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GEOTECHNICAL ENGINEERING STUDY AND PAVEMENT THICKNESS DESIGN PROPOSED MISTER CAR WASH FACILITY 13225 PARKER ROAD PARKER, COLORADO

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SUMMARY

1. Pre-existing fill was encountered to depths ranging from about 1 to 3 feet underlain by natural granular soils extending to clay soils at an approximate depth of 14 feet in Borings 1 and 3. The natural soils were underlain by claystone bedrock at depths ranging from 19 to 20 feet in Borings 1 through 3, extending to the maximum depth explored of about 25 to 30 feet below the ground surface. Boring 4 consisted of about 1 foot of pre-existing fill underlain by relatively shallow bedrock, extending to the maximum drilled depths of about 20 feet.

   Groundwater was not encountered in any of the exploratory borings during drilling or when stabilized groundwater measurements were made 5 days subsequent to drilling. The borings were backfilled following stabilized groundwater measurements.

2. Based on the conditions encountered in our borings, a shallow foundation system is considered feasible for the car wash when supported on a prepared subgrade. Footings placed on a prepared subgrade as described herein should be designed for a net allowable bearing pressure of 3,000 psf.

3. Slab-on-grade floor slabs are considered feasible when supported on a prepared subgrade similar to subgrade preparations for foundations.

4. Pavement section alternatives based on the on-site material properties and local industry standard of practice are presented below:

LOCATION	Full Depth Asphalt Pavement (inches)	Asphalt Over Aggregate Base Course (inches)	PCCP (inches)
Standard Duty	5.5	4.0 over 6.0	6.0
Heavy Duty	6.0	4.5 over 6.0	7.0

PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical engineering study and pavement thickness design for the proposed Mister Car Wash Facility to be constructed at 13225 Parker Road in Parker, Colorado. The project site is shown on Fig. 1. The geotechnical study was performed in general accordance with the scope of work presented in our Proposal No. P-23-513 to Mister Car Wash, dated June 13, 2023.

A field exploration program consisting of drilling exploratory borings and performing field percolation tests was conducted to obtain information on subsurface conditions. Representative samples of the soils and bedrock materials obtained during the field exploration program were tested in the laboratory to determine their classification and engineering characteristics. The results of the field exploration and laboratory testing programs were analyzed to develop geotechnical engineering recommendations for design and construction of the proposed facility.

This report has been prepared to summarize the data obtained during this study and to present our conclusions and recommendations based on the proposed construction and the subsurface conditions encountered. Design parameters and a discussion of geotechnical engineering considerations related to construction of the proposed facility are included in the report.

PROPOSED CONSTRUCTION

We understand the car wash structure will consist of a single-story, at-grade building consisting of a 130-foot-long tunnel wash. Construction will also include 3 queuing lanes, an ADA stall, and 18 vacuum stations. Site pavements will include the entry/exit driveways, the service detailing station area, drive lanes, and associated parking stalls within the site.

We assume the building will be a combination of masonry structure and steel frame construction. We anticipate reinforced concrete foundations and stem walls will be used. Floor support will consist of a slab-on-grade floor system supported on a prepared subgrade. Foundation loads are expected to be relatively light, typical of this type of construction. Based on the current development at the site, we anticipate site grading will be minimal, with cuts and fills on the order of a few feet to establish level grades.

If the proposed construction varies significantly from that described above or depicted in this report, we should be notified to reevaluate the recommendations provided in this report.

SITE CONDITIONS

At the time of our field exploration program, the site was vacant. The site is located on Lot 3A in the Parker Point Subdivision Filing No. 2 and is bounded on the north by Stroh Road, the east by South Parker Road, and the west and south by vacant land. The topography at the site was nearly level as previous site grading for the entire development has taken place.

Based on available historical imagery, the site was previously occupied by a single-family residential structure with several ancillary out structures. Prior to our field exploration program, the structures had been razed and the site had been regraded in preparation for development. It appears the majority of the site grading consisted of cuts on the order of about 10 to 15 feet.

SUBSURFACE CONDITIONS

Field Exploration: The subsurface conditions were explored by drilling six (6) exploratory borings and two (2) percolation holes at the approximate locations shown on Fig. 1. The borings were advanced through the pre-existing fill and natural soils into the underlying bedrock, where encountered, using 4-inch-diameter, continuous-flight augers. The borings were logged by a representative of K+A. Samples of the soils and bedrock materials were obtained with a 2-inch-I.D. California-liner sampler driven into the various strata with blows from a 140-pound hammer falling 30 inches. The California-liner sampling procedure is similar to the standard penetration test (SPT) described by ASTM Method D1586. Sampler penetration resistance values (blow counts), when properly evaluated, indicate the relative density or consistency of the materials encountered.

The depths at which samples were taken and the associated blow counts are shown on the boring logs presented on Fig. 2, with a legend and associated explanatory notes presented on Fig. 3.

Subsurface Conditions: Pre-existing fill was encountered to depths ranging from about 1 to 3 feet and was underlain by natural granular soils extending to clay soils at an approximate depth of 14 feet in Borings 1 and 3. The natural soils were underlain by claystone bedrock at depths ranging from 19 to 20 feet in Borings 1 through 3, extending to the maximum depths explored of about 25 to 30 feet below the ground surface. Boring 4 consisted of about 1 foot of pre-existing fill underlain by claystone bedrock, extending the maximum drilled depth of about 20 feet.

The pre-existing fill generally consisted of sandy lean clay to clayey sand with fine- to coarse-grained sand content, that was moist and dark brown to brown to tan. The lateral and vertical limits and degree of compaction of the fill were not determined as part of this study.

The natural soils consisted of a mix of granular and cohesive soils. The granular soils consisted of fine- to medium-grained silty sand and clayey sand that was moist and brown to tan. The cohesive soils consisted of sandy lean clay and was moist and brown. Based on blow counts, the granular soils were generally medium dense, and the cohesive soils were very stiff.

The claystone bedrock was moist, and dark brown to brown to dark gray to gray. The bedrock was medium hard to hard, with a firm zone found in Boring 4 in the first 10 feet below the ground surface.

Groundwater Conditions: Groundwater was not encountered in any of the exploratory borings during drilling or when stabilized groundwater measurements were made 5 days subsequent to drilling. The borings were backfilled following stabilized groundwater measurements.

LABORATORY TESTING

Samples obtained from the exploratory borings were visually classified in the laboratory by the project engineer. Laboratory testing was performed on representative samples to evaluate in-situ moisture content and dry unit weight, liquid and plastic limits, gradation, and concentration of water-soluble sulfates. These tests were performed in accordance with applicable ASTM and Colorado Department of Transportation (CDOT) test procedures. The results of the laboratory tests are shown to the right of the logs on Fig. 2, plotted graphically on Figs. 4 and 5, and summarized in Table I.

Index Properties: Samples were classified into categories of similar engineering properties in general accordance with the Unified Soil Classification System. This system is based on index properties, including liquid limit, plasticity index, and grain size distribution. Values for in-situ moisture content and dry unit weight, liquid limit, plasticity index, and the percent of soil passing the U.S. No. 200 sieve are presented in Table I and adjacent to the corresponding sample on the boring logs.

Swell-Consolidation: Swell-consolidation tests were conducted on a sample of the clayey pre-existing fill as well as a sample of the claystone bedrock to determine the swell and/or compressibility potential under loading and when submerged in water. The samples were prepared and placed in a confining ring between porous discs, subjected to a surcharge pressure of 200 or 1,000 psf, and allowed to consolidate before being submerged in water. The samples were then inundated with water, and the change in sample height when deformation ceased was measured with a dial gauge. The samples were then loaded incrementally to maximum surcharge pressure of 3,000 to 20,000 psf, and the sample height was monitored until deformation practically ceased under each load increment.

Results of the swell-consolidation tests conducted on the relatively undisturbed drive samples are presented on Fig. 4 as plots of the curve of the final strain at each increment of pressure against the log of the pressure. Based on the results of the swell-consolidation test, the sample of pre-existing fill exhibited low swell potential and the sample of claystone bedrock exhibited moderate swell potential under the applied surcharge pressure when wetted

GEOTECHNICAL ENGINEERING CONSIDERATIONS

Based on conditions encountered in the borings, the site is overlain by up to 3 feet of pre-existing fill materials in places. Deeper fills may be present in other portions of the site and should be anticipated. Without documentation of placement conditions including density testing documenting the degree of compaction, the existing fill materials should generally be considered non-engineered and generally not suitable in their current condition for support of foundations or floor slabs.

The natural overburden soils and pre-existing fill materials are generally suitable for reuse as structural fill, provided they meet the material and placement criteria presented in the “Site Grading and Earthwork” section of this report. The geotechnical engineer of record should evaluate onsite and/or import fill material prior to placement.

Shallow foundations consisting of spread footings and slabs-on-grade are considered feasible for the proposed construction when placed on a prepared subgrade. Subgrade preparation should consist of removal of all pre-existing fill materials down to natural soils beneath building areas.

FOUNDATION RECOMMENDATIONS

Considering the subsurface conditions encountered in the exploratory borings and the nature of the proposed construction, we recommend the pre-existing fills be completely removed and the proposed building be founded on spread footings placed on natural granular soils or structural fill extending to natural granular soils.

The design and construction criteria presented below should be observed for a spread footing foundation system. The construction details should be considered when preparing project documents.

1. Footings placed as described above should be designed for a net allowable bearing pressure of 3,000 psf. The net allowable bearing capacity may be increased by one-third for transient loads (wind and seismic).

2. Structural fill should extend down and out from the edges of the footings at a 1 (horizontal) to 1 (vertical) projection. Structural fill should be prepared and placed as recommended in the "Site Grading and Earthwork" section of this report. Prior to placing structural fill, the exposed subgrade should be scarified to a minimum depth of 12 inches, moisture-conditioned and recompacted similarly to structural fill as discussed herein.

3. Based on experience, we estimate total settlement for spread footings designed and constructed as discussed herein will be approximately one inch or less for footings prepared in accordance with the previous recommendations. Differential settlements across the structure are estimated to be approximately one-half of the total settlements.

4. Spread footings should have a minimum width of 16 inches for continuous footings and 24 inches for isolated pads.

5. Exterior footings and footings beneath unheated areas should be provided with adequate soil cover above their bearing elevation for frost protection. Placement of foundations at least 36 inches below the exterior grade is typically used in this area.

6. The lateral resistance of a spread footing will be a combination of the sliding resistance of the footing on the foundation bearing materials and passive earth pressure against the side of the footing. Resistance to sliding at the bottoms of the footings can be calculated based on a coefficient of friction of 0.35. Passive pressure against the sides of the footings may be calculated assuming an equivalent fluid unit weight of 205 pcf. The above values are working values.

7. Continuous foundation walls should be reinforced top and bottom to span an unsupported length of at least 10 feet.

8. A representative of the geotechnical engineer should observe all footing excavations prior to placement of structural fill or formwork.

FLOOR SUPPORT

Considering the subsurface conditions encountered in the exploratory borings and the nature of the proposed construction, soil-supported slabs are considered suitable when placed on a properly prepared subgrade as previously discussed in the "Geotechnical Engineering Considerations" section of this report.

For slab-on-grade floors, the following measures should be taken to reduce damage which could result from movement should the under-slab materials be subjected to changes in moisture content.

1. Floor slabs should be supported on natural granular soil or structural fill extending to undisturbed natural granular soils. Prior to placing structural fill or concrete formwork, the exposed subgrade should be scarified to a minimum depth of 12 inches and processed in place in accordance with the structural fill placement requirements provided in the "Site Grading and Earthwork" section of this report. Soft or excessively loose soils, non-engineered fill, or other deleterious materials encountered at the base of excavation should be removed to firm, suitable soils prior to placement of structural fill.

2. Floor slabs should be separated from all bearing walls and columns with expansion joints which allow unrestrained vertical movement.

3. Interior nonbearing partitions resting on floor slabs should be provided with slip joints at the bottoms so, if the slabs move, the movement cannot be transmitted to the upper structure. This detail is also important for wallboards, stairways and door frames. Slip joints which will allow at least 2 inches of vertical movement are recommended.

   If wood or metal stud partition walls are used, the slip joints should preferably be placed at the bottoms of the walls so differential slab movement will not damage the partition wall. If slab bearing masonry block partitions are constructed, the slip joints will have to be placed at the tops of the walls. If slip joints are provided at the tops of walls and the floors move, it is likely the partition walls will show signs of distress, such as cracking. An alternative, if masonry block walls or other walls without slip joints at the bottoms are required, is to found them on grade beams and piers and to construct the slabs independently of the foundation. If slab bearing partition walls are required, distress may be reduced by connecting the partition walls to the exterior walls using slip channels.

4. Floor slabs should not extend beneath exterior doors or over foundation stem walls, unless saw cut at the beam after construction.

5. Floor slab control joints should be used to reduce damage due to shrinkage cracking. Joint spacing is dependent on slab thickness, concrete aggregate size, and slump, and should be consistent with recognized guidelines such as those of the Portland Cement Association (PCA) or American Concrete Institute (ACI). We suggest joints be provided on the order of 12 to 15 feet apart in both directions. The requirements for slab reinforcement should be established by the designer based on experience and the intended slab use.

6. All fill materials for support of floor slabs should be placed and compacted according to the criteria presented in "Site Grading." The suitability of the on-site soils for use as under-slab fill is also discussed in "Site Grading."

7. All plumbing lines should be tested before operation. Where plumbing lines enter through the floor, a positive bond break should be provided. Flexible connections should be provided for slab-bearing mechanical equipment.

The precautions and recommendations itemized above will not prevent the movement of floor slabs if the underlying materials are subjected to moisture increases. However, the precautions should reduce the damage if such movement occurs.

EXTERIOR FLATWORK

Subgrade preparation for exterior flatwork considered movement-sensitive should be done in accordance with the “Floor Slabs” section of this report. Subgrade preparation for exterior flatwork that can tolerate some degree of movement should be done in accordance with the subgrade preparation recommendations for flexible pavements presented in the "Pavement Design" section of this report.

It is extremely important that exterior flatwork and pavements be isolated from the building foundations. Many problems associated with expansive/collapsible soils are related to ineffective isolation between pavements and exterior slabs and foundation-supported components of structures.

Movement of exterior flatwork adjacent to the building may result in adverse drainage conditions with runoff directed toward the building. Additionally, upward movement of exterior flatwork may restrict movement of outward swinging doors. Site grading and drainage design should consider those possibilities, particularly at entryways.

SITE SEISMIC CRITERIA

The Colorado Front Range is located in a low seismic activity area. The soil profile will generally consist of relatively medium-dense granular fill and natural granular soils extending to bedrock at depths ranging from about 19 to 20 feet. In accordance with the International Building Code (IBC), the overburden consisting of new fill and/or existing granular soils classifies as IBC Site Class D and the bedrock classifies as IBC Site Class C. The weighted average indicates an IBC design Site Class D. Based on the subsurface profile and site seismicity, liquefaction is not a design consideration.

INFILTRATION RATES

Percolation tests were performed by drilling two (2) borings in the middle of the site to a depth of about 5 feet. The approximate percolation test locations are shown on Fig. 1.

Based on the subsurface conditions encountered in the percolation borings and other nearby borings, the material at the infiltration test elevation likely consists of granular soils in accordance with the USDA soil classification system. Based on our experience, the natural granular soils are considered high to very high permeable soil, and normally will classify between Hydrologic Soil Group (HSG) HSG-A and HSG-B.

The corresponding infiltration rates calculated using procedures from the Michigan LID Manual and the corresponding HSG per USDA classification are summarized in the following table.

Test Hole	Depth of Percolation Test Hole (ft.)	Calculated Infiltration Rate (cm /sec)	Calculated Infiltration Rate (in /hr.)	Hydrologic Soil Group Classification (HSG)
Perc 1	5.19	5.83E-02	82.6	A
Perc 2	5.22	8.67E-02	122.8	A

Considerations: Based on the results of the profile borings, percolation testing, and laboratory testing, the granular soils generally classify as HSG-A soils.

The calculated infiltration rates provided above can be expected to diminish over time as a result of contamination by fine particles and organic material. The design team should consider the diminishment of the infiltration rate in the final design.

Claystone bedrock was found near the surface of the site in Boring 4 and is known to have very low infiltration causing a high runoff potential. Accordingly, the percolation test results should be used for natural granular materials. Additional tests should be performed in the claystone if that information is needed.

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SURFACE DRAINAGE

Proper surface drainage is very important for acceptable performance of the facility during construction and after construction has been completed. Drainage recommendations provided by local, state, and national entities should be followed based on the intended use of the facility. The following recommendations should be used as guidelines and changes should be made only after consultation with the geotechnical engineer.

1. Excessive wetting or drying of foundation and slab subgrades should be avoided during and following construction.

2. The ground surface surrounding the exterior of the buildings and exterior flatwork and paved areas should be sloped to drain away in all directions. We recommend a minimum slope of 6 inches in the first 10 feet in unpaved areas and a minimum slope of 3 inches in the first 10 feet in impervious flatwork and paved areas. Site drainage beyond the 10-foot zone should be designed to promote runoff and reduce infiltration. These slopes may be changed as required for handicap access points in accordance with the Americans with Disabilities Act.

3. To promote runoff, the upper 2 feet of the backfill adjacent to building should be a relatively impervious on-site soil or be covered by impervious flatwork or a pavement structure.

4. Exterior backfill should be adjusted to within 2 percentage points of optimum moisture content for granular materials, and between optimum and 3 percentage points above optimum for clay materials, placed in maximum 8-inch loose lifts, and compacted to at least 95% of the standard Proctor (ASTM D698) maximum dry density.

5. Care should be taken when compacting around the foundation walls and underground structures to avoid damage to the structure. Hand compaction procedures, if necessary, should be used to prevent lateral pressures from exceeding the design values.

6. Ponding of water should not be allowed in backfill material or in a zone within 10 feet of the building foundations, whichever is greater.

7. Roof downspouts and drains should discharge well beyond the limits of all backfill.

8. Excessive landscape irrigation should be avoided within 10 feet of the foundation walls.

WATER-SOLUBLE SULFATES

The concentration of water-soluble sulfates measured in representative samples of pre-existing fill and natural granular soils was 0.02% and 0.07%, respectively. These values represent a Class S0 exposure to sulfate attack on concrete when exposed to these materials. The degree of attack is based on a range of Class S0 (not applicable), Class S1 (moderate), Class S2 (severe), and Class S3 (very severe) severity of exposure as presented in ACI 201.2R-16.

Based on the laboratory test results, we believe special sulfate-resistant concrete will not be required.

SITE GRADING AND EARTHWORK

Existing Fill: As indicated, our borings encountered existing pre-existing fill within the upper few feet of the subsurface profile, which may be deeper in places. As mentioned in the “Geotechnical Engineering Considerations” section, the fill should be considered non-engineered and is not suitable for support of foundation elements and soil-supported slabs due to potential for unpredictable movement. All existing fill encountered in the building footprint areas should be removed, and the material replaced according to the moisture and density specifications provided below.

The on-site overburden soils and existing fill materials should be suitable for reuse as structural fill, provided they do not contain organic material, trash, debris, or other deleterious material, and they meet the criteria presented in this section.

Temporary Excavations: Temporary excavations should be constructed in accordance with OSHA requirements, as well as state, local and other applicable requirements. Site excavations will generally encounter pre-existing fill materials and natural granular soils classifying as OSHA Type C soils. Excavations encountering loose granular soils may require much shallower side slopes than those allowed by OSHA and/or the use of temporary shoring.

Excavated slopes may soften or loosen due to construction traffic and erode from surface runoff. Measures to keep surface runoff from excavation slopes, including diversion berms, should be considered.

Material Specifications: Unless specifically modified in the preceding sections of this report, the following recommended material and compaction requirements are presented for engineered fills on the project site. The geotechnical engineer should evaluate the suitability of all proposed fill materials for the project prior to placement.

1. *Structural Fill*: Structural fill should consist of on-site overburden materials or imported materials. Imported fill, if necessary, should be non-expansive with a maximum of 60% passing the No. 200 sieve, a maximum liquid limit of 30, and a maximum plasticity index of 10.

2. *Pipe Bedding Material*: Pipe bedding material should be a free draining, coarse-grained sand and/or fine gravel. The on-site soils anticipated to be available for use as fill should be suitable for reuse; however, some of the onsite soils available may include materials with relatively high fines content that may not be suitable for pipe bedding.

3. *Utility Trench Backfill*: Materials excavated from the utility trenches may be used for trench backfill above the pipe zone fill provided they do not contain unsuitable material or particles larger than 4 inches.

4. *Material Suitability*: Unless otherwise defined herein, all fill material should be a non-expansive soil free of vegetation, brush, sod, trash and debris, and other deleterious substances, and should not contain rocks or lumps having a diameter of more than 4 inches. A fill material should be considered non-expansive if the swell potential of the material, when remolded to 95% of the standard Proctor (ASTM D698) maximum dry density at optimum moisture content, does not exceed 0.5% when wetted under a 200 psf surcharge pressure. If grading is performed during times of freezing weather, the fill should not contain frozen materials, and, if the subgrade is allowed to freeze, all frozen material should be removed prior to additional fill placement or footing, slab or pavement construction.

   Evaluation of potential structural fill sources, particularly those not meeting the above liquid limit and plasticity index criteria for imported fill materials, should include determination of laboratory moisture-density relationships and swell-consolidation tests on remolded samples prior to acceptance.

<u>Compaction Requirements</u>: We recommend the following compaction criteria be used on the project:

1. *Moisture Content*: Fill materials should be compacted at moisture contents between optimum and 3 percentage points above optimum for predominantly clay materials, and within 2 percentage points of the optimum moisture content for predominantly granular materials and. The contractor should be aware the clay soils, including on-site and imported materials, may become somewhat unstable and deform under wheel loads if placed near the upper end of the moisture range.

2. *Placement and Degree of Compaction*: General site grading fill and fill beneath slab-on-grade floors should be placed in maximum 8-inch-thick lifts.

Unless recommended otherwise in specific sections of this report, the following compaction criteria should be followed during construction:

Fill Location	Percentage of Maximum Standard Proctor Density (ASTM D698)
Beneath Footings/Foundation Elements	100%
Beneath Floor Slabs and Pavements[1]	95%
Utility Trenches	95%
General Site Grading and Landscape Areas	95%

[1] Aggregate base course should be compacted to a minimum of 95 percent of the modified Proctor (ASTM D1557) maximum dry density at moisture contents within 2 percentage points of optimum.

Subgrade Preparation: Prior to placing site grading fill or structural fill, the upper 12 inches of the subgrade soils at the base of the fill zone should be scarified, moisture-conditioned, and compacted to at least 95% of the standard Proctor (ASTM D698) maximum dry density at the moisture contents recommended above. Where feasible, the prepared subgrade should be proof rolled with moderately heavy to heavy compaction equipment to identify soft areas exhibiting excessive deflection. Those areas should be removed from suitable soils or bedrock and replaced with site grading or structural fill.

If grading is performed during times of cold weather, the fill should not contain frozen materials. If the subgrade is allowed to freeze, all frozen material should be removed prior to additional fill placement or pavement construction.

Excessive wetting and drying of excavations and prepared subgrade areas should be avoided during construction. It is extremely important that moisture-conditioned fill placed during construction is not allowed to dry-out. Allowing the fill to dry after placement increases the materials' potential to heave if the moisture content of the fill is increased in the future.

PAVEMENT DESIGN

A pavement section is a layered system designed to distribute concentrated traffic loads to the subgrade. Performance of the pavement structure is directly related to the physical properties of the subgrade soils and traffic loadings. Soils are represented for pavement design purposes by means of a subgrade resilient modulus for flexible pavements and a modulus of subgrade reaction for rigid pavements.

Subgrade Materials: Based on the results of the field and laboratory studies, the near-surface subgrade materials at the site classify as A-1-a, A-1-b, A-2-4, A-6, and A-7-5 soils with a group between 0 and 18 in accordance with the American Association of State Highway and Transportation Officials (AASHTO) classification. Soils classifying as such are generally considered to provide fair to good subgrade support. For design purposes, a resilient modulus value of 4,500 psi was selected for flexible pavements and a corrected modulus of subgrade reaction of 60 pci was selected for rigid pavements.

Design Traffic: Since anticipated traffic loading information was not available at the time of this report preparation, an 18-kip equivalent single axle loading (ESAL) value of 36,500 was assumed for the paved parking surfaces (Standard-Duty), and an ESAL of 109,000 was assumed for drive and fire lane areas (Heavy-Duty). The values are selected based on our past experience for facilities of this nature. The Heavy-Duty pavement section should be constructed in locations of concentrated vehicular traffic movements.

If estimated daily traffic volumes for the facility are known to be different from those assumed, we should be provided with this information in order to reevaluate the pavement sections provided below.

Pavement Thickness Requirements: The Town of Parker requires that all publicly maintained streets and facilities located on Town-owned properties be paved using a composite pavement section. Since the carwash resides on private property, a full depth pavement section may be considered. Recommendations for a full depth asphalt section, a composite section of hot mix asphalt (HMA) over aggregate base course (ABC) and for a rigid Portland cement concrete pavements (PCCP) section are presented in the table below. The pavement sections were determined in accordance with the 1993 AASHTO pavement design procedures and the Town of Parker Roadway Design and Construction Criteria. The following table presents the minimum pavement thickness alternatives for the project:

LOCATION	Full Depth Asphalt Pavement (inches)	Asphalt Over Aggregate Base Course (inches)	PCCP (inches)
Standard-Duty	5.5	4.0 over 6.0	6.0
Heavy-Duty	6.0	4.5 over 6.0	7.0

Dumpster pads and any areas of the pavement that will be subjected to concentrated truck turning movements should be paved using a minimum section consisting of 7.0 inches of PCCP.

Pavement Materials: The following are recommended material and placement requirements for pavement construction for this project site. We recommend that properties and mix designs for all materials proposed to be used for pavements be submitted for review to the geotechnical engineer prior to placement.

1. *Aggregate Base Course*: Aggregate base course (ABC) used beneath hot mix asphalt (HMA) pavements or as a working surface below PCCP, should meet the material specifications for Class 6 ABC stated in the current CDOT Specifications. The ABC should be placed and compacted as outlined in the "Site Grading and Earthwork" section of this report.

2. *Hot Mix Asphalt:* Hot mix asphalt (HMA) materials and mix designs should meet the applicable requirements indicated in the current CDOT Specifications. We recommend that the HMA used for this project is designed in accordance with the SuperPave gyratory mix design method. The mix should generally meet Grading S or SX specifications with a SuperPave gyratory design revolution (N_{DESIGN}) of 75. The mix design for the HMA should use a performance grade PG 58-28 asphalt binder. A PG 64-22 binder will also be sufficient to carry the traffic loads, but will be more susceptible to low temperature cracking. Placement and compaction of HMA should follow current CDOT standards and specifications.

3. *Portland Cement Concrete Pavement*: PCCP should meet Class D or P specifications and requirements in the current CDOT Specifications. Rigid PCCP pavements are more sensitive to distress due to movement resulting from settlement or heave of the underlying base layer and/or subgrade than flexible asphalt pavements. The PCCP should contain sawed or formed joints to 1/3 of the depth of the slab at a maximum distance of 12 feet on center. The PCCP thickness design presented above is for an unreinforced pavement section.

Subgrade Preparation: Prior to placing the pavement section, we recommend the upper 12-inches of the subgrade should be removed. Prior to replacing the fill, the entire subgrade area should be thoroughly scarified to a depth of 12 inches, adjusted to a moisture content and compaction as indicated in the Site Grading section of this report. Additionally, in areas where claystone is exposed, an additional 12-inches of subgrade should be removed and replaced with structural fill.

The pavement subgrade should be proofrolled with a heavily loaded pneumatic-tired vehicle. Pavement design procedures assume a stable subgrade. Areas which deform excessively under heavy wheel loads are not stable and should be removed and replaced to achieve a stable subgrade prior to paving. Areas of existing fill may also require removal and replacement if they are either unstable or not well compacted.

Drainage: The collection and diversion of surface drainage away from paved areas is extremely important to the satisfactory performance of pavement. Drainage design should provide for the removal of water from paved areas and prevent the wetting of the subgrade soils. Joints should be routinely inspected, and joints and cracks that develop after construction should be sealed to reduce the potential for water to migrate through the pavement.

DESIGN AND CONSTRUCTION SUPPORT SERVICES

K+A should be retained to review the project plans and specifications for conformance with the recommendations provided in our report. We are also available to assist the design team in preparing specifications for geotechnical aspects of the project, and performing additional studies, if necessary to accommodate possible changes in the proposed construction.

We recommend that K+A be retained to provide construction observation and testing services to document that the intent of this report and the requirements of the plans and specifications are being followed during construction. This will allow us to identify possible variations in subsurface conditions from those encountered during this study and to allow us to re-evaluate our recommendations, if needed. We will not be responsible for implementation of the recommendations presented in this report by others, if we are not retained to provide construction observation and testing services.

LIMITATIONS

The conclusions and recommendations submitted in this report are based upon data obtained from the exploratory borings at the locations indicated on Fig. 1, and the proposed construction. This report may not reflect subsurface variations that occur between the explorations, and the nature and extent of variations across the site may not become evident until site grading and excavations are performed. If during construction, fill, soil, bedrock or groundwater conditions appear to be different from those described herein, K+A should be advised at once so that a re-evaluation of the recommendations presented in this report can be made. K+A is not responsible for liability associated with interpretation of subsurface data by others.

Swelling soils occur on this site. Such soils are stable at their natural moisture content but will undergo high volume changes with changes in moisture content. The extent and amount of perched water beneath the structure as a result of area irrigation and inadequate surface drainage is difficult, if not impossible, to foresee.

The recommendations presented in this report are based on current theories and experience of our engineers on the behavior of swelling soils in this area. Standards of practice in this area evolve over time. The owner should be aware there is a risk in constructing in an expansive soil area. Following the recommendations given by a geotechnical engineer, careful construction practice and prudent maintenance by the owner can, however, decrease the risk of foundation movement due to expansive soils.

RAE/mm
Rev: JLB
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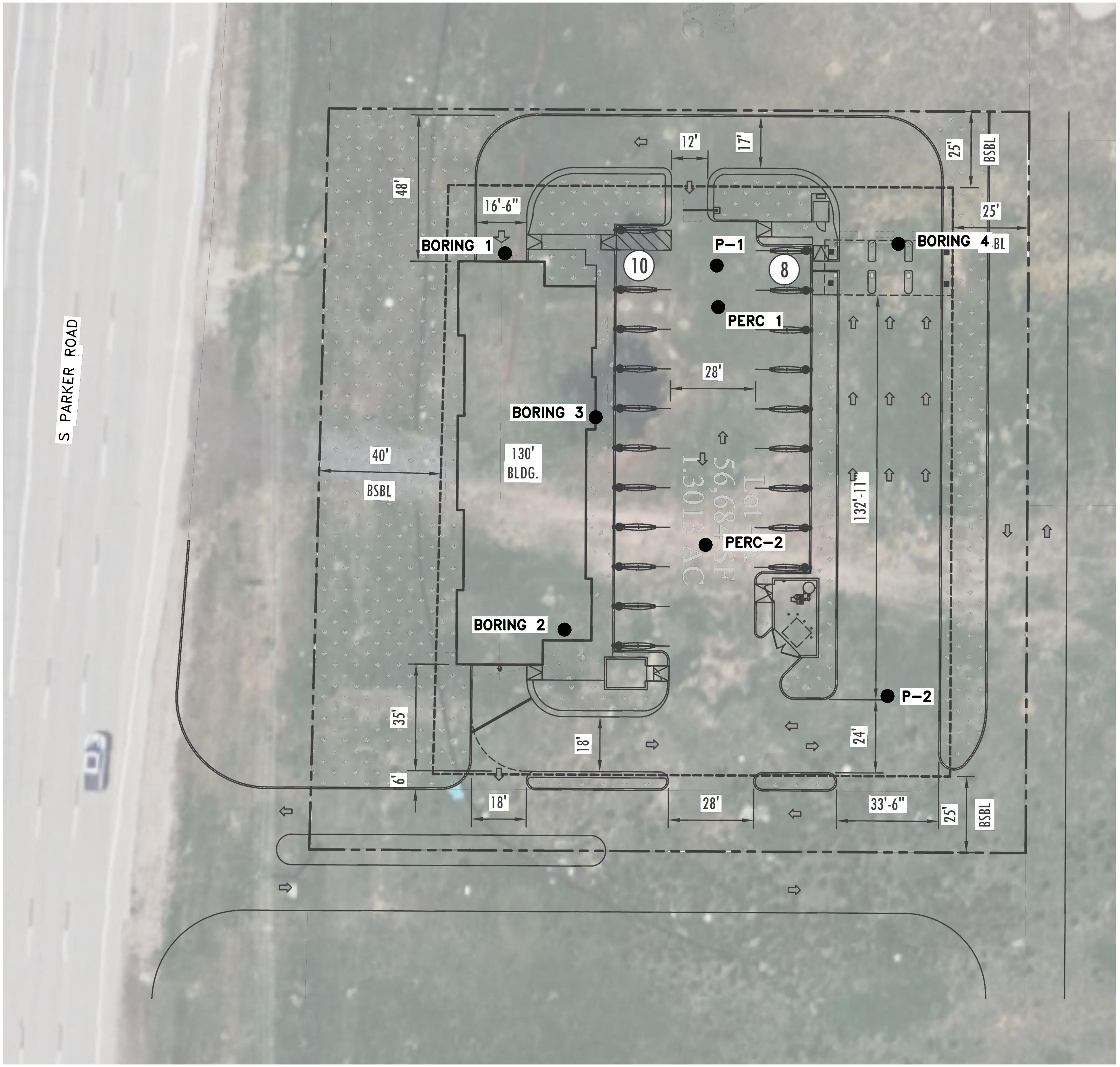
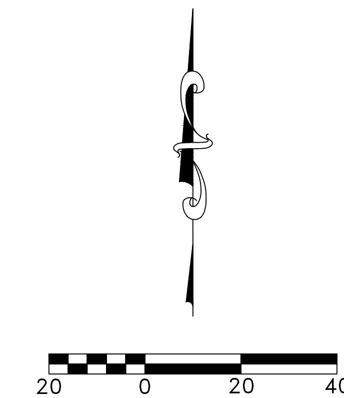
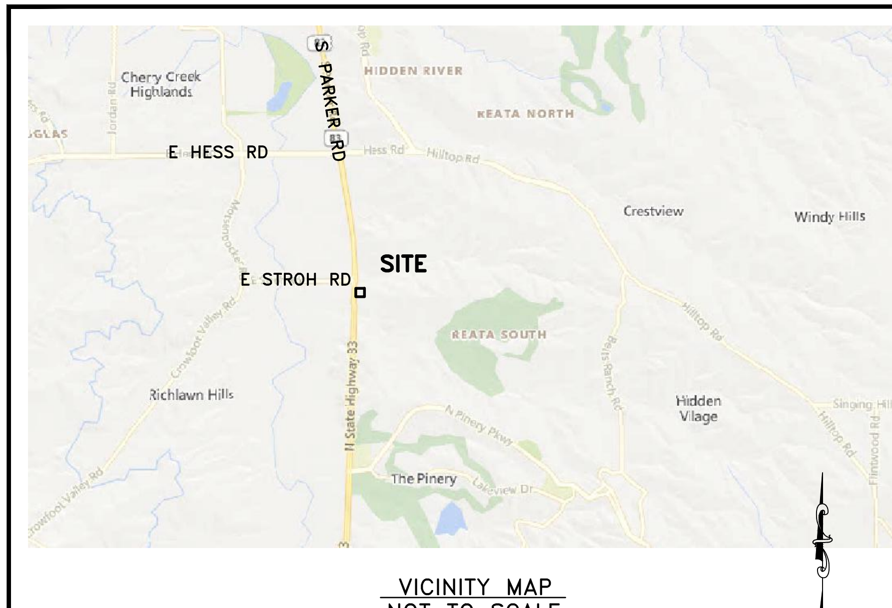




APPROXIMATE SCALE-FEET



VICINITY MAP
NOT TO SCALE

23-1-430	Kumar & Associates	MISTER CARWASH – PARKER, 13225 PARKER ROAD, PARKER, COLORADO	LOCATION OF EXPLORATORY BORINGS	Fig. 1

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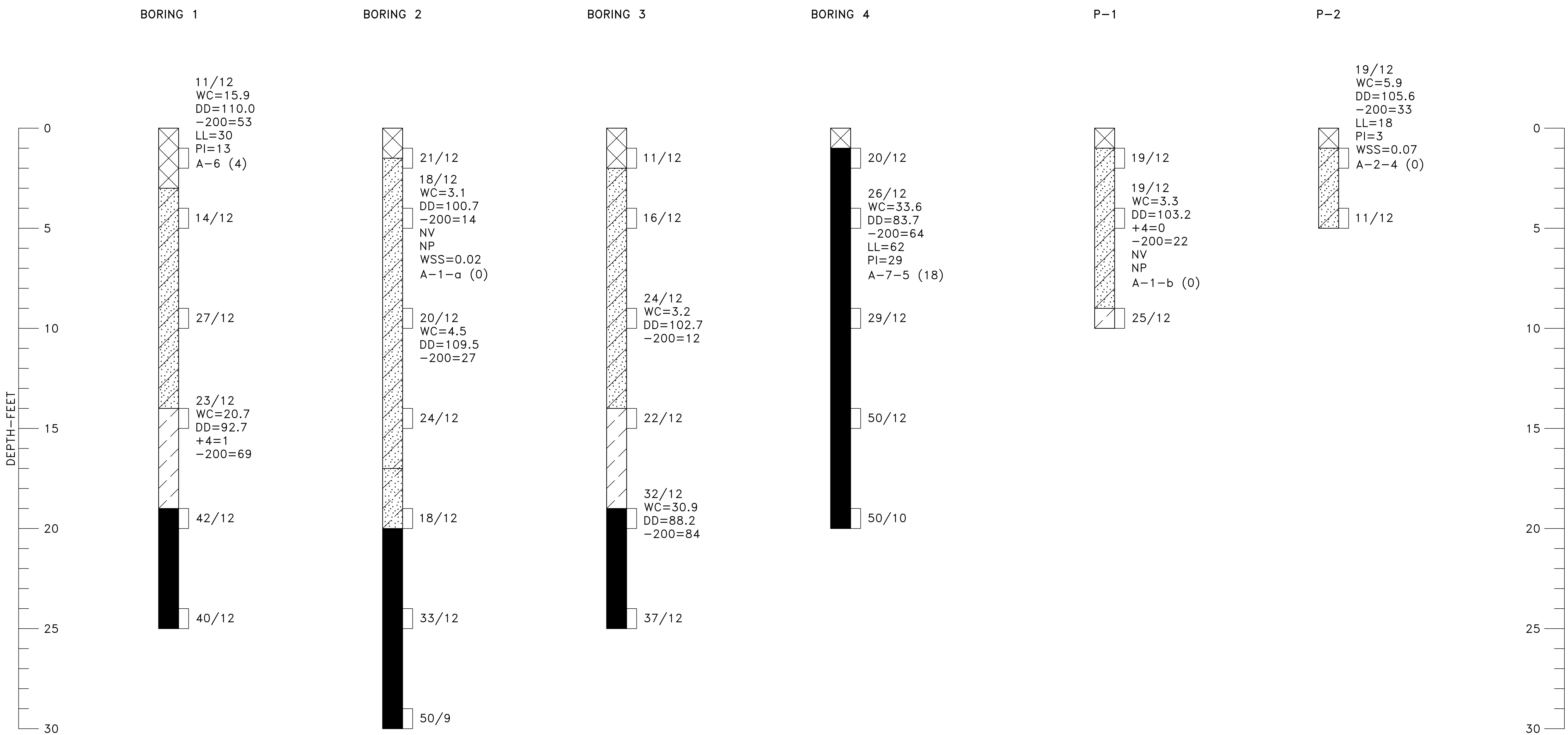

23-1-430	Kumar & Associates	MISTER CARWASH - PARKER, 13225 PARKER ROAD, PARKER, COLORADO	LOGS OF EXPLORATORY BORINGS	Fig. 2

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LEGEND

FILL: SANDY LEAN CLAY (CL) TO CLAYEY SAND (SC), FINE– TO COARSE–GRAINED SAND FRACTION, MOIST, DARK BROWN TO BROWN TO TAN.

SILTY SAND (SM), FINE– TO MEDIUM–GRAINED, MEDIUM DENSE, MOIST, BROWN TO TAN.

SANDY LEAN CLAY (CL), FINE– TO COARSE–GRAINED SAND FRACTION, VERY STIFF, MOIST, DARK BROWN TO REDDISH BROWN.

CLAYEY SAND (SC), FINE– TO MEDIUM–GRAINED, MEDIUM DENSE, MOIST, BROWN.

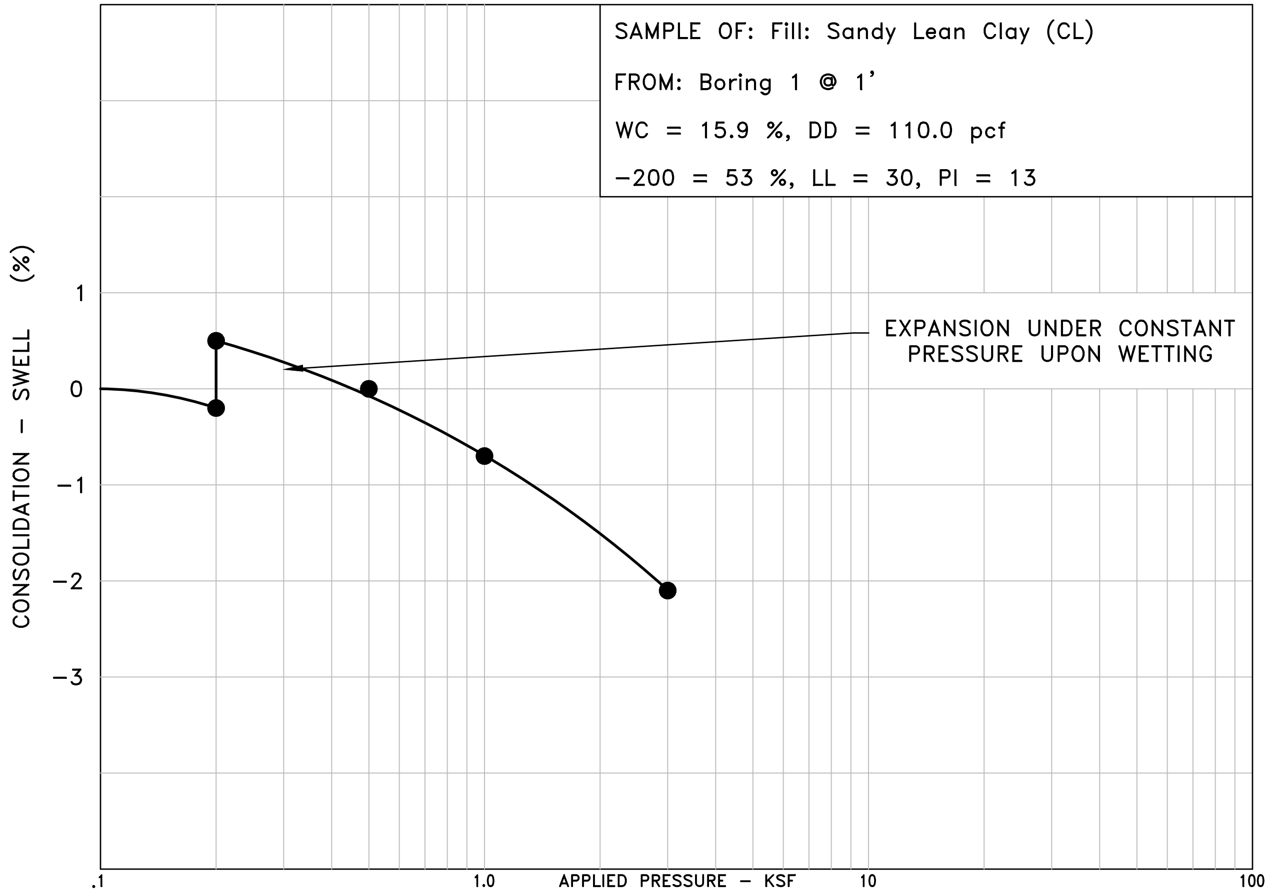
CLAYSTONE BEDROCK, FINE–GRAINED, MEDIUM HARD TO HARD WITH ISOLATED FIRM ZONES, MOIST, DARK BROWN TO DARK GRAY TO GRAY.

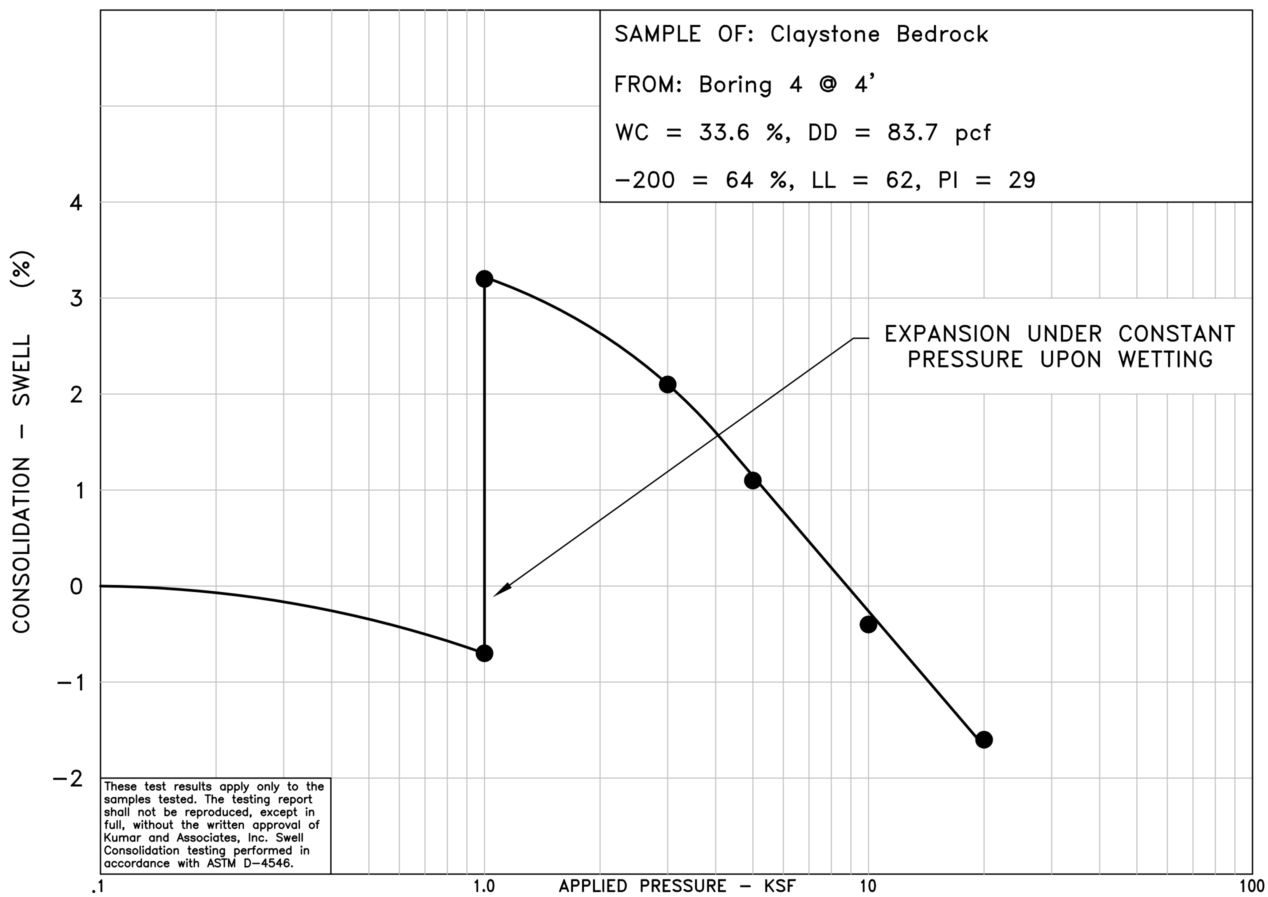
DRIVE SAMPLE, 2–INCH I.D. CALIFORNIA LINER SAMPLE.

11/12 DRIVE SAMPLE BLOW COUNT. INDICATES THAT 11 BLOWS OF A 140–POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE THE SAMPLER 12 INCHES.

NOTES

1. THE EXPLORATORY BORINGS WERE DRILLED ON SEPTEMBER 8, 2023 WITH A 4–INCH–DIAMETER CONTINUOUS–FLIGHT POWER AUGER.
2. THE LOCATIONS OF THE EXPLORATORY BORINGS WERE LOCATED BY GPS COORDINATES OBTAINED FROM GOOGLE EARTH™ AND LOCATED IN THE FIELD WITH A HANDHELD GPS UNIT.
3. THE ELEVATIONS OF THE EXPLORATORY BORINGS WERE NOT MEASURED AND THE LOGS OF THE EXPLORATORY BORINGS ARE PLOTTED TO DEPTH.
4. THE EXPLORATORY BORING LOCATIONS SHOULD BE CONSIDERED ACCURATE ONLY TO THE DEGREE IMPLIED BY THE METHOD USED.
5. THE LINES BETWEEN MATERIALS SHOWN ON THE EXPLORATORY BORING LOGS REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN MATERIAL TYPES AND THE TRANSITIONS MAY BE GRADUAL.
6. GROUNDWATER WAS NOT ENCOUNTERED IN THE BORINGS AT THE TIME OF DRILLING OR WHEN CHECKED 5 DAYS LATER.
7. LABORATORY TEST RESULTS:
   WC = WATER CONTENT (%) (ASTM D2216);
   DD = DRY DENSITY (pcf) (ASTM D2216);
   +4 = PERCENTAGE RETAINED ON NO. 4 SIEVE (ASTM D6913);
   –200 = PERCENTAGE PASSING NO. 200 SIEVE (ASTM D1140);
   LL = LIQUID LIMIT (ASTM D4318);
   PI = PLASTICITY INDEX (ASTM D4318);
   NV = NO LIQUID LIMIT VALUE (ASTM D4318);
   NP = NON–PLASTIC (ASTM D4318);
   WSS = WATER SOLUBLE SULFATES (%) (CP–L 2103);
   A–6 (4) = AASHTO CLASSIFICATION (GROUP INDEX) (AASHTO M 145).





23−1−430	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULTS	Fig. 4

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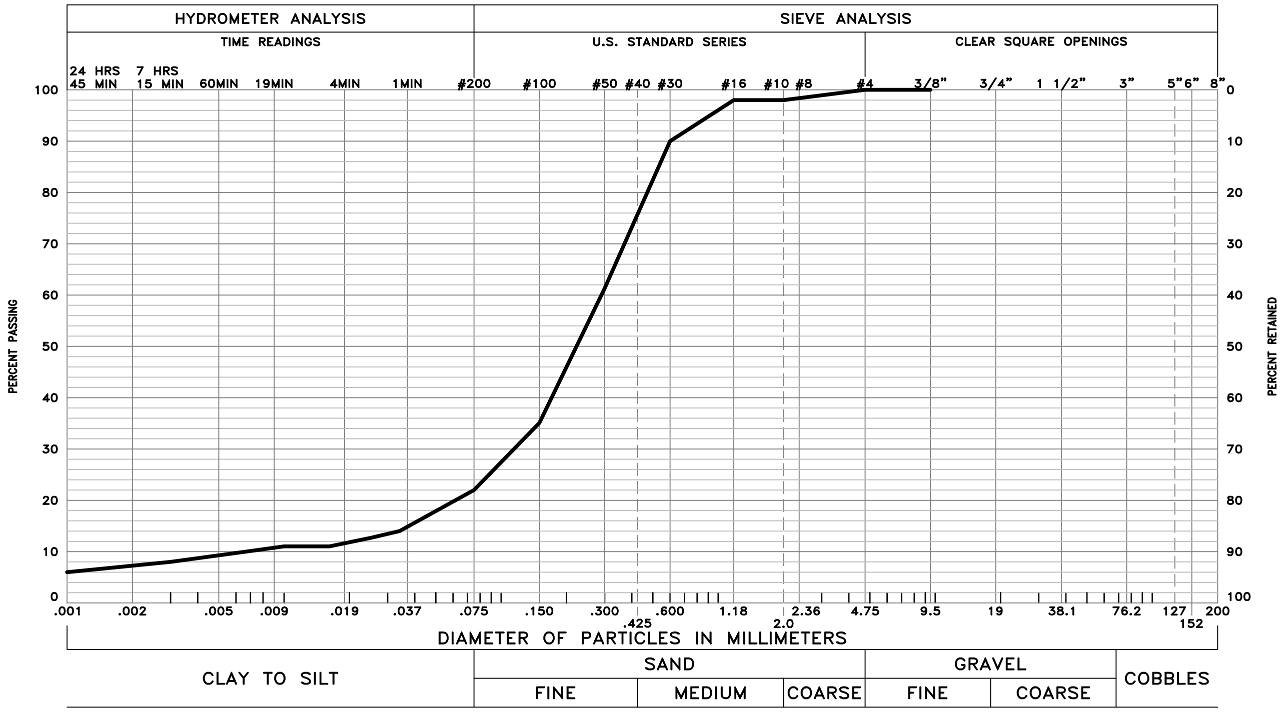

GRAVEL 0 % SAND 78 % SILT AND CLAY 22 %

LIQUID LIMIT NV PLASTICITY INDEX NP

SAMPLE OF: Silty Sand (SM) FROM: P-1 @ 4'

These test results apply only to the samples which were tested. The testing report shall not be reproduced, except in full, without the written approval of Kumar & Associates, Inc. Sieve analysis testing is performed in accordance with ASTM D6913, ASTM D7928, ASTM C136 and/or ASTM D1140.

23-1-430	Kumar & Associates	GRADATION TEST RESULTS	Fig. 5

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TABLE I
SUMMARY OF LABORATORY TEST RESULTS

PROJECT NO.: 23-1-430
PROJECT NAME: Mister Car Wash Parker
DATE SAMPLED: 9/8/2023
DATE RECEIVED: 9/12/2023

SAMPLE LOCATION		DATE TESTED	NATURAL MOISTURE CONTENT (%)	NATURAL DRY DENSITY (pcf)	GRADATION		PERCENT PASSING NO. 200 SIEVE	ATTERBERG LIMITS		WATER SOLUBLE SULFATES (%)	AASHTO CLASSIFICATION (group index)	SOIL OR BEDROCK TYPE
BORING	DEPTH (feet)				GRAVEL (%)	SAND (%)		LIQUID LIMIT (%)	PLASTICITY INDEX (%)			
1	1	9/13/23	15.9	110.0			53	30	13		A-6 (4)	Fill: Sandy Lean Clay (CL)
1	14	9/13/23	20.7	92.7	1	30	69					Sandy Lean Clay (CL)
2	4	9/13/23	3.1	100.7			14	NV	NP	0.02	A-1-a (0)	Silty Sand (SM)
2	9	9/13/23	4.5	109.5			27					Silty Sand (SM)
3	9	9/13/23	3.2	102.7			12					Poorly-Graded Sand with Silt (SP-SM)
3	19	9/13/23	30.9	88.2			84					Claystone Bedrock
4	4	9/13/23	33.6	83.7			64	62	29		A-7-5 (18)	Claystone Bedrock
P-1	4	9/13/23	3.3	103.2	0	78	22	NV	NP		A-1-b (0)	Silty Sand (SM)
P-2	1	9/13/23	5.9	105.6			33	18	3	0.07	A-2-4 (0)	Silty Sand (SM)