120.4	7.9	3467
92	-1.3962	130.736	121.3	8.1	3482
93	-1.3836	131.559	122.1	8.4	3496
94	-1.3711	132.445	123.0	8.6	3512
95	-1.3585	133.316	123.8	8.9	3527
96	-1.3459	134.055	124.6	9.1	3538
97	-1.3333	134.903	125.4	9.4	3553
98	-1.3207	135.778	126.3	9.6	3567
99	-1.3082	136.360	126.9	9.9	3574
100	-1.2956	137.539	128.1	10.1	3597
101	-1.2830	137.941	128.5	10.4	3598
102	-1.2704	138.624	129.1	10.6	3607
103	-1.2578	138.785	129.3	10.9	3601
104	-1.2452	139.417	129.9	11.1	3609
105	-1.2326	139.803	130.3	11.4	3609
106	-1.2200	140.584	131.1	11.7	3620
107	-1.2074	141.312	131.8	11.9	3630
108	-1.1949	142.208	132.7	12.2	3644
109	-1.1822	142.727	133.2	12.4	3648
110	-1.1696	142.947	133.5	12.7	3643

Test Readings for Specimen No. 1

No.	Def. Dial in.	Load Dial	Load lbs.	Strain %	Deviator Stress psf
111	-1.1574	143.263	133.8	12.9	3642
112	-1.1446	143.156	133.7	13.2	3628
113	-1.1320	142.624	133.1	13.4	3603
114	-1.1194	142.506	133.0	13.7	3589
115	-1.1068	143.171	133.7	13.9	3597
116	-1.0942	143.966	134.5	14.2	3607
117	-1.0816	143.928	134.5	14.4	3596
118	-1.0691	144.322	134.8	14.7	3596
119	-1.0565	144.829	135.4	14.9	3598
120	-1.0439	144.721	135.2	15.2	3585
121	-1.0313	144.627	135.1	15.4	3572
122	-1.0188	144.073	134.6	15.7	3547
123	-1.0061	143.981	134.5	16.0	3533
124	-0.9935	143.571	134.1	16.2	3512
125	-0.9809	143.323	133.8	16.5	3495
126	-0.9684	142.716	133.2	16.7	3469
127	-0.9558	142.056	132.6	17.0	3441
128	-0.9432	140.475	131.0	17.2	3390
129	-0.9306	137.302	127.8	17.5	3297
130	-0.9181	132.175	122.7	17.7	3155
131	-0.9054	124.978	115.5	18.0	2961
132	-0.8928	118.128	108.7	18.2	2777
133	-0.8802	111.616	102.1	18.5	2603
134	-0.8676	106.423	96.9	18.7	2463
135	-0.8551	105.033	95.6	19.0	2420
136	-0.8425	103.022	93.5	19.2	2361
137	-0.8302	101.619	92.1	19.5	2319
138	-0.8175	100.450	91.0	19.7	2282
139	-0.8049	99.309	89.8	20.0	2246
140	-0.8039	96.778	87.3	20.0	2183

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	630			
Undrained shear strength, psf	315			
Failure strain, %	14.9			
Strain rate, %/min.	1.00			
Water content, %	23.8			
Wet density, pcf	124.7			
Dry density, pcf	100.7			
Saturation, %	95.5			
Void ratio	0.6743			
Specimen diameter, in.	2.42			
Specimen height, in.	4.96			
Height/diameter ratio	2.05			

Description:

LL =	PL =	PI =	Assumed GS= 2.7	Type: Undisturbed

Project No.: DV108-298-02-400

Date Sampled: 9/19/13

Remarks:

Figure _______

Client: Niyo & Moore

Project: Encana Hub Facility
N&M #500707001

Sample Number: B-6 **Depth:** 4'

Knight Piésold CONSULTING

Tested By: JHK **Checked By:** JDB

UNCONFINED COMPRESSION TEST

9/23/2013

Date: 9/19/13
Client: Niyo & Moore
Project: Encana Hub Facility
N&M #500707001
Project No.: DV108-298-02-400
Depth: 4' **Sample Number:** B-6
Description:
Remarks:
Type of Sample: Undisturbed
Assumed Specific Gravity=2.7 **LL=** **PL=** **PI=**

Parameters for Specimen No. 1

Specimen Parameter	Initial
Moisture content: Moist soil+tare, gms.	928.700
Moisture content: Dry soil+tare, gms.	785.400
Moisture content: Tare, gms.	184.300
Moisture, %	23.8
Moist specimen weight, gms.	746.6
Diameter, in.	2.42
Area, in.²	4.60
Height, in.	4.96
Wet density, pcf	124.7
Dry density, pcf	100.7
Void ratio	0.6743
Saturation, %	95.5

Test Readings for Specimen No. 1

Strain rate, %/min. = 1.00
Unconfined compressive strength = 630 psf **at reading no.** 118

No.	Def. Dial in.	Load Dial	Load lbs.	Strain %	Deviator Stress psf
0	-1.7994	9.357	0.0	0.0	0
1	-1.7980	9.106	-0.3	0.0	-8
2	-1.7967	9.053	-0.3	0.1	-10
3	-1.7955	9.242	-0.1	0.1	-4
4	-1.7942	9.366	0.0	0.1	0
5	-1.7930	9.172	-0.2	0.1	-6
6	-1.7917	9.644	0.3	0.2	9
7	-1.7904	9.529	0.2	0.2	5
8	-1.7892	9.868	0.5	0.2	16
9	-1.7879	9.946	0.6	0.2	18
10	-1.7866	10.061	0.7	0.3	22
11	-1.7854	10.097	0.7	0.3	23
12	-1.7841	10.376	1.0	0.3	32
13	-1.7829	10.387	1.0	0.3	32
14	-1.7816	10.264	0.9	0.4	28
15	-1.7803	10.558	1.2	0.4	37
16	-1.7778	10.622	1.3	0.4	39

Test Readings for Specimen No. 1

No.	Def. Dial in.	Load Dial	Load lbs.	Strain %	Deviator Stress psf
17	-1.7765	10.916	1.6	0.5	49
18	-1.7753	10.826	1.5	0.5	46
19	-1.7740	11.135	1.8	0.5	55
20	-1.7727	10.871	1.5	0.5	47
21	-1.7715	11.368	2.0	0.6	63
22	-1.7702	11.372	2.0	0.6	63
23	-1.7689	11.447	2.1	0.6	65
24	-1.7677	11.673	2.3	0.6	72
25	-1.7664	11.480	2.1	0.7	66
26	-1.7651	11.815	2.5	0.7	76
27	-1.7639	11.581	2.2	0.7	69
28	-1.7626	12.000	2.6	0.7	82
29	-1.7613	11.756	2.4	0.8	75
30	-1.7601	12.020	2.7	0.8	83
31	-1.7588	12.238	2.9	0.8	89
32	-1.7575	12.052	2.7	0.8	84
33	-1.7563	12.409	3.1	0.9	95
34	-1.7550	12.180	2.8	0.9	88
35	-1.7538	12.554	3.2	0.9	99
36	-1.7525	12.428	3.1	0.9	95
37	-1.7512	12.624	3.3	1.0	101
38	-1.7500	12.391	3.0	1.0	94
39	-1.7487	12.892	3.5	1.0	110
40	-1.7437	13.181	3.8	1.1	118
41	-1.7386	13.487	4.1	1.2	128
42	-1.7336	13.714	4.4	1.3	135
43	-1.7286	14.082	4.7	1.4	146
44	-1.7236	14.379	5.0	1.5	155
45	-1.7186	14.700	5.3	1.6	165
46	-1.7136	14.836	5.5	1.7	169
47	-1.7086	15.015	5.7	1.8	174
48	-1.7036	15.342	6.0	1.9	184
49	-1.6986	15.532	6.2	2.0	189
50	-1.6936	15.595	6.2	2.1	191
51	-1.6886	16.298	6.9	2.2	212
52	-1.6835	16.639	7.3	2.3	223
53	-1.6785	16.873	7.5	2.4	230
54	-1.6735	17.178	7.8	2.5	239
55	-1.6684	17.109	7.8	2.6	236
56	-1.6634	17.275	7.9	2.7	241
57	-1.6584	17.826	8.5	2.8	258
58	-1.6534	18.037	8.7	2.9	264
59	-1.6484	18.397	9.0	3.0	274
60	-1.6434	18.670	9.3	3.1	282
61	-1.6384	18.817	9.5	3.2	287
62	-1.6333	19.014	9.7	3.3	292
63	-1.6283	19.104	9.7	3.4	295

Test Readings for Specimen No. 1

No.	Def. Dial in.	Load Dial	Load lbs.	Strain %	Deviator Stress psf
64	-1.6233	19.261	9.9	3.6	299
65	-1.6183	19.539	10.2	3.7	307
66	-1.6133	20.070	10.7	3.8	323
67	-1.6083	20.236	10.9	3.9	327
68	-1.6033	20.486	11.1	4.0	335
69	-1.5983	20.558	11.2	4.1	336
70	-1.5933	20.944	11.6	4.2	348
71	-1.5883	20.973	11.6	4.3	348
72	-1.5833	21.156	11.8	4.4	353
73	-1.5782	21.497	12.1	4.5	363
74	-1.5731	21.773	12.4	4.6	371
75	-1.5681	22.003	12.6	4.7	377
76	-1.5631	22.160	12.8	4.8	382
77	-1.5581	22.238	12.9	4.9	384
78	-1.5531	22.233	12.9	5.0	383
79	-1.5481	22.497	13.1	5.1	391
80	-1.5356	23.088	13.7	5.3	407
81	-1.5231	23.532	14.2	5.6	419
82	-1.5105	23.751	14.4	5.8	424
83	-1.4980	24.108	14.8	6.1	434
84	-1.4855	24.880	15.5	6.3	455
85	-1.4730	25.203	15.8	6.6	463
86	-1.4605	25.397	16.0	6.8	468
87	-1.4481	25.743	16.4	7.1	477
88	-1.4355	26.252	16.9	7.3	490
89	-1.4230	26.866	17.5	7.6	507
90	-1.4104	26.907	17.5	7.8	506
91	-1.3980	27.170	17.8	8.1	513
92	-1.3854	27.375	18.0	8.3	517
93	-1.3729	28.091	18.7	8.6	536
94	-1.3604	28.148	18.8	8.9	536
95	-1.3479	28.586	19.2	9.1	547
96	-1.3354	29.152	19.8	9.4	562
97	-1.3229	29.260	19.9	9.6	563
98	-1.3104	29.405	20.0	9.9	566
99	-1.2979	29.910	20.6	10.1	578
100	-1.2853	29.994	20.6	10.4	579
101	-1.2728	29.921	20.6	10.6	575
102	-1.2602	30.323	21.0	10.9	585
103	-1.2477	30.619	21.3	11.1	592
104	-1.2352	30.671	21.3	11.4	591
105	-1.2227	31.195	21.8	11.6	604
106	-1.2102	30.515	21.2	11.9	584
107	-1.1977	31.408	22.1	12.1	607
108	-1.1852	31.717	22.4	12.4	613
109	-1.1727	31.690	22.3	12.6	611
110	-1.1601	31.839	22.5	12.9	613

Test Readings for Specimen No. 1

No.	Def. Dial in.	Load Dial	Load lbs.	Strain %	Deviator Stress psf
111	-1.1476	32.027	22.7	13.1	616
112	-1.1350	31.861	22.5	13.4	610
113	-1.1225	32.274	22.9	13.6	620
114	-1.1100	32.520	23.2	13.9	624
115	-1.0975	32.463	23.1	14.2	621
116	-1.0850	32.597	23.2	14.4	623
117	-1.0724	32.779	23.4	14.7	626
118	-1.0599	33.018	23.7	14.9	630
119	-1.0474	33.010	23.7	15.2	628
120	-1.0348	33.048	23.7	15.4	627
121	-1.0223	33.029	23.7	15.7	625
122	-1.0098	33.278	23.9	15.9	630
123	-0.9972	33.356	24.0	16.2	630
124	-0.9847	33.181	23.8	16.4	623
125	-0.9722	33.352	24.0	16.7	626
126	-0.9597	33.389	24.0	16.9	625
127	-0.9471	33.541	24.2	17.2	627
128	-0.9348	33.548	24.2	17.4	625
129	-0.9222	33.490	24.1	17.7	622
130	-0.9095	33.749	24.4	17.9	627
131	-0.8970	33.694	24.3	18.2	623
132	-0.8845	33.894	24.5	18.4	626
133	-0.8720	33.493	24.1	18.7	614
134	-0.8595	33.924	24.6	19.0	623
135	-0.8470	33.873	24.5	19.2	620
136	-0.8345	34.018	24.7	19.5	622
137	-0.8220	33.951	24.6	19.7	618
138	-0.8095	34.046	24.7	20.0	619
139	-0.8074	33.136	23.8	20.0	596