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GEOTECHNICAL ENGINEERING STUDY
AND PAVEMENT THICKNESS DESIGN
PROPOSED CIRCLE K CONVENIENCE STORE AND
FUEL STATION
JORDAN ROAD AND MAIN STREET
PARKER, COLORADO

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Project No. 20-1-336

July 2, 2020

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SUMMARY

1. Beneath about 6 inches of rooted topsoil, the subsurface conditions generally consisted of about 2 to 2.5 feet of existing man-placed fill underlain by natural cohesive soils extending to the maximum depth explored of about 20 feet in Boring 2, and to natural granular soils beginning at depths ranging from about 10 to 24.5 feet in the other borings. Bedrock was not encountered in the borings.

   Groundwater was not encountered in the borings during drilling or when follow-up measurements were made 3 days subsequent to drilling.

2. The onsite soils exhibited a low to high swell potential. In general, the onsite clay soils had relatively low in-situ moisture contents and high dry density, which we believe corresponds to an overall higher swell potential. Due to the presence of soils with a high swell pressure, deep foundations are recommended. Because bedrock is expected to be relatively deep in this area, helical piers would be well suited for the support of the proposed structures. Alternatively, if a greater risk of heave related foundation movement can be tolerated, shallow footings constructed over a zone of prepared fill may be considered. Recommendations for both foundation types are presented in this report.

3. Helical piers should extend to a zone of relatively stable moisture content and bear within soils stiff or dense enough to offer adequate bearing capacity. Based on the subsurface conditions encountered, we recommend that helical piers extend to a nominal depth of 20 feet. An allowable bearing pressure of 13,500 should be able to be achieved.

4. Footings placed on a prepared fill zone should be designed for a net allowable bearing pressure of 2,500 psf. The allowable bearing pressure may be increased by one-third for transient loads. The footings should also be designed for a minimum dead load pressure of 500 psf.

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

LOCATION	Asphalt Over Aggregate Base Course (inches)	Portland Cement Concrete (inches)
Standard-Duty	4.5 over 7.0	6.0
Heavy-Duty	5.5 over 8.0	7.0

PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical engineering study and pavement thickness design for the proposed Circle K convenience store and fuel station to be constructed at the vacant lot located at the northwest corner of Jordan Road and Cedar Gulch Parkway in Parker, Colorado. The project site is shown on Fig. 1.

A field exploration program consisting of drilling exploratory borings was conducted to obtain information on subsurface conditions. Representative samples of the soils and bedrock 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 a new single-story, at-grade, rectangular-shaped convenience store building with an approximate footprint of 5,187 sf will be constructed at the southeast side of the property. Fuel islands, a canopy structure, and three underground storage tanks (UST) will be constructed on the east side of the property, and a new car wash will be constructed on the north side of the property. We assume finished site grades are roughly +/- 2 feet of existing site grades.

We assume the building will be a masonry structure with relatively light loading conditions. We anticipate reinforced concrete foundations with reinforced concrete foundation/stem walls. Floor support will consist of a slab-on-grade floor system supported on a prepared fill zone. Access to the site will be from the northeast corner of the site. Asphalt or concrete drive lanes and parking areas will be constructed with concrete flatwork surrounding the majority of the building.

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 drilling, the site was vacant. The site is bounded on the north by similarly vacant land and Main Street further north, on the east by Jordan Road, on the south by Cedar Gulch Parkway, and on the west by similarly vacant land and East Auburn Hills Drive further west. Minor site-grading has occurred at the site with the existing topography nearly level. Vegetation was consisted of natural grasses and weeds. Beyond the immediate boundaries, the site is surrounded by predominantly single-family residential neighborhoods and a retail space to the east beyond Jordan Road.

SUBSURFACE CONDITIONS

Field Exploration: The subsurface conditions were explored by drilling five exploratory borings at the approximate locations shown on Fig. 1. The exploratory borings were advanced through the fill into the natural subsoils using 4-inch-diameter continuous-flight augers. Samples were obtained using 2-inch-I.D. California liner samplers driven into the subsurface materials with blows from a 140-pound hammer falling 30 inches. The sampling procedure is similar to the standard penetration test described by American Society for Testing and Materials (ASTM) test procedure D1586. The California liner sampler is used locally in order to obtain relatively undisturbed cohesive soil and bedrock samples. Penetration resistance values (blow counts) indicate the relative density or consistency of the soils.

Depths at which samples were taken and the associated blow counts are shown on the Logs of Exploratory Borings, on Fig. 2. A legend and notes associated with the graphic logs and describing the soils encountered are also presented on Fig. 2.

Subsurface Conditions: Beneath about 6 inches of rooted topsoil the subsurface conditions generally consisted of about 2 to 2.5 feet of existing man-placed fill underlain by natural cohesive soils extending to the maximum depth explored of about 20 feet in Boring 2, and to natural granular soils beginning at depths ranging from about 10 to 24.5 feet in the other borings. The natural granular soils extended to the maximum depths drilled of 20 to 25 feet, where they were encountered. The cohesive soils contained clayey sand lenses and the granular soils contained lean clay lenses. The transition from the cohesive soils to the granular soils may be more gradual than indicated on the logs on Fig. 2. Bedrock was not encountered in the borings.

The existing fill generally consisted of slightly moist to moist, dark brown, lean to fat clay with a variable fine- to coarse-grained sand fraction. The vertical and horizontal extent along with placement conditions of the fill were not determined as part of this study.

The natural soils consisted of a mix of lean clay (cohesive) and poorly-graded sand with clay and gravel to clayey sand (granular). The natural soils contained a variable fine- to coarse-grained sand fraction, and were generally slightly moist to moist, reddish-brown to brown, and calcareous in places. Based on the blow counts recorded, the cohesive soils were generally very stiff to hard, and the granular soils were medium dense to dense.

Groundwater Conditions: Groundwater was not encountered during drilling or when follow-up measurements were taken 3 days subsequent to drilling.

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, grain size distribution, liquid and plastic limits, and swell-consolidation behavior. These tests were performed in accordance with the applicable ASTM standard test procedures. The percentage of water-soluble sulfates was determined in accordance with the Colorado Department of Transportation (CDOT) CP-L 2103 test procedure. The results of the laboratory tests are shown to the right of the logs on Fig. 2, plotted graphically on Figs. 3 and 5, and summarized in Table 1.

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 and plasticity index and grain size distribution. Values for in-situ moisture content and dry unit weight, liquid limit and plasticity index, and the percent of soil retained on the U.S. No. 4 sieve and 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 representative samples of the existing fill and natural cohesive soils to determine their swell and/or compressibility under loading and when submerged in water. Each sample was prepared and placed in a confining ring between porous discs, subjected to a surcharge pressure of either 200 or 1,000 psf, and allowed to consolidate before being submerged. 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 pressures ranging from 5,000 to 20,000 psf, and the sample heights were monitored until deformation practically ceased under each load increment.

Results of the swell-consolidation tests are presented on Figs. 3 through 5 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 tests, the samples of the existing fill and natural cohesive soils exhibited a low to high swell potential when wetted.

GEOTECHNICAL ENGINEERING CONSIDERATIONS

General: Existing fill materials encountered in the borings extended to depths ranging from 2 to 2.5 feet. Without documentation regarding placement and testing, the fill should be considered nonengineered and unsuitable in its current condition for support of foundations, floor slabs, exterior flatwork, and rigid pavements. Further, the existing fill included fat clays that were found to be expansive. The fill should be completely removed and replaced with structural fill beneath foundations, floor slabs, and movement-sensitive flatwork. The fill materials onsite may be suitable for reuse as general site grading and potentially structural fill, provided they meet the material and placement criteria presented in the "Site Grading and Earthwork" section of this report.

The onsite soils exhibited a low to high swell potential. In general, the onsite clay soils had relatively low in-situ moisture contents and high maximum dry density, which we believe corresponds to an overall higher swell potential.

Deep foundations will provide the lowest risk of post construction settlement. Because bedrock is relatively deep in this area, helical piers would be a suitable deep foundation system. Recommendations for helical piers have been provided herein, but other deep foundation alternatives may be feasible. We should be contacted to provide additional recommendations if alternative foundations are considered. Alternatively, shallow foundations constructed over a zone of prepared fill as outlined in this report may be considered if an increased risk of heave related foundation movement can be tolerated.

Floor slabs present a difficult problem where expansive soils are present near the floor slab elevation because sufficient deadload cannot be imposed upon them in order to resist the uplift pressure generated when the soils are wetted and expand. To mitigate the risk of damage to the floor, structural slabs constructed over crawlspaces are often used when highly expansive soils such as those encountered at this site are present. As a cost saving alternative, slabs constructed on moisture conditioned and recompacted soil may be considered. It should be noted that a greater risk of cracking and associated distress due to heaving soils can be anticipated with slab-on-grade construction. The following discussion presents estimates of ground heave for different wetting depth scenarios to aid in the decision-making process for foundation and floor support systems.

Discussion of Foundation and Floor Slab Movement: In accordance with the practice in this area, the following discussion presents estimates of ground heave for different wetting depth scenarios to aid in the decision-making process for foundation and floor support systems. The risk of ground heave beneath the buildings can be reduced to a certain degree by providing a zone of non- to low-swelling, relatively impervious, compacted fill directly beneath foundations and floor slabs. Heave estimate calculations can be useful in evaluating the relative effectiveness of varying the thickness of this prepared fill zone. However, such calculations cannot address the uncertainty in the potential depth and degree of wetting that may occur beneath the buildings or the variability of swell potential across the site.

We have performed calculations to demonstrate the potential for ground heave if the expansive soils beneath the proposed structures should be thoroughly wetted to significant depth, including below the depth of the prepared fill zone. The following table presents estimates of potential heave based on the results of swell-consolidation tests using analysis methods generally accepted along the Colorado Front Range. Both depth of wetting and depth of the prepared fill zone were considered as variables in the analysis.

Slab-On-Grade Prepared Fill Zone Alternatives		Ground Heave in Inches	
		10 feet of wetting	15 feet of wetting
1	Existing soils without moisture treatment (informational)	4.4	6.0
2	3 feet of non-expansive (0% swell at 200 psf) imported structural fill over 4 feet of moisture-treated low-swelling on-site clay soil (maximum 1% swell at 200 psf).	1.2	2.7
3	7 feet of non-expansive (0% swell at 200 psf) imported structural fill placed on a prepared subgrade	0.4	2.0

It should be noted that the heave estimates presented above are somewhat conservative and would be reduced to varying degrees by the presence and thickness of the natural granular zones. Also, the heave estimates are for floor slabs, which are generally lightly loaded; the amount of heave that would be experienced by a spread footing with a minimum dead load would be less.

The heave estimate calculations demonstrate moderate to significant heave should be expected if thorough wetting of the expansive soils beneath the buildings occurs to significant depth below the bottom of the prepared fill zone. However, our experience indicates the large majority of similar structures underlain by similar materials do not experience extreme moisture increases in the underlying materials to significant depth provided good surface and subsurface drainage is designed, constructed, and maintained, and good irrigation practices are followed. Wetting can also occur as a result of unforeseeable influences such as plumbing leaks or breaks, or, in some cases, even due to off-site influences depending on geologic conditions. We estimate under normal conditions there is a low risk of total and differential movement in excess of 2 inches for slabs-on-grade and 1 inch for shallow foundations designed and constructed as recommended herein.

Car wash facilities are at an inherently higher risk of distress due to moisture related movements due to the likelihood of surface water migrating into the subsoils. Use of an underdrain system and other maintenance practices to intercept and control water that may otherwise reach the underlying expansive clay soils is highly recommended.

We recommend low intensity drip irrigation, and limited vegetation in the area around the structures, which should result in a relatively low risk of excessive heave. The risk could be further reduced by eliminating landscape irrigation within about 15 to 20 feet of the building and limiting irrigation elsewhere on site.

To reduce the potential for development of perched water in the fill, a perimeter underdrain system should extend below the base of the prepared fill zone. Recommendations for a perimeter underdrain system are provided in the “Underdrain System” section of this report. The underdrain should be placed at the base of the prepared fill zone.

With adequate subgrade preparation, we believe shallow foundation systems will be feasible for the support of the main building, car wash, and canopy structures. The depth of over-excavation presented herein was determined based on the above table in order to reduce the swell potential of the onsite soils to a tolerable level should they become saturated. Given the recommended depth of over-excavation, the existing fill onsite would likely be removed in its entirety.

Because of the presence of expansive soils extending to significant depths below the existing ground surface, drilled footings are not recommended for the support of the proposed canopy structures.

The subgrade preparation recommendations for foundations and soil-supported slabs would result in a uniform fill zone beneath the building footprint. Risk of post-construction settlement increases when the thickness of the depth of a fill zone increases. Creating a uniform fill zone with the elevation of over-excavation depths beneath foundations and soil-supported slabs matching would reduce the potential risk of differential settlements across the building footprint area. Should settlement of the prepared fill zone occur, a fill zone prepared as with a uniform base elevation will likely settle uniformly with less of an impact on architectural finishes. We believe a fill zone prepared as recommended herein would result in total settlements of less than about 1 inch, with differential settlements between ½ to ¾ of the total settlement.

Considering the above discussion, we believe soil-supported floor slabs may be considered for the project, provided that the potential for floor slab movement due to ground heave and associated possible distress is recognized by the owner. The intent of our recommendations for soil-supported floor slabs is to provide for conditions where there is a good chance ground heave beneath the building will not exceed amounts acceptable to the owner. The recommendations should result in heave movements that are unlikely to significantly exceed about 2 inches unless extreme wetting is allowed. Barring unforeseen events, we do not believe extreme wetting is likely to occur if the surface drainage and irrigation recommendations presented in this report are followed.

Underground Storage Tanks: Groundwater and bedrock are not anticipated within UST excavations. Therefore, a dewatering system during construction and an anchoring system to counteract buoyant forces should not be necessary. However, due to the swelling soils, we recommend USTs be supported on a minimum of 3 feet of structural fill. An underdrain system should also be considered for the USTs due to the swelling soils encountered. Recommendations for the design and construction of underdrain systems is presented in the "Underdrain System" section of this report.

FOUNDATION RECOMMENDATIONS

Helical Pier Foundations: The axial design load of helical piers should be determined in general accordance with the current International Building Code (IBC), which states the allowable axial design load, P_a, should be determined as follows:

P_a= 0.5 P_u, where P_u (the ultimate load) is the least value of:

1. Sum of the areas of the helical bearing plates times the ultimate bearing capacity of the soil or rock comprising the bearing stratum.
2. Ultimate capacity determined from well-documented correlations with installation torque.
3. Ultimate capacity determined from load tests.
4. Ultimate capacity of pile shaft.
5. Ultimate capacity of pile couplings.
6. Sum of the Ultimate axial capacity of helical bearing plates affixed to pile.

Items 1 through 3 are related to the geotechnical capacity of the piers; Items 4 through 6 are related to the structural capacity and should be evaluated by the structural engineer. The owner and structural designer should be aware that certain proprietary helical pier systems have been subjected to acceptance testing administered by the International Code Council (ICC), while other systems provided by specialty contractors may be fabricated according to designs by registered professional engineers. The certified systems have documentation that addresses many of the structural capacity issues, while the non-certified systems require structural design by an engineer. Many of the lighter-duty helical pile systems available, with working capacities on the order of 50 kips or less, are certified, which can simplify the design and submittal process. However, higher capacity systems, where single piers may have working capacities of 200 kips or more, sometimes referred to as screw piles, are often designed and fabricated and are not certified, manufactured systems.

Based on consideration of bearing capacity theory and published correlations of boring penetration resistance values with ultimate bearing capacity, we recommend an ultimate bearing capacity of 13,500 psf for a helical pile embedded in the native soils. We also recommend a minimum pier length of 20 feet. We anticipate it will be possible to achieve adequate capacities at nominal depths of 20 feet by using the appropriate size and number of bearing plates. Nominal depths should be measured from the topmost bearing plate.

Helical piers are typically very slender foundation elements with a low capacity for resisting lateral loads. Lateral restraint of a helical pile foundation system is normally provided through the use of passive pressure on pile caps or foundation walls, or through the use of battered piers. It is normally assumed that a battered pile can be designed for the same axial load as a vertical pile, with the lateral restraint being provided by the horizontal component of the battered pile. Helical piers are often assumed to have tension capacities similar to the axial compressive capacity, although that should be evaluated through load testing or otherwise addressed by the specialty contractor's submittal.

Acceptance of helical pile installation should be based on attaining a specified torque in the recommended bearing stratum determined in accordance with correlations of installation torque to capacity based on calibrated torque measurements and axial load test data. In our opinion, the ultimate bearing capacity recommended above may be exceeded if supported by adequate site-specific load test data. If site-specific load tests are not performed, the specialty helical pile contractor's submittal should contain torque-to-capacity data for their pile system in similar soil conditions. If that information cannot be provided, site-specific load tests should be performed in accordance with ASTM D 1143.

We recommend that a qualified helical pile specialty contractor be retained to provide the required design submittal and to provide and install the helical piers. The project design should include a performance specification indicating required capacities, structural requirements, and submittal requirements. At a minimum, the submittal should be required to contain information supporting capacity determination, a description of equipment and installation procedures that will ensure penetration to the required depths, and acknowledgement that the helical bearing plates will be installed into the recommended bearing stratum, as well as all necessary information to satisfy the requirements of the project structural designer.

We should be retained to review the contractor's submittal, and to provide installation observation including monitoring depths and general conformance with the plans and specifications. Our observation and testing services will be intended to document that all of the helix bearing plates on the piers are installed into an adequate bearing stratum.

Shallow Foundations: If shallow spread footings are selected for support of the main building, canopy, and carwash, we recommend spread footings be placed on a minimum of 3 feet of import structural fill underlain by a minimum of 4 feet of prepared onsite soils. The overall fill zone should extend down and out from the bottom of the building footprint at a 1 Vertical to 1 Horizontal projection.

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 on a prepared fill zone should be designed for a net allowable bearing pressure of 2,500 psf. The allowable bearing pressure may be increased by one-third for transient loads. The footings should also be designed for a minimum dead load pressure of 500 psf. In order to satisfy the minimum dead load pressure and minimum footing width criteria, it may be necessary to concentrate loads by using a grade beam and pad or similar foundation design. If this system is used, a void should be provided beneath the grade beams between pads.

2. Based on experience, we estimate total settlement for footings designed and constructed as discussed in this section will be less than 1 inch. Differential settlements between individual foundations are estimated to be approximately ½ to ¾ of the total settlement. We anticipate settlements should occur during or shortly after construction.

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

4. 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.

5. The lateral resistance of a spread footing supported as recommended herein will be a combination of the sliding resistance of the footing on the foundation 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 0.30. Passive pressure against the sides of the footings can be calculated using an equivalent fluid unit weight of 175 pcf. The above values are working values with a factor of safety of 2.0 applied. Structural fill placed against the sides of footings to resist lateral loads should meet the material and placement requirements provided in the "Site Grading and Earthwork" section of this report.

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

7. Areas of existing fill, loose and/or soft material, or deleterious substances encountered within footing excavations should be removed and replaced with structural fill. The prepared fill zone should extend down from the edges of the footings at a 1 horizontal to 1 vertical projection.

8. Care should be taken when excavating the foundations to avoid disturbing the supporting materials.

9. A representative of the geotechnical engineer should observe the base of the prepared fill zone prior to placement of structural fill and/or moisture-conditioned fill and observe and test the prepared fill zone to determine if proper compaction has been achieved prior to concrete placement.

Canopy Foundations: Canopy foundations can be designed using the helical pier or shallow foundation recommendations presented above. As noted in the “Geotechnical Considerations” section, drilled footings are not recommended for this project. If buried mat foundations are considered, a soil density of 115 pcf may be used to calculate resistance to uplift.

SITE SEISMIC CRITERIA

According to International Building Code (IBC) 2018 and Chapter 20 of ASCE 7, following construction the onsite soils should generally classify as IBC Site Class D. Based on the soil profile and anticipated depth to bedrock encountered in our borings, IBC Site Class D should be used. Based on the subsurface profile, site seismicity, and the anticipated depth of groundwater, liquefaction is not a design consideration.

FLOOR SLABS

As mentioned above, floor slabs present a problem where expansive materials are present near floor slab elevation because sufficient dead load cannot be imposed on them to resist the uplift pressure generated when the materials are wetted and expand. Based on the low to high swelling characteristics of the materials encountered, we believe slab-on-ground construction may be used, provided the risk of distress resulting from slab movement is accepted by the owner. The following measures should be taken to reduce damage which could result from movement should the underslab materials be subjected to moisture changes.

1. Floor slabs should be supported on an owner-selected prepared fill zone meeting the material and placement requirements provided in the “Site Grading and Earthwork” section of this report.

2. To reduce the effects of some differential movement, floor slabs should be separated from all bearing walls and columns with expansion joints which allow unrestrained vertical movement.

3. Non-bearing partitions resting on floor slabs should be provided with slip joints so that, if the slabs move, the movement cannot be transmitted to the upper structure. This detail is also important for wallboards and door frames. Slip joints that 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 won't 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 unless saw cut at the beam after construction.

5. Floor slab control joints should be used to reduce damage due to shrinkage cracking. The appropriate 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). The joint spacing and any requirements for slab reinforcement should be established by the designer based on experience and the intended slab use.

6. If moisture-sensitive floor coverings will be used, additional mitigation of moisture penetration into the slabs, such as by use of a vapor barrier may be required. If an impervious vapor barrier membrane is used, special precautions will be required to reduce differential curing problems which could cause the slabs to warp. American Concrete Institute (ACI) 302.1R addresses this topic.

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

8. The geotechnical engineer should evaluate the suitability of proposed prepared fill zone materials. Evaluation of potential structural fill sources will require determination of laboratory moisture-density relationships and swell consolidation tests on remolded samples.

We recommend an underdrain system be constructed at the base of the prepared fill zone to prevent development of perched water in the fill. Inclusion of a properly designed and constructed underdrain system will be a critical component in reducing potential slab heave. This underdrain system should be designed in accordance with recommendations in the “Underdrain System” section of this report.

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

EXTERIOR FLATWORK

To limit potential movement due to swelling soils and frost conditions, subgrade preparation beneath exterior flatwork immediately adjacent to the buildings, including sidewalks and apron areas, where reduction of heave potential is considered critical should be done in accordance with the recommendations provided in the “Floor Support” section of this report, including depth of sub-excavation and backfilling with prepared fill zone. Where reduction of heave potential is less of a concern, such as flatwork located more than 10 feet from buildings, subgrade preparation may be done in accordance with the subgrade preparation recommendations provided in the “Pavement Design” section of this report. Proper surface drainage measures as recommended in following sections of this report are also critical to limiting moisture- or frost-related movement.

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

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

UNDERDRAIN SYSTEM

It is our experience that perched water conditions typically will occur post-construction where excavation for a prepared fill zone extends into relatively hard natural clay soils. Perched water is primarily due to natural precipitation surface runoff, water use for irrigation, and, in some cases, accidents such as broken utility lines.

To prevent development of perched water in the prepared fill zone, an underdrain system should be constructed at the base of the prepared fill zone below the building footprint. This recommendation is for slab-on-grade floors and should be considered for flatwork areas immediately adjacent to the buildings. The underdrain system should consist of drain lines extending along the perimeter of the subslab excavation. Where feasible, the alignment of the underdrain system should preferably be just outside of the structure perimeter.

The drain lines should consist of minimum 4-inch-diameter, rigid, perforated PVC drain pipe placed in the bottom of a trench excavated to a depth of at least 1 foot below the base of the over-excavated zone. The drain pipe should be surrounded above the invert level by drainage aggregate. Drainage aggregate used in the perimeter subdrain systems should conform to the requirements of CDOT Class B or Class C Filter Material, and the drain pipe should be factory slotted or otherwise perforated in accordance with graded filter criteria. Alternatively, if a filter geotextile is used in subdrain trenches to wrap the drainage aggregate, the pipes may be covered by free-draining gravel not meeting graded filter criteria, such as AASHTO No. 57 or No. 67 coarse aggregate. During design, alternative drain aggregates and filtration methods can be considered. The perforated drain pipes themselves should not be directly wrapped in geotextile due to the potential for clogging of the geotextile at the perforations or slots.

The base of the over-excavation should be graded to slope towards the drain lines with a minimum slope of ½%. The overall underdrain pipe system should be sloped at a minimum slope of ½% to an overall site subdrain collection system or to a sump or sumps where water can be removed by pumping or gravity drainage. Sumps should be provided with alarms and/or redundant pumps in the event the pumping equipment malfunctions. In addition, the drain lines should be provided with appropriately spaced cleanouts for maintenance and inspection, which we recommend be performed on a routine basis. An over-designed sump and pump capacity is desirable in the event that groundwater or other subsurface conditions change. We also believe that standby pump capacity and standby generators should be provided in the event of pump or energy failure.

Groundwater flow rates are anticipated to be relatively low due to the low permeability characteristics expected of the clay soils underlying the site and the deep groundwater level inferred by the subsurface information gathered in our study. The volume of water collected by the underdrain system will likely vary with excavation size, and the total volume collected across the site will depend on the number of perimeter underdrain systems. Based on our experience, we believe groundwater volume collected by any one underdrain system is likely to be less than 10 gallons per minute during normal operation.

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 construction.

2. The ground surface surrounding the exterior of the building and exterior flatwork and paved areas should be sloped to drain away in all directions. We recommend a minimum slope of 12 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 1 to 2 feet of the backfill adjacent to buildings should be a relatively impervious on-site soil or be covered by flatwork or a pavement structure.

4. Exterior backfill should be moisture conditioned and compacted in controlled lifts in accordance with the “Site Grading” section.

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

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

7. Landscaping which requires relatively heavy irrigation and lawn sprinkler heads should be located at least 10 feet from foundation walls.

8. Plastic membranes should not be used to cover the ground surface adjacent to foundation walls.

WATER-SOLUBLE SULFATES

Concentrations of water-soluble sulfates measured in representative samples of the clay soils were 0.01% and 0.45%. Concentrations less than or equal to 0.10% represents a Class S0 exposure to sulfate attack on concrete exposed to these materials, and concentrations between 0.20% and 2.0% represent a Class S2 exposure. 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 recommend C150 Type V cement or equivalent sulfate resistant cement should be used for concrete exposed to the onsite soils. Equivalent Class S2 sulfate resistant cement materials are presented in ACI 201 Sections 2.2.5, 2.2.7, and 2.2.9. Alternatively, the concrete could meet the Colorado Department of Transportation's (CDOT) cement requirements for Class 2 exposure as presented in Section 601.04 of the CDOT Standard Specifications for Road and Bridge Construction (2019).

SITE GRADING AND EARTHWORK

Site Preparation: We recommend subgrade preparation below shallow foundations, slabs-on-grade floors , and movement sensitive flatwork consist of 3 feet of import granular structural fill extending to a 4-foot-thick zone of prepared onsite soils corresponding to Alternative 3 in the table presented in the "Geotechnical Engineering Considerations" section of this report. Subgrade preparation for concrete flatwork greater than 10 feet away from the structure can be done in accordance with the recommendations provided in the "Pavement Thickness Design" section of this report.

Prior to placement of fill, the exposed subgrade consisting of onsite natural clay soils should be scarified to a depth of 12 inches, adjusted to 0 to 4 percentage points above optimum moisture content, and placed at a minimum of 95% the standard Proctor (ASTM D698). The earthwork contractor should be aware that clay soils may become difficult to work with when approaching the upper end of the moisture range.

Temporary Excavations: We assume that site excavations will be constructed by over-excavating the side slopes to a stable configuration where enough space is available. Where insufficient lateral space is available due to the proximity to property boundaries and existing structures and facilities, temporary shoring may be required. It is our experience that temporary shoring systems are typically designed and built by specialty contractors and that the designers will typically develop their own design criteria based on soil data presented in the owner's geotechnical study report. Temporary shoring provided in close proximity to existing structures or traffic areas should be designed and installed to prevent lateral soil movement which could result in settlement and distress to the surrounding construction.

All excavations should be constructed in accordance with OSHA requirements, as well as state, local and other applicable requirements. Site excavations will generally encounter natural clay soils classifying as OSHA Type B soils, and if encountered, granular soils will classify as Type C soil. If localized perched water or groundwater is encountered much flatter side slopes than those allowed by OSHA or temporary shoring may be required. Excavations encountering loose granular soils, or groundwater, will require much shallower side slopes than those allowed by OSHA.

Excavated slopes in existing fill and granular soils may 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 Suitability: Based on the conditions encountered in the exploratory borings, the onsite clay soils are generally suitable for reuse as site-grading, and should be suitable for reuse as structural fill, provided it meets the material and placement criteria presented in this section. Fill should be free of organic matter, debris, trash, and other deleterious substances. The geotechnical engineer should evaluate the suitability of the proposed fill materials prior to placement.

Structural Fill: Structural fill should consist of onsite or import granular fill with non- to low-swelling minus 2-inch material with a minimum of 80 percent passing the No. 4 sieve and a maximum of 35 percent passing the No. 200 sieve with a Liquid Limit of less than 30 and a Plasticity Index of less than 15. The onsite clay soils are generally not suitable for reuse as structural fill.

Import granular structural fill shall have a nil swell potential when wetted under a 200 psf surcharge when remolded to 95% of the standard Proctor (ASTM D698) maximum dry density at the optimum moisture content.

Moisture-Conditioned Onsite Fill: Moisture-conditioned fill consisting of the onsite native clay and sand overburden soils with less than 75 percent passing the No. 200 sieve will be suitable for reuse below the proposed structure foundations. The onsite fat clay soils are unsuitable as moisture-conditioned fill.

Underground Storage Tank Considerations: To reduce the risk of excessive moisture-related heave, we recommend that the existing clay soils be removed to a depth of at least 3 feet below the base of the tank and be replaced with suitable non-expansive fill. This recommendation may be waived for excavations that would extend into the underlying granular soils. The tank bedding should meet the tank manufacturer's specifications and will likely require imported material. The recommendations for compaction and for reuse of the existing onsite soils are presented in this section.

Groundwater was not encountered several feet below the anticipated depth of the base of the tank. Although not anticipated, perched water may develop within the backfill zone, particularly after significant precipitation or during seasonal runoff. The potential for shallow groundwater or perched water should be considered when excavating the underground tanks and for the design of the tanks with respect to buoyancy when emptied.

Compaction Requirements: We recommend the following compaction criteria be used on the project:

1. *Moisture Content*: Fill materials should be compacted at moisture contents between 0 and +3 percentage points of optimum for clays and within 2 percentage points of the optimum moisture content for sands, if used. The contractor should be aware clay

materials, 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*: Unless otherwise defined herein, compacted fill should be placed in maximum 8-inch-thick loose lifts. The following compaction criteria should be followed during construction:

Fill Location:	Percentage of Maximum Standard Proctor Density (ASTM D698)
Beneath Spread Footing Foundations and Fills Deeper than 8 Feet	98%
Sides of Spread Footing Foundations and Fills Less than 8 Feet	95%
Beneath Floor Slabs, Settlement-Sensitive Flatwork Areas and Pavements[1]	95%
Utility Trenches	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.

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 soil support value 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 majority of the near-surface subgrade materials at the site classify as A-7-6 soils with group indices ranging between 12 and 26 in accordance with the American Association of State Highway and Transportation Officials (AASHTO) classification. For design purposes, a resilient modulus value of 3,025 psi was selected for flexible pavements and a modulus of subgrade reaction of 45 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 73,000 was assumed for the paved parking surfaces and automobile drive lanes (Standard-Duty), and an ESAL of 219,000 was assumed for areas accessed by tanker trucks (Heavy-Duty). The values are selected based on our past experience for facilities of this nature. We believe the ESAL values of 73,000 and 219,000 should be considered to classify as Standard Duty and Heavy-Duty pavement sections, respectively. The Heavy-Duty pavement section should be constructed in locations of heavy vehicular traffic movements such as truck and tanker routes.

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: We recommend a composite section of hot mix asphalt (HMA) over aggregate base course (ABC) and rigid Portland cement concrete pavements (PCCP) are presented in the table below. The use of a full depth pavement section is not recommended due to the clay subgrade present at the site. The pavement sections were determined in accordance with the 1993 AASHTO pavement design procedures. For design purposes, a reliability of 80% was assumed for all pavement areas. The following table presents the minimum pavement thickness alternatives that meet or exceed the structural number of the subgrade materials for the project:

LOCATION	Asphalt Over Aggregate Base Course (inches)	Portland Cement Concrete (inches)
Standard-Duty	4.5 over 7.0	6.0
Heavy-Duty	5.5 over 8.0	7.0

The above PCCP thicknesses are presented as un-reinforced slabs. Based on projects with similar heavy vehicular loading in certain areas, we recommend dowels be provided at transverse and longitudinal joints within the slabs located in the travel lanes of heavily loaded vehicle, loading docks and areas where truck turning movements are likely to be concentrated. Additionally, curbs and/or pans should be tied to the slabs. The dowels and tie bars will help minimize the risk for differential movements between slabs to assist in more uniformly transferring axle loads to the subgrade. The current CDOT *"Standard Specifications for Road and Bridge Construction"* provides some guidance on dowel and tie bar placement, as well as in the Standard Plans: M&S Standards. The proper sealing and maintenance of joints to minimize the infiltration of surface water is critical to the performance of PCCP, especially if dowels and tie bars are not installed.

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 should meet the material specifications for Class 5 or Class 6 ABC stated in the current Colorado Department of Transportation (CDOT) *"Standard Specifications for Road and Bridge Construction"*. The ABC should be placed and compacted as outlined in the Site Grading 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 *"Standard Specifications for Road and Bridge Construction"*. 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 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*: PCCP should meet Class D or P specifications and requirements in the current CDOT *"Standard Specifications for Road and Bridge Construction"*. 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. Joint locations and spacing should be established by a qualified engineer.

Subgrade Preparation: Existing fills should be completely removed from below pavements. However, we believe a portion of the fill may be left in place beneath pavements and exterior flatwork that is less movement-sensitive, provided the owner recognizes and accepts the risk of distress resulting from moisture-related movements of the underlying fill may result for undocumented fill that remains in place.

If the partial removal off existing fills is acceptably to the owner, flexible pavements and exterior flatwork that is less movement-sensitive should be supported on a minimum of 1 foot of moisture-conditioned fill. Prior to placing the fill, the entire pavement subgrade area should be scarified and well mixed to a depth of at least 12 inches, adjusted to a moisture content within 2 percentage points of optimum moisture content and compacted to 95% of the maximum standard Proctor dry density (ASTM D698). This will result in a minimum of 2 feet of prepared subgrade soil beneath the pavements and exterior flatwork. The pavement subgrade should be proof-rolled with a heavily loaded pneumatic-tired vehicle. Pavement design procedures assume a stable subgrade. Areas that deform excessively under heavy wheel loads are not stable and should be removed and replaced to achieve a stable subgrade prior to paving. Areas where excessive deflection occurs should be ripped, scarified, wetted or dried if necessary, and re-compacted to the required moisture and density specifications.

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.

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, rock 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 building site 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 soil in this area. Standards of practice in this area evolve over time. The owner should be aware that there is a risk in constructing a building 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.

AJY/as
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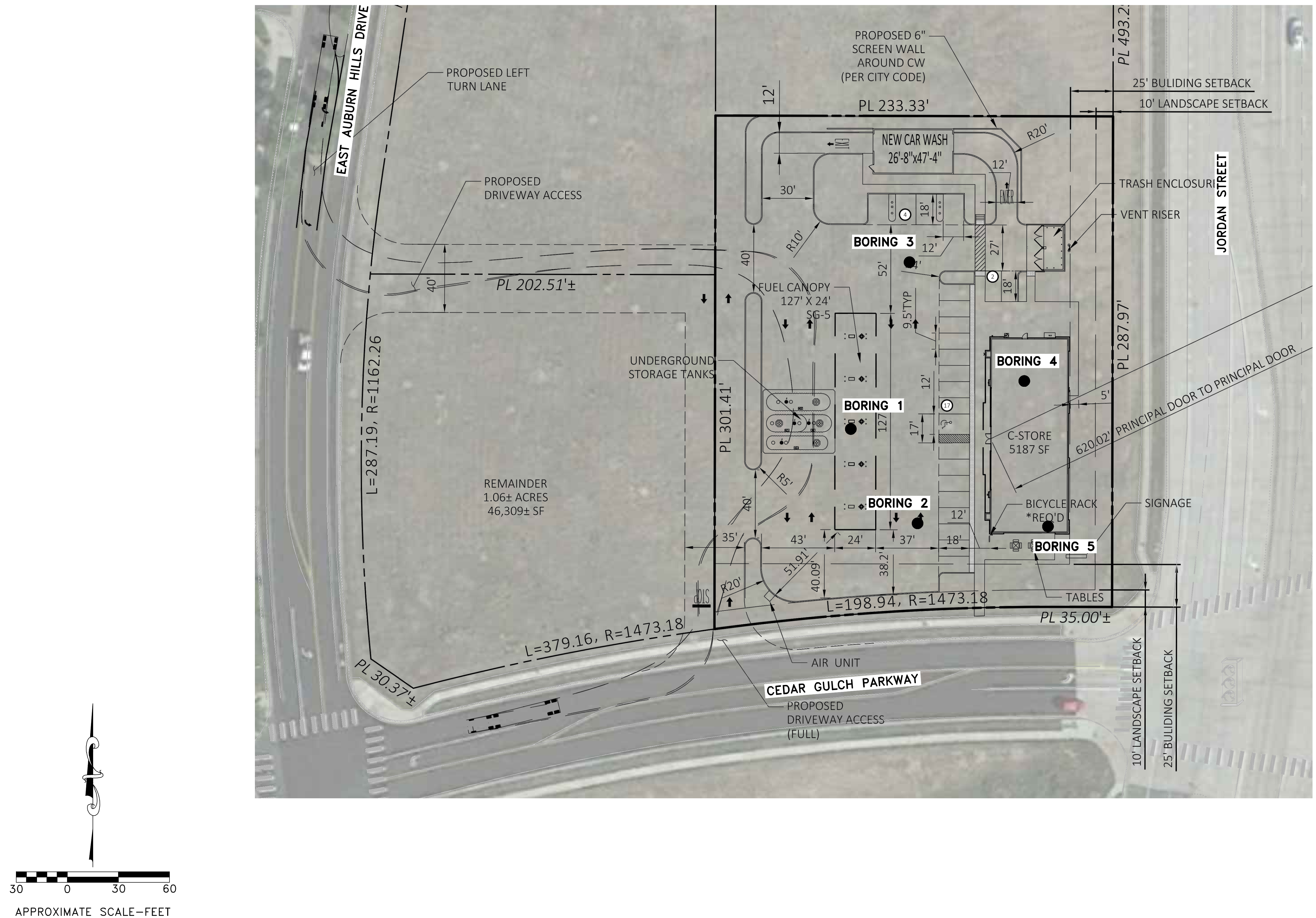

20–1–336	Kumar & Associates	PROPOSED CIRCLE K FACILITY SOUTH JORDAN ROAD & MAIN STREET, PARKER, COLORADO	LOCATION OF EXPLORATORY BORINGS	Fig. 1

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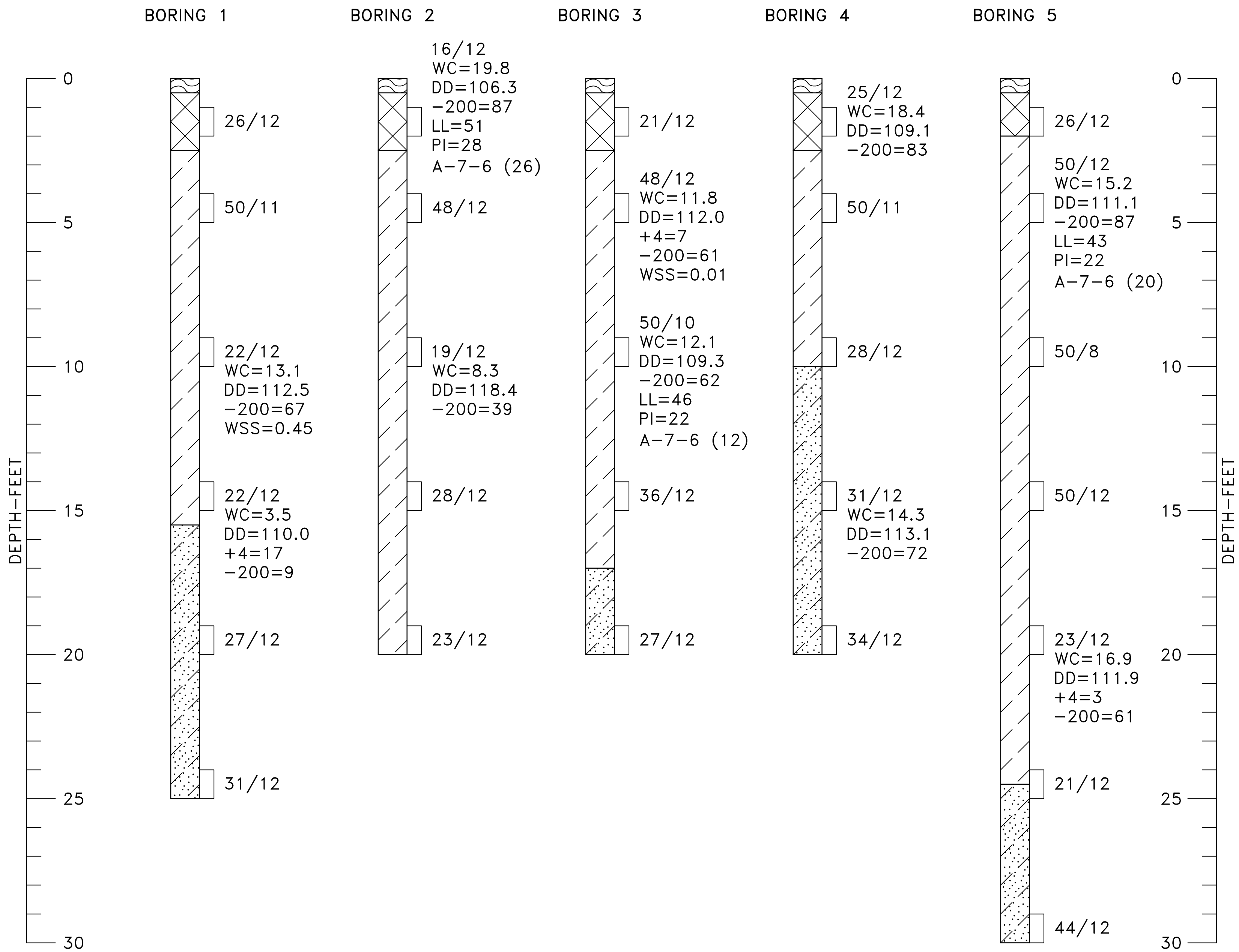

LEGEND

TOPSOIL.

FILL: LEAN CLAY (CL) TO FAT CLAY (CH) WITH VARIABLE SAND FRACTION, SLIGHTLY MOIST TO MOIST, DARK BROWN.

LEAN CLAY (CL) WITH VARIABLE FINE— TO COARSE—GRAINED SAND FRACTION, VERY STIFF TO HARD, SLIGHTLY MOIST TO MOIST, REDDISH—BROWN TO BROWN, CALCAREOUS.

POORLY—GRADED SAND WITH CLAY AND GRAVEL (SP—SC) TO CLAYEY SAND (SC), MEDIUM DENSE TO DENSE, SLIGHTLY MOIST TO MOIST, REDDISH—BROWN TO BROWN.

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

26/12 DRIVE SAMPLE BLOW COUNT. INDICATES THAT 26 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 MAY 19, 2020 WITH A 4—INCH—DIAMETER CONTINUOUS—FLIGHT POWER AUGER.
2. THE LOCATIONS OF THE EXPLORATORY BORINGS WERE MEASURED APPROXIMATELY BY PACING FROM FEATURES SHOWN ON THE SITE PLAN PROVIDED.
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 3 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 D 4318);
   WSS = WATER SOLUBLE SULFATES (%) (CP—L 2103);
   A−7−6 (26) = AASHTO CLASSIFICATION (GROUP INDEX) (AASHTO M145).

20—1—336	Kumar & Associates	PROPOSED CIRCLE K FACILITY SOUTH JORDAN ROAD & MAIN STREET, PARKER, COLORADO	LOGS, LEGEND AND NOTES OF EXPLORATORY BORINGS	Fig. 2

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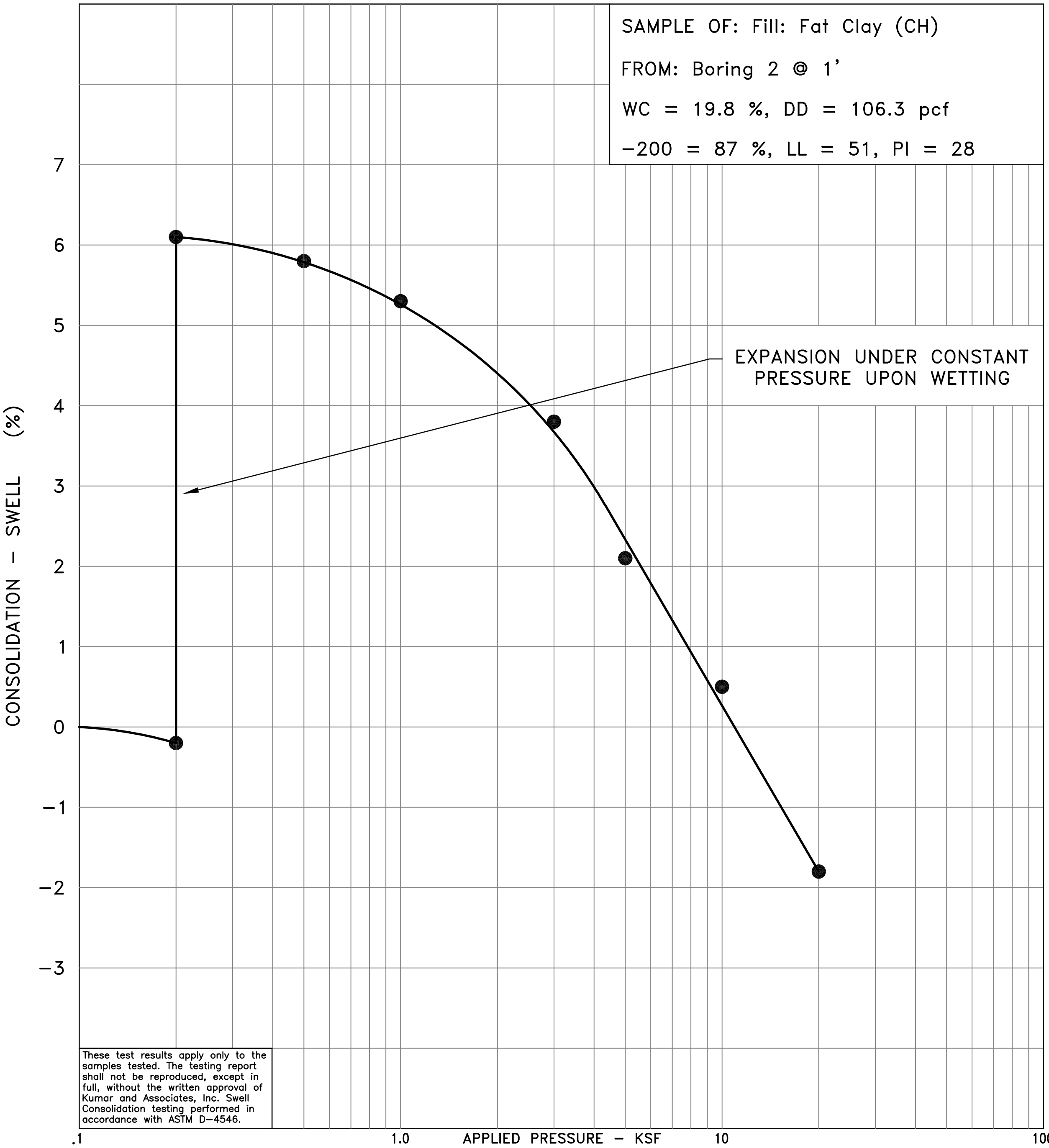

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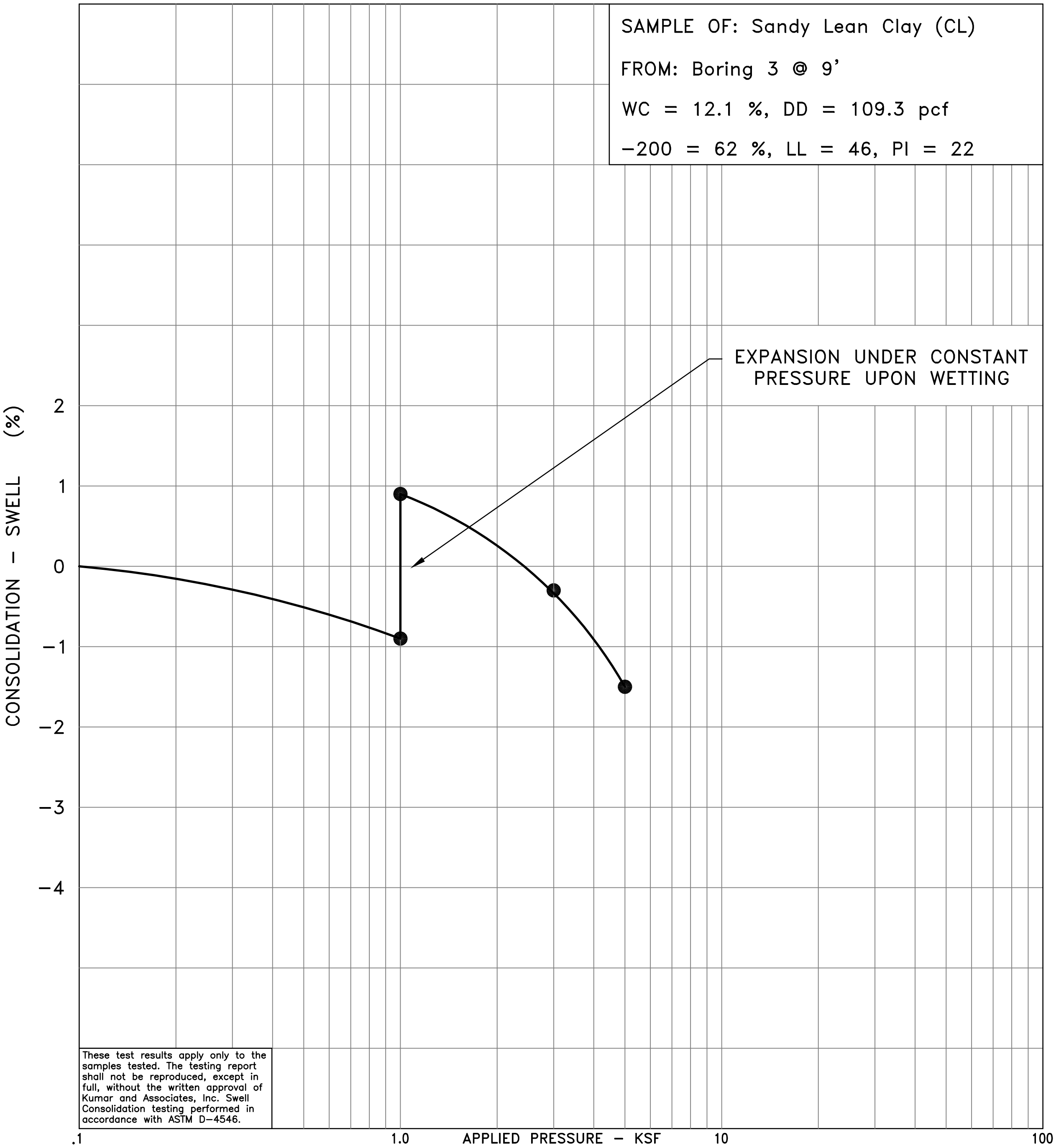
20-1-336	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULT	Fig. 3



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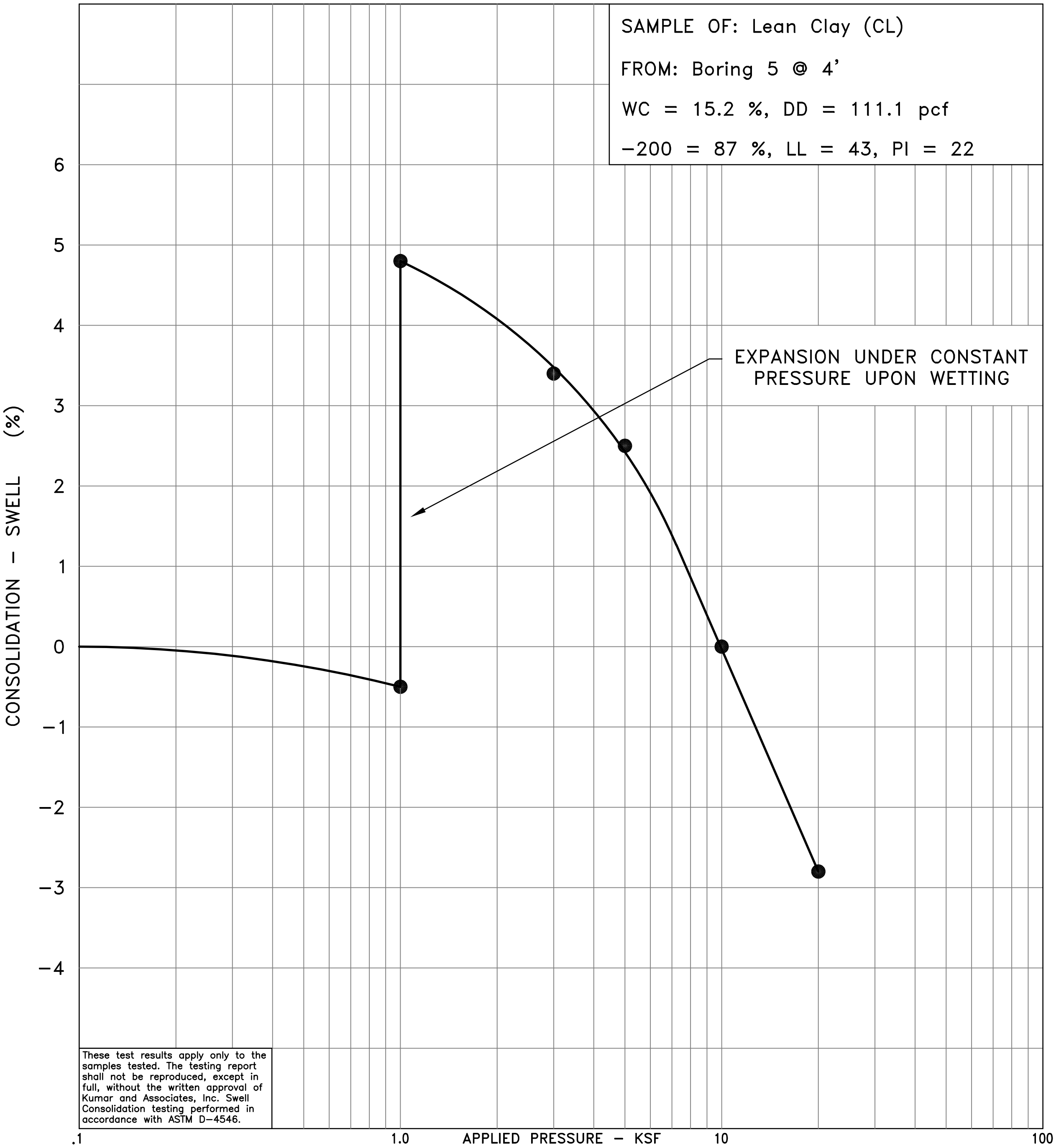
20−1−336	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULT	Fig. 4



These test results apply only to the samples tested. The testing report shall not be reproduced, except in full, without the written approval of Kumar and Associates, Inc. Swell Consolidation testing performed in accordance with ASTM D−4546.

20−1−336	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULT	Fig. 5

TABLE I
SUMMARY OF LABORATORY TEST RESULTS

PROJECT NO.: 20-1-336
PROJECT NAME: Jordan Road & Main Street, Circle K
DATE SAMPLED: 5-19-2020
DATE RECEIVED: 5-20-2020

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	9	5-20-20	13.1	112.5			67			0.45		Sandy Lean Clay (CL)
1	14	5-20-20	3.5	110.0	17	74	9					Poorly-Graded Sand with Clay and Gravel (SP-SC)
2	1	5-20-20	19.8	106.3			87	51	28		A-7-6 (26)	Fill: Fat Clay (CH)
2	9	5-20-20	8.3	118.4			39					Clayey Sand (SC)
3	4	5-20-20	11.8	112.0	7	32	61			0.01		Sandy Lean Clay (CL)
3	9	5-20-20	12.1	109.3			62	46	22		A-7-6 (12)	Sandy Lean Clay (CL)
4	1	5-20-20	18.4	109.1			83					Fill: Lean Clay with Sand (CL)
4	14	5-20-20	14.3	113.1			72					Lean Clay with Sand (CL)
5	4	5-20-20	15.2	111.1			87	43	22		A-7-6 (20)	Lean Clay (CL)
5	19	5-20-20	16.9	111.9	3	36	61					Sandy Lean Clay (CL)