Kumar & Associates, Inc.®
Geotechnical and Materials Engineers
and Environmental Scientists

2390 South Lipan Street
Denver, CO 80223
phone: (303) 742-9700
fax: (303) 742-9666
email: kadenver@kumarusa.com
www.kumarusa.com

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Office Locations: Denver (HQ), Parker, Colorado Springs, Fort Collins, Glenwood Springs, and Summit County, Colorado

GEOTECHNICAL ENGINEERING STUDY
PROPOSED HOSPITAL TOWER ADDITION
ADVENTHEALTH PARKER
9395 CROWN CREST BOULEVARD
PARKER, COLORADO

Prepared By:

Kamal M. Hasan, E.I.

Reviewed By:

Wade Gilbert, P.E.

COLORADO REGISTERED
JAMES WADE GILBERT
39256
3/28/24
PROFESSIONAL ENGINEER

Prepared For:

AdventHealth Parker
9395 Crown Crest Boulevard
Parker, Colorado

Attention: Mr. Mike Scarlett

Project No. 23-1-737

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TABLE OF CONTENTS

SUMMARY

1. A field exploration program consisting of drilling seven (7) exploratory borings was conducted to obtain information on subsurface conditions. Six borings (Borings 1 through 6) were drilled within the area of the proposed building addition, and one boring (H-1) was drilled within the area of the proposed replacement helipad.

   Borings 1, 3, 5, and H-1 encountered an asphalt pavement section with thicknesses ranging from about 6 to 10 inches, Boring 2 encountered a concrete section with a thickness of about 8 inches, and Borings 4 and 6 encountered a thin layer of topsoil. The subsurface conditions encountered in the borings below the surficial materials consisted of man-placed fill extending to natural soils or bedrock at depths ranging from about 6 to 27 feet. Natural soil was encountered in Borings 1, 2, 5, and 6 below the pre-existing fill and extended to bedrock at depths ranging from about 17.5 feet to 32.5 feet. Building borings were terminated in bedrock at depths ranging from about 45 to 55 feet. Boring H-1 was terminated in natural soil at a depth of about 10 feet.

   Groundwater was encountered in Borings 1 through 6 during drilling at depths ranging from about 27 to 49 feet, and Boring H-1 was found to be dry. Borings 2 through 5 were left open for 2 days to allow for measurement of stabilized groundwater levels, and Borings 1, 6, and H-1 were backfilled and surface patched after completion of drilling. Stabilized groundwater was measured in Borings 3 and 5 at depths of about 24 and 28.5 feet, respectively. After these measurements, the borings were backfilled and patched to match the existing surface.

2. Based on the subsurface conditions encountered in the borings, anticipated foundation loads, and the presence of deep foundations of adjacent buildings, a deep foundation system consisting of straight shaft drilled piers is recommended. Drilled piers should have a minimum of 3 pier diameters or 5 feet of competent bedrock embedment, whichever is greater. Piers with the minimum embedment should be designed for an allowable unit end bearing pressure of 30,000 psf, and piers with a minimum bedrock embedment of 15 feet should be designed for an allowable unit end bearing pressure of 42,500 psf. Piers may also be designed for a unit allowable compressive skin friction of 3,000 psf for the portion of the pier embedded less than 15 feet into bedrock and 4,500 psf for the portion embedded more than 15 feet into bedrock. Uplift due to structural loadings on the piers can be resisted using 75% of the allowable unit skin friction plus an allowance for pier weight. Side friction should be ignored in the upper 2 feet of bedrock penetration.

   Shallow foundations consisting of spread footings are considered feasible for more lightly loaded structures and equipment, recommendations for shallow foundations are presented herein.

3. Soil-supported floor slabs are considered feasible for the proposed construction. Soil-supported floor slabs and movement-sensitive flatwork recommendations are presented herein.

4. Recommendations for the proposed helipad and incidental pavements are presented herein. Pavement section alternatives based on the anticipated traffic volume, on-site material properties, and local industry standards of practice are presented on the following page:

LOCATION	Full Depth Asphalt Pavement (inches)	Asphalt Over Aggregate Base Course (inches)	PCCP (inches)
Standard Duty	6.0	4.0 over 8.0	6.0
Heavy Duty	7.0	5.0 over 8.0	7.0

PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical engineering study for the proposed hospital tower addition to be constructed at AdventHealth Parker located at 9395 Crown Crest Boulevard 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 revised Proposal No. P-23-862 to AdventHealth Parker on November 28, 2023.

A field exploration program consisting of drilling exploratory borings was conducted to obtain information on subsurface conditions. Representative samples of the onsite 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 addition.

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 the construction of the proposed facility are included in the report.

PROPOSED CONSTRUCTION

Based on the information provided, we understand planning is in the early stages but that the proposed project will consist of a building addition to the existing Parker Adventist Hospital (PAH). The proposed addition will consist of a five-story building over a partial basement located south and east of the east and south wings of the existing hospital building and southwest of the MOB II building. The exact footprint of the building has not been established but will likely have an overall footprint area of around 25,000 square feet. We assume the building will be a steel- or concrete-frame structure with relatively heavy column loads. We understand the addition may be designed for future additional floors.

The project may also include upgrades to portions of the central utility plant (CUP) and relocation of the existing helipad.

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 occupied by lawn and paved areas and the eastern portion of the CUP facility. We anticipate site grading will require minor cuts and fills to establish subgrade elevations. Based on available topographic information, the site is generally level, with a maximum grade change of about 5 feet.

SUBSURFACE CONDITIONS

Field Exploration: The subsurface conditions were explored by drilling 4 exploratory borings at the approximate locations shown on Fig. 1. The borings were advanced through the onsite existing fill, natural soils, and into the underlying bedrock using 4-inch-diameter, continuous-flight augers, and were logged by a representative of Kumar & Associates, Inc. (K+A). Samples of the existing fill, natural soils, and bedrock materials were obtained using a 2-inch-I.D. California-liner sampler 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 the ASTM International (ASTM) D1586 test procedure. The California-liner sampler is used locally to obtain relatively undisturbed soil and bedrock samples. Penetration resistance values (blow counts) indicate the relative density or consistency of the soils and bedrock.

Depths at which samples were taken and the associated blow counts are shown on the Logs of the Exploratory Borings, Figs. 2 and 3. A legend and notes describing the materials encountered are presented on Fig. 4.

K+A previously provided a geotechnical engineering study and pavement thickness design for the construction of a portion of the east wing of the existing PAH building and the northwest parking garage, with the results provided in a report under Project No. 14-1-178, dated April 24, 2014 (2014 Study).

Subsurface Conditions: Borings 1, 3, 5, and H-1 encountered an asphalt pavement section with thicknesses ranging from about 6 to 10 inches, Boring 2 encountered a concrete section with a thickness of about 8 inches, and Borings 4 and 6 encountered a thin layer of topsoil. The subsurface conditions encountered in the borings below the surficial materials consisted of man-placed fill extending to natural soils or bedrock at depths ranging from about 6 to 27 feet. Natural soil was encountered in Borings 1, 2, 5, and 6 below the pre-existing fill extending to bedrock at depths ranging from about 17.5 feet to 32.5 feet. Building borings were terminated in bedrock at depths ranging from about 45 to 55 feet. Boring H-1 was terminated in natural soil at a depth of about 10 feet.

The existing fill encountered in the borings generally consisted of lean clay with occasional fat clay zones and a variable sand and claystone content. The existing fill was very moist and gray to dark gray to brown to dark brown. The lateral and vertical extents of the fill and degree of compaction were not determined as part of this study. Based on the results of the laboratory testing, the in-situ dry unit weights of the existing fill were relatively low.

The natural soils encountered in the borings varied between cohesive and granular. The cohesive soils consisted of lean clay with a variable sand fraction and were moist and dark gray to brown. The natural granular soils consisted of fine- to coarse-grained silty sand to clayey sand and were generally moist to wet (below groundwater) and brown. Based on blow counts, the cohesive soils were generally stiff to very stiff, and the natural granular soils were loose to medium dense.

The bedrock consisted of claystone and sandstone bedrock, and zones of interbedded claystone and sandstone. The claystone bedrock was very moist and gray to brown. The sandstone bedrock was moist to very moist and dark gray to brown. The interbedded claystone and sandstone were very moist and gray to brown. Based on blow counts, the claystone bedrock was hard to very hard, the sandstone bedrock was firm to very hard, and the interbedded claystone and sandstone were medium hard to hard.

<u>Groundwater Conditions</u>: Groundwater was encountered in Borings 1 through 6 during drilling at depths ranging from about 27 to 49 feet, and Boring H-1 was found to be dry. Borings 2 through 5 were left open for 2 days to allow for measurement of stabilized groundwater levels, and Borings 1, 6, and H-1 were backfilled and surface patched after completion of drilling. Stabilized groundwater was measured in Borings 3 and 5 at depths of about 24 and 28.5 feet, respectively. After these measurements, the borings were backfilled and patched to match the existing surface. Based on observed and measured depths, groundwater appears to reside primarily within the bedrock or perched just above bedrock in places.

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, unconfined compressive strength, and swell-consolidation characteristics. These tests were performed in accordance with applicable ASTM test procedures. The percentage of water-soluble sulfates was evaluated in general 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 Figs. 2 and 3, plotted graphically on Figs. 5 through 15, 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 eleven samples of the onsite existing fills and one sample of the claystone bedrock to determine their swell and/or compressibility 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 either 200 psf 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 a maximum surcharge pressure of 3,000 psf, and the sample heights were monitored until deformation practically ceased under each load increment.

Results of the swell-consolidation test are presented on Fig. 5 through 13 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, four samples of the existing fills and the claystone sample exhibited low swell potential, one sample exhibited no movement, and the remaining six samples of the existing fills exhibited additional generally slight compression under the applied surcharge pressure when wetted. For several samples of the fill, significant compression occurred under higher post-wetting surcharge loads, which we believe is due to relatively low unit weights of those fill samples.

Unconfined Compressive Strength: The strength and load-bearing characteristics for two representative samples of the claystone bedrock were evaluated based on the results of unconfined compressive strength (UC) testing. The UC test is a rapid test procedure used to evaluate the undrained shear strength of the tested material. Results from UC testing performed on two samples of the claystone bedrock indicated ultimate unconfined compressive strength, q_u, values of 143.4 psi and 153.7 psi.

GEOTECHNICAL ENGINEERING CONSIDERATIONS

General: The project will consist of the demolition of portions of existing structures, underground levels, if present, pavements, and utilities. Foundations for the addition and cup improvements should be properly located to prevent conflict with the foundations of the existing building and other adjacent structures. Existing structural elements should be removed to a depth of at least three feet below the bottom of slabs, exterior flatwork, and pavement. Void spaces resulting from demolition should be backfilled with structural fill, flowable fill, or low-strength concrete. Any remaining utility conduit openings should be grouted to prevent the accumulation of water.

Presence of Fat Clays: As indicated, the existing fills include fat clays. Fat clay is problematic in that the elevated moisture contents of the clay will make it difficult to stabilize from a proof-roll standpoint prior to paving. Although the samples of fat clay tested exhibited no movement to low swell potential, it will become more expansive if dried during construction and become wetter following placement of structures and pavements. Consequently, fat clay should not be re-used beneath soil-supported slabs and rigid pavements. The contractor should be aware that fat clay may become unstable and difficult to place at the upper end of the recommended moisture content range and should not be used as fill below new structures.

The fat clay soils may be used as general fill in landscape areas or areas not considered movement-sensitive.

On-Site Existing Fill: Up to 27 feet of man-placed fill was encountered in our borings. Although not indicated in our borings, deeper fills may be present across the site and should be anticipated. Without documentation regarding placement and compaction testing, the fill should be considered non-engineered and unsuitable in its current condition for support of foundation elements.

Foundations: Based on conditions encountered in the borings and our understanding of the proposed construction, a deep foundation system is considered the most feasible approach for support of the proposed building addition. Building loads are anticipated to be relatively high, and the site is generally underlain by deep fills of relatively low unit weight and relatively stiff or dense natural soils. Bedrock is relatively deep, and groundwater was encountered at depths ranging from about 24 feet to 48 feet. These conditions, particularly the depth to bedrock, are the primary considerations in the selection of a foundation system at the site.

Deep foundation systems consisting of straight-shaft drilled piers are considered feasible at the site. The advantages of deep foundations extending into competent bedrock include the ability to support heavy foundation loads with limited foundation settlement through relatively high load-bearing capacity in relatively incompressible material. Disadvantages to deep foundations include the considerable depth to competent bedrock, which would result in long foundation lengths to obtain adequate penetration into bedrock, and, for drilled pier foundations, the presence of isolated zones of saturated soils above the bedrock, which may require the use of relatively costly casing and slurry drilling methods in places.

Shallow foundations consisting of spread footings are considered feasible for more lightly loaded structures and equipment, provided the potential for some movement due to settlement and associated possible distress is recognized by the owner. Spread footings should be placed on a zone of structural fill, improved existing fills, or a combination of both.

Soil-supported Slabs: Soil-supported slabs and movement-sensitive exterior flatwork should be feasible in places with ground improvement. Ground improvement alternatives at the site include the removal and replacement of the existing fill or the use of a Load Transfer Platform (LTP) supported on a grid of aggregate piers. While complete removal and replacement of the existing fill would be ideal, the blow counts of the existing fills suggest those fills may have been placed under controlled conditions during original site construction. We believe a partial-depth removal and replacement approach should be suitable and may be considered, provided the owner understands and accepts the risk of post-construction settlement-related movement potentially in excess of project design tolerances. Use of aggregate piers, with or without LTP, would likely reduce that risk significantly.

FOUNDATIONS RECOMMENDATIONS

Drilled Piers: The design and construction criteria presented below should be observed for a straight-shaft pier foundation system. The construction details should be considered when preparing project documents.

1. Piers should have a minimum bedrock embedment equivalent to 3 times the pier diameter or 5 feet, whichever is greater. Total pier lengths are anticipated to range from around 40 to 50 feet.

2. Piers with a minimum bedrock embedment should be designed for an allowable unit end-bearing pressure of 30,000 psf, and piers with a minimum bedrock embedment of 15 feet should be designed for an allowable unit end-bearing pressure of 42,500 psf. Piers should also be designed for a unit compressive skin friction of 3,000 psf for the portion of the pier embedded less than 15 feet into bedrock and 4,500 psf for the portion embedded more than 15 feet into bedrock.

3. Uplift due to structural loadings on the piers can be resisted using 75% of the unit compressive skin friction values plus an allowance for pier weight. Resistance due to side friction can be applied over the full bedrock embedment length, with the exception of the top 2 feet of bedrock penetration. Frictional uplift resistance for the portion of the pier extending through the natural overburden soils may be considered based on an allowable unit side shear at 0.8 ksf applied uniformly regardless of depth.

4. The lateral capacity of the piers may be analyzed using the LPILE computer program and the parameters provided in the following table. The strength criteria provided in the table are for use with that software application only and may not be appropriate for other uses:

Material	c (psi)	ø	g_T	k_s	k_c	e_{50}	Soil Type
New Granular Structural Fill	0	34	0.072	90	90	----	1
Natural Granular Soils Above Groundwater	0	34	0.064	90	90	----	1
Natural Granular Soils Below Groundwater	0	34	0.039	60	60	----	1
Existing Fill/Natural Clay soils	15	0	0.069	500	200	0.007	2
Claystone Bedrock	40	0	0.075	1,500	600	0.005	2

c Cohesion intercept (psi)
ø Angle of internal friction (degrees)
γ_t Total unit weight (pci)
k_s Initial static modulus of horizontal subgrade reaction (pci)
k_c Initial cyclic modulus of horizontal subgrade reaction (pci)
ϵ_{50} Strain at 50% of peak shear strength

Soil Types:
1. Sand (Reese)
2. Stiff Clay without Free Water (Reese)

5. Closely spaced piers will require appropriate reductions of the axial, uplift and lateral capacities based on the effective envelope of the pier group. These reductions can be avoided by spacing the piers at a distance of at least 4 pier diameters center-to-center for axial loading, 5 pier diameters center-to-center in the direction parallel to lateral loading, and 5 pier diameters center-to-center in the direction perpendicular to lateral loading. More closely spaced piers should be studied on an individual basis to determine the appropriate reduction in axial and lateral load design parameters.

If the recommended minimum center-to-center pier spacings for lateral loading cannot be achieved, we recommend the load-displacement curve (p-y curve) for an isolated pier be modified for closely spaced piers using p-multipliers to reduce all the p values on the curve. With this approach, the computed load-carrying capacity of the pier in a group is reduced relative to the isolated pier capacity. The modified p-y curve should then be reentered into the LPILE software to calculate the pier deflection. The reduction in capacity for the leading pier, the pier leading the direction of movement of the group, is less than that for the trailing piers.

For center-to-center pier spacing of piers in the group in the direction of loading expressed in multiples of the pier diameter, we recommend p-multipliers of 0.7 and 1.0 for pier spacing of 3 and 5 diameters, respectively, for the leading row of piers, 0.4 and 0.85 for pier spacing of 3 and 5 diameters, respectively, for the second row of piers, and 0.3 and 0.7 for pier spacing of 3 and 5 diameters, respectively, for rows 3 and higher. For loading in a direction perpendicular to the row of piers, the p-multipliers are 1.0 for a pier spacing of 5 diameters, 0.7 for a pier spacing of 3 diameters, and 0.5 for a pier spacing of 1 diameter. P-multiplier values for other pier spacing values should be determined by interpolation. These values are consistent with Table 10.7.2.4-1 of the 2020 AASHTO LRFD Bridge Design Specifications. It will be necessary to determine the load distribution between the piers that attain deflection compatibility because the leading pier carries a higher proportion of the group load, and the pier cap prevents differential movement between the piers.

Previously installed but currently unloaded piers are present within a portion of the likely footprint area of the addition. If those piers are subsequently loaded, the above reduction criteria, if necessary, will apply to both the existing piers and new piers. If the existing piers are to remain unloaded reduction of the end bearing resistance of the new piers is not required. However, a slight reduction in resistance from side friction may be necessary for new piers depending on the distance between the new and existing piers.

6. Based on the results of our field exploration and laboratory testing programs and our experience with similar, properly constructed drilled pier foundations, we estimate pier settlement will be low. Generally, we estimate the settlement of drilled piers will be less than 1 inch when designed according to the criteria presented herein. The settlement of closely spaced piers will be larger and should be studied on an individual basis.

7. A minimum pier diameter of 18 inches is recommended to facilitate proper cleaning and observation of the pier hole. The pier length-to-diameter ratio should not exceed 30.

8. The drilled shaft contractor should mobilize equipment of sufficient size and operating condition to achieve the required bedrock penetration. Hard to very hard bedrock was encountered at the site. The contractor should keep in mind much harder rock may be encountered during drilling and should be prepared to use the appropriate equipment. Drilling at the site in such zones may require the use of a core barrel or pilot hole to facilitate drilling with the required size auger.

9. The presence of water and saturated natural soils in the exploratory borings in places during and after drilling operations indicates the use of temporary casing and dewatering equipment may be required in places. The contractor should be prepared to address such conditions.

10. Construction with temporary casing can reduce the side shear strength for axial compression and uplift capacity. We assume the Denver slurry method would be utilized if casing is necessary. Successful construction of drilled pier foundations using the slurry displacement method is extremely dependent on the construction procedures.

The Denver slurry method consists of drilling the overburden as a wet hole, placing a temporary casing sealed in the bedrock, pumping the slurry out, and excavating the bedrock below the casing as a dry hole. Oversizing the portion of the hole in the overburden to allow casing insertion can reduce the lateral pier capacity if the hole is processed with a dense, viscous mixture of water and soil (slurry) and that material is not displaced from the outside of the temporary casing during concrete placement.

If slurry processing is required, this may be accomplished by placing controls in the construction specifications on the slurry composition at the time of temporary casing insertion to facilitate slurry displacement from the sides during concrete placement. Federal Highway Administration publication, “Drilled Shafts: Construction Procedures and LRFD Design Methods” (FHWA-NHI-10-016) provides guidance on construction methods and includes guide specifications. The following table provides a summary of the test methods and acceptable ranges for slurry at the time of casing placement:

Property of Slurry (Units)	Requirement	Test Method (API Standard Method)
Density (lb./ft^3)	64.3 to 72	Mud Weight Density Balance (API 13B-1)
Viscosity (sec/quart)	28 to 50	Marsh Funnel and Cup (API 13B-1)
Sand Content prior to casing/concrete placement (percent by Volume)	≤4.0	Sand Content (API 13B-1)

When water and/or drilling slurry is present outside the casing, care should be taken that concrete of sufficiently high slump is placed to a sufficiently high elevation inside the casing to prevent intrusion of water and/or slurry into the concrete when the casing is withdrawn. Ideally, the slurry should be completely displaced outside the casing by maintaining a positive hydraulic head, using the concrete inside the casing until the casing is removed. Mixing of concrete and slurry near the top of the casing should not be allowed.

Without controls in place, excessively thick/stiff slurry can result in less resistance to lateral loads, and reductions to the lateral resistance parameters (LPILE parameters) may be necessary. Alternatively, the lateral support contribution from the cased portion of the pier may be ignored in the design. If lateral resistance is necessary with limited displacement, performance testing of an on-site pier (either sacrificial or production where lateral resistance and/or displacement is not a concern) constructed using the Denver Slurry Method should be considered.

Skin friction should be neglected in the cased portion of the hole within the bedrock. The amount of additional penetration should be determined in the field at the time of construction. The contract documents should advise potential drilled shaft contractors of these subsurface conditions. In addition, careful consideration should be given to preparing bid items to avoid high costs for potential overruns.

11. Although groundwater is present, significant dewatering is not anticipated. However, if water infiltration does occur, the requirements for dewatering can sometimes be reduced by placing concrete immediately upon cleaning and observing the pier hole. If water cannot be removed or prevented using casing and dewatering equipment before placing concrete, the tremie method should be used after the hole has been cleaned. In no case should concrete be placed in more than 3 inches of water unless placed through an approved tremie method.

12. Pier holes should be properly cleaned prior to the placement of concrete.

13. When water is present outside the casing, care should be taken that concrete of sufficiently high slump is placed to a sufficiently high elevation inside the casing to prevent intrusion of the water into the concrete when the casing is withdrawn. The water should be completely displaced outside the casing by maintaining a positive hydraulic head, using the concrete inside the casing until the casing is removed. Mixing of concrete and water near the top of the casing should not be allowed.

14. Concrete used in the piers should be a fluid mix with sufficient slump so it will fill the void between reinforcing steel and the pier hole. We recommend a concrete slump in the range of 5 to 8 inches be used.

15. Concrete should be placed in pier holes the same day they are drilled. The presence of water may require that concrete be placed immediately after the pier hole is completed. Failure to place concrete on the day of drilling will normally result in a requirement for additional bedrock penetration.

16. A representative of the geotechnical engineer should observe pier drilling operations on a full-time basis to assist in the identification of adequate bedrock strata and monitor pier construction.

<u>Spread Footing</u>: Provided the risk of some foundation movement due to settlement is acceptable to the owner, as discussed in the "Geotechnical Engineering Considerations" section of this report, a spread footing foundation system may be used to support more lightly loaded structures.

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. Spread footings should be placed on a zone of structural fill with a minimum thickness of 4 feet or equal to the width of the footing, whichever is greater. Structural fill should extend down and out from the edges of the footings at a 1 (horizontal) to 1 (vertical) projection. Alternatively, footings may be supported on aggregate piers. Aggregate piers are discussed in the "Floor Slabs" section of this report.

2. Footings placed on structural fill as described above should be designed for a net allowable bearing pressure of 3,000 psf. It should be feasible to design footings supported on aggregate piers for higher bearing pressures, as determined by a specialty contractor. The allowable bearing pressure may be increased by one-third for transient loads.

3. Based on experience, we estimate total settlement due solely to structural loads for spread footings designed and constructed as discussed herein will be approximately one inch or less. Differential settlements across the structure are estimated to be approximately 0.5 to 0.75 of the total settlements.

Settlement due to long-term compression of deeper unimproved fills may occur and would be considered in addition to settlement due to structural loads. The risk of such additional settlement would be reduced more effectively using an appropriately designed aggregate pier improvement approach.

4. Spread footings should have a minimum width of 16 inches for continuous footings, 24 inches for isolated pads, and a minimum embedment of 18 inches.

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 0.32. Passive pressure against the sides of the footings may be calculated assuming an equivalent fluid unit weight of 190 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. Care should be taken when excavating the foundations to avoid disturbing the supporting materials of existing foundations. Excavation methods that reduce soil disturbance, such as hand excavation or careful soil removal with a backhoe positioned outside the excavation may be required.

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

FLOOR SLABS

Due to the presence of deep, potentially compressible fills, a structural floor system above a void or crawlspace should be considered, particularly for areas where control of slab movement is critical. A structural floor would be supported on grade beams and piers the same as the main structure.

Design of a crawl space or underfloor void should consider drainage and moisture control. We recommend a minimum 12-inch void beneath floors. Providing a full crawl space (3 feet or more) rather than a void beneath the floor has the advantages that utilities can be suspended above the expansive subgrade and crawl space surface drainage can be provided. Utility lines should not be supported on the subgrade unless adequate measures are taken to account for differential movement between grade-supported utilities and slabs. If utilities are connected to the floor or floor openings, void spaces should also be provided below the utility lines. The utility lines should be supported by suitable means such as hangers as necessary. We recommend that void and crawl spaces be designed with positive surface drainage and a collection point or outlet so that free water introduced into these spaces can be removed.

High humidity can develop in crawl/void spaces due to the transmission of water vapor through moist soils. For basement and/or garden level areas control of humidity in crawl/void spaces is important for indoor air quality and performance of structural floor systems, if used. We believe the current practices to control humidity involve the use of a vapor retarder (10-mil minimum), placed on the exposed soils below accessible sub-floor areas. The vapor retarder should be sealed at joints and attached to concrete foundation elements. The contractor should design for ventilation systems that can be installed in association with a vapor retarder, to improve control of humidity in crawl space areas. We can provide additional information on this issue, if requested.

Slab-On-Grade: As discussed in the “Geotechnical Engineering Considerations” section of this report, slab-on-grade floors underlain by a zone of non-expansive structural fill may be considered. We believe the complete removal and replacement of the existing fills would be impractical and unnecessary. It should be feasible to support floor slabs on a zone of new structural fill extending a few feet below the planned finished floor level (partial removal and replacement alternative) or on existing fill materials improved using aggregate piers. A primary benefit of using an aggregate pier approach would be a significant reduction in the amount and cost of site earthwork, particularly disposal costs of exported existing fill.

Floor slabs supported on a zone of replacement fill would be considered at a somewhat higher risk of long-term, post-construction settlement than a system supported on aggregate piers. However, the risk of post-construction, settlement-related movement should be very low for either approach.

For a slab-on-grade approach, the following measures should be considered to mitigate or reduce slab movement and reduce the potential for damage should the underlying soils be subjected to excessive moisture changes:

1. For a partial removal and replacement approach, we recommend supporting floor slabs on a minimum of 4 feet of structural fill. Alternatively, floor slabs may be underlain by a relatively thin layer of structural fill, generally referred to as a Load Transfer Platform (LTP), blanketing a grid of aggregate piers. For the aggregate pier approach, the need or thickness of an LTP layer should be evaluated by the aggregate pier specialty contractor and the structural engineer. Pier spacing for slab support would depend on the thickness and resulting stiffness of the LTP, if included, reinforced or not. Structural fill should meet the material and placement requirements presented in the "Site Grading and Earthwork" section of this report.

2. A modulus of vertical subgrade reaction of 125 pci is recommended for a subgrade consisting of structural fill or an LTP. The modulus of vertical subgrade reaction of 125 pci was correlated based on material classification and blow count values from several engineering resources, including Joseph E. Bowles Foundation Analysis and Design.

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

4. Interior non-bearing 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 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 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 shallow foundations and construct the slabs independently of the wall foundation. If slab-bearing partition walls are required, distress may be reduced by connecting the partition walls to the exterior walls using slip channels.

Floor slabs should not extend beneath exterior doors or over foundation grade beams 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 depends 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 joint spacing and slab reinforcement should be established by the designer based on experience and the intended slab use.

6. 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.

7. 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 or the slabs are subjected to loads significantly greater than typical slab loads. However, the precautions should reduce distress or damage if such movement occurs.

Aggregate Piers: Ground improvement using aggregate piers is useful for providing a stiffer subgrade and higher soil modulus values and reducing settlements. Aggregate piers are typically constructed by auguring or vibrating 12- to 30-inch-diameter holes to prescribed depths below foundation subgrade and backfilling the holes with lifts of highly compacted, high-strength aggregate materials. Compaction both densifies the aggregate and increases lateral stress in the surrounding soils, resulting in a foundation subgrade with a stiffer composite soil matrix. The auger method will result in spoils that will need management; the vibratory displacement process produces little to no spoils as the displaced soils are densified in place.

Aggregate piers are proprietary systems, and the specialty contractor who installs the piers typically provides the design, including the stiffness of the soil matrix, bearing capacity, and settlement criteria. The pier contractor should submit the details of their proposed system, including capacity determination, pier group sizes, pier spacing, and proposed installation depths.

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 Thickness 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, although not anticipated based on the conditions encountered in our borings, upward movement of exterior flatwork may restrict movement of outward swinging doors. Site grading and drainage design should consider those possibilities, particularly at entryways.

LATERAL EARTH PRESSURES

Walls or retaining structures should be designed for the lateral earth pressure generated by the backfill materials, which is a function of the degree of rigidity of the wall or retaining structure and the type of backfill material used. Walls or retaining structures that are laterally supported and can be expected to undergo only a moderate amount of deflection should be designed for an at-rest lateral earth pressure based on the following equivalent fluid densities:

Backfill Materials	Equivalent Fluid Densities Above Groundwater (pcf)
Imported free-draining granular backfill (< 5% passing No. 200 sieve)	45
CDOT Class 1 (<20% passing No. 200 Sieve)	55
On-site silty to clayey sand	60
On-site or imported, moisture-conditioned clay backfills*	65

* Swell potential less than 0.5%

Cantilevered retaining structures that can be expected to deflect sufficiently to mobilize the full active earth pressure condition should be designed for the following equivalent fluid densities:

Backfill Materials	Equivalent Fluid Densities Above Groundwater (pcf)
Imported free-draining granular backfill (< 5% passing No. 200 sieve)	40
CDOT Class 1 (<20% passing No. 200 Sieve)	45
On-site silty to clayey sand	50
On-site or imported, moisture-conditioned clay backfills*	55

* Swell potential less than 0.5%

Care should be taken not to over-compact the backfill since this could cause excessive lateral pressure on the structure. Hand compaction procedures, if necessary, should be used to prevent lateral pressures from exceeding the design values.

All wall or retaining structures should also be designed for appropriate surcharge pressures such as traffic, construction materials, and equipment. We recommend calculating design lateral pressures due to surcharge loads using a lateral earth pressure coefficient of 0.6.

The zone of backfill placed behind a wall or retaining structure should be sloped upward from the base of the structure at an angle no steeper than 45 degrees measured from horizontal.

SEISMIC DESIGN CRITERIA

The soil profile across the site after construction will generally consist of relatively stiff to very stiff/ loose to medium dense overburden soils extending to medium hard to very hard bedrock at depths ranging from 17.5 feet to 32.5 feet below the finished ground surface. The overburden soils classify as International Building Code (IBC) Site Class D, and the bedrock should classify as Site Class C.

In absence of measured shear wave velocities specific to the existing hospital building that would support a higher Site Class, we recommend IBC Site Class D be used for design in accordance with the International Building Code (IBC). Considering the subsurface profile and site seismicity, liquefaction is not a design consideration.

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 prepared 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 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 handicapped access points in accordance with the Americans with Disabilities Act.

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

4. Exterior backfill should be adjusted to near optimum moisture content (generally within 2 percentage points of optimum unless indicated otherwise in this report) and compacted to at least 95% of the standard Proctor (ASTM D698) maximum dry density.

5. Ponding of water should not be allowed in backfill material or in a zone within 20 feet of new structures or movement-sensitive flatwork during and following construction.

6. Landscaping which requires relatively heavy irrigation and lawn sprinkler heads should be located a minimum of 10 feet from new structures or movement-sensitive flatwork. Use of drip irrigation lines with limited irrigation quantities is generally acceptable within 10 feet of that zone, provided the main lines are located outside that zone.

7. Roof downspouts and drains should discharge well beyond the limits of all backfill or directly to site stormwater collection systems.

SITE GRADING AND EARTHWORK

Demolition: Site development may require demolishing portions of existing structures and pavements and possibly other miscellaneous at-grade and buried structures, as well as the abandonment and/or removal of subsurface utilities. Ideally, all at-grade and below-ground structural elements and utility lines should be removed. However, some deep foundation elements and underground level slabs, if present, may be left in place provided they are separated from the bottom of new foundations, soil-supported slabs, and pavements by a zone of structural fill. Buried utility lines may be abandoned in place provided they are filled with grout and are of sufficient depth to not interfere with the construction and/or performance of new utilities, foundations, soil-supported slabs, and pavements.

Site Preparation: The existing fills across the site should be considered non-engineered and unsuitable in their current condition for support of floor slabs and settlement-sensitive exterior flatwork and/or hardscape areas. Accordingly, at least 4 feet of the existing fills should be removed from beneath soil-supported floor slabs and movement-sensitive flatwork or/or hardscape. Alternatively, floor slabs and movement-sensitive exterior flatwork and/or hardscape can be supported on existing fills improved using aggregate piers. If aggregate piers are not used, over-excavated areas should be prepared and backfilled as recommended in specific sections of this report, with structural fill meeting the material and compaction criteria presented in this section.

Temporary Excavations: We assume site excavations will be constructed by generally over-excavating the side slopes to a stable configuration where enough space is available. Care should be taken when excavating the foundations to avoid disturbing the supporting materials of the existing foundation of the existing historic building. Excavation methods that reduce soil disturbance, such as hand excavation or careful soil removal with a backhoe positioned outside the excavation, may be required.

Where insufficient lateral space is available due to the proximity to property boundaries, existing facilities, and traffic areas, temporary shoring may be required. It is our experience 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 facilities or traffic areas should be sufficiently stiff to prevent movement.

All excavations should be constructed in accordance with OSHA requirements, as well as state, local, and other applicable requirements. Site excavations will generally encounter fill and natural soils. The on-site existing fills and natural granular soil will classify as OSHA Type C soils. The natural clay soils should classify as OSHA Type B soils, and bedrock should generally classify as Type A soils. Fractured and weathered bedrock may also classify as Type B soils, depending on the level of fracturing. If localized perched water or seepage is encountered, much flatter side slopes than those allowed by OSHA, or temporary shoring may be required.

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.

<u>Fill Material Specifications</u>: The following material specifications are presented for fills on the project site. We believe the onsite fill materials and natural soils, minus fat clay, are suitable for reuse as structural fill. We recommend the structural fill used beneath slabs-on-grade and movement-sensitive flatwork consist of imported materials meeting the structural fill criteria presented below:

1. *Structural Fill*: Structural fill generally may consist of on-site overburden soil, including processed existing fill, meeting the criteria of Item 5 below. Imported structural fill material, if necessary, should be non-expansive and have a maximum of 65 percent passing the No. 200 sieve, a maximum liquid limit of 30, and a maximum plasticity index of 15. Imported fill materials not meeting the above liquid limit and plasticity index criteria may be acceptable provided the maximum percentage passing the No. 200 sieve specified above and the non-expansive criteria outlined in Item 5 are satisfied.

   If a grid-reinforced LTP is used to bridge the aggregate piers beneath floor slabs, the material should consist of a coarser granular material with a gradation curve appropriate to the grid aperture size.

2. *Site Grading Fill*: Site grading fill used beneath exterior flatwork and/or hardscape not considered sensitive to movement should consist of suitable moisture-conditioned on-site soils or low-swelling imported soil materials. Imported fill materials for general site grading should have a maximum of 70% passing the No. 200 sieve, a maximum liquid limit of 35, and a maximum plasticity index of 15. Imported or on-site fill materials not meeting the above liquid limit and plasticity index criteria may be acceptable, provided the maximum percentage passing the No. 200 sieve and the swell criteria outlined in Item 5 below are satisfied.

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

4. *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.

5. *Material Suitability*: Unless otherwise defined herein, all fill materials should be non- to low-swelling, free of vegetation, brush, sod, trash and debris, claystone, 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 under a 200 psf surcharge pressure does not exceed 0.5 percent when a sample remolded to 95 percent of the standard Proctor (ASTM D698) maximum dry density at optimum moisture content is wetted. A fill material should be considered low swelling if the swell potential under a 200 psf surcharge pressure does not exceed 1.0 percent when a sample remolded to 95 percent of the standard Proctor (ASTM D698) maximum dry density at optimum moisture content is wetted. The geotechnical engineer should evaluate the stability of the proposed fill materials prior to placement.

We recommend the following fill placement criteria be used on the project:

1. *Moisture Content*: Fill materials should be compacted at moisture contents within 2 percentage points of the standard Proctor (ASTM D698) optimum moisture content for predominantly granular materials and between 0 and +3 percentage points of optimum for predominantly clay materials. The contractor should be aware that the clay soils, if used, 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*: New fill should be placed in maximum 8-inch-thick lifts. Wall backfill may be placed in maximum 12-inch-thick lifts provided proper compaction can be achieved.

   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)
Below Shallow Foundations, Floor Slabs, Flatwork, and Hardscape	95%
Utility Trenches, Exterior Flatwork, and Pavements	
Fill less than 8 feet deep	95%
Fill placed deeper than 8 feet	98%
Landscape and Other Areas	95%

3. *Fill Subgrade Preparation*: Areas receiving new fill should be scarified to a depth of at least 12 inches and compacted to at least 95% of the standard Proctor (ASTM D698) maximum dry density at 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 to suitable soils and replaced with site grading or structural fill, whichever is applicable.

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.

WATER-SOLUBLE SULFATES

The concentrations of water-soluble sulfates measured in representative samples of the onsite existing fill and claystone bedrock ranged from 0.01% to 0.13%. These concentrations represent Class S0 and Class S1 severities of exposure to sulfate attack on concrete exposed to these materials. The degrees of attack are 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.

Based on the laboratory data and our experience, we recommend all concrete exposed to the existing fill materials meet the cement requirements for Class S1 exposure as presented in ACI 201.2R. Alternatively, the concrete could meet the Colorado Department of Transportation's (CDOT) cement requirements for Class 1 exposure as presented in the most recent version of the CDOT Standard Specifications for Road and Bridge Construction. Special sulfate resistance should not be required for concrete in contact with the natural granular soils.

PAVEMENT THICKNESS DESIGN

Based on the similarities of subsurface conditions encountered in our 2014 Study and in this study, we recommend the traffic loading design, pavement thickness design, subgrade preparation, and drainage conform to our recommendations presented in the 2014 Study.

<u>Subgrade Materials</u>: Based on the results of the field exploration and laboratory testing programs, the soils anticipated to be at or near the pavement subgrade generally classify as A-7-6 with group indices between 0 and 19 in accordance with the AASHTO soil classification system. Soils classifying as A-7-6 are generally considered to provide poor subgrade support. For design purposes, a resilient modulus value of 3,025 psi was selected for flexible pavements, and a climate adjusted modulus of subgrade reaction of 34 pci was selected for rigid pavements.

<u>Design Traffic</u>: 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: 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. For design purposes, a design reliability of 80% was assumed for all pavement areas. 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	6.0	4.0 over 8.0	6.0
Heavy Duty	7.0	5.0 over 8.0	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. The PCCP thicknesses presented above are for unreinforced sections.

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 5 or Class 6 ABC stated in the current CDOT Specifications. The ABC should be compacted to a minimum of 95% of the modified Proctor (ASTM D1557) maximum dry density at a moisture content within 2% of the optimum.

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. Sealing of the joints and installation of tie-bars, where necessary, should be in accordance with the latest CDOT M&S Standards.

Subgrade Preparation: Subgrade preparation below pavements should consist of thorough scarification and well-mixing to a minimum depth of 12 inches for flexible pavement and 24 inches for PCCP pavement, and moisture-conditioning and recompacting the material in accordance with the "Site Grading and Earthwork" section of this report.

Pavement design procedures assume a stable subgrade. Prior to placing the pavement section, the pavement subgrade should be proof-rolled with a heavily loaded pneumatic-tired vehicle with a tire pressure of at least 100 psi capable of applying a minimum load of 18-kips per axle. 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 are extremely important to the satisfactory performance of the pavement. Drainage design should provide for the removal of water from paved areas and prevent the wetting of the subgrade soils.

Helipad Support: The subsurface conditions encountered in the boring drilled within the proposed helipad footprint area, Boring H-1, consisted of about 6 feet of man-placed fill underlain by natural clay soil to the maximum drilled depth of about 10 feet. Based on the subsurface conditions encountered in H-1 and the expected traffic loading applied by the emergency vehicles, we believe the proposed helipad can be a reinforced soil-supported concrete slab with a minimum thickness of 6 inches. The slab thickness, reinforcement size, and rebar spacing should be designed by a structural engineer depending on the future loads applied to the helipad. We recommend the helipad subgrade be prepared similar to PCCP 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 perform additional studies, if necessary, to accommodate possible changes in the proposed construction.

We recommend 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 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

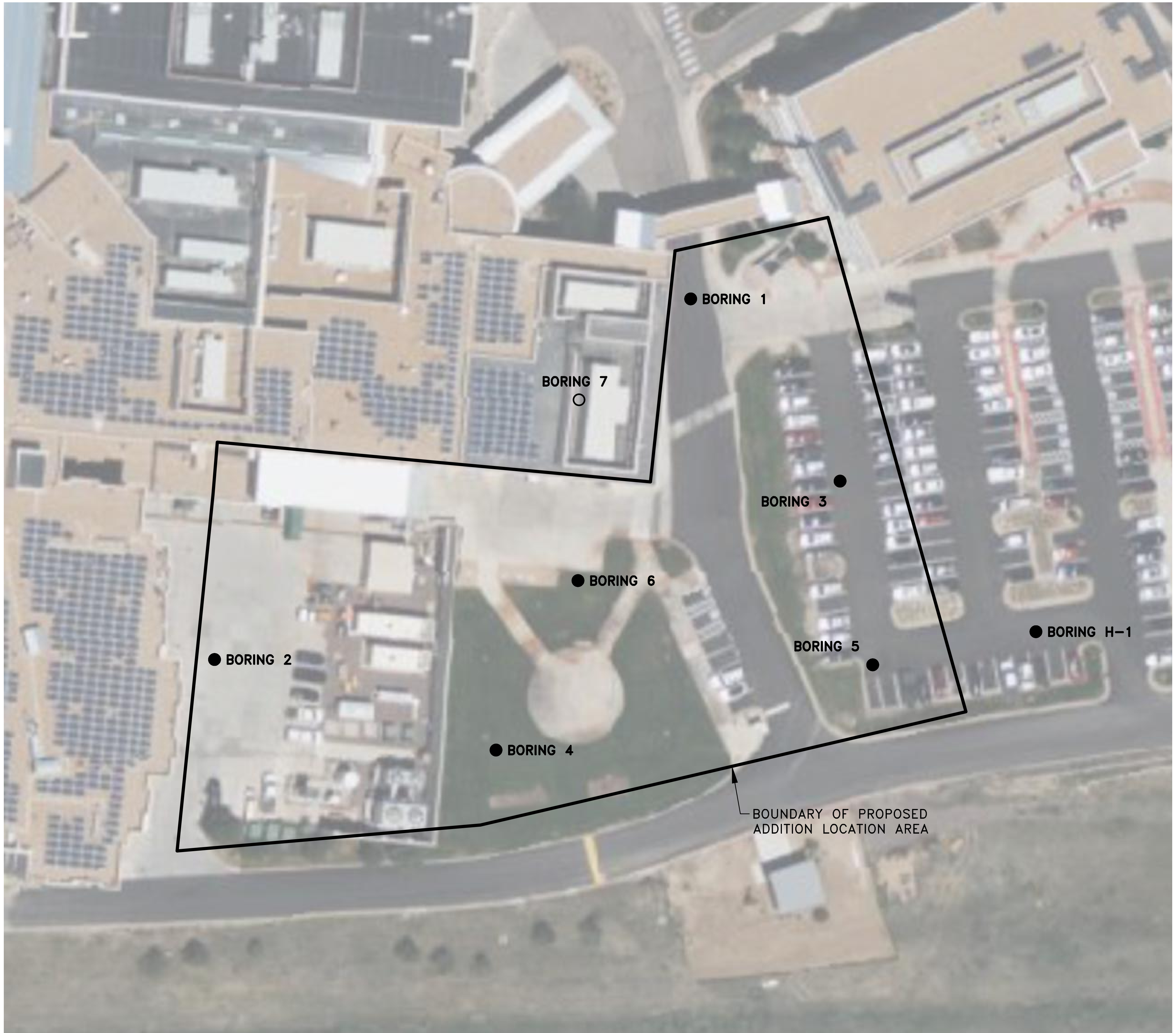
The conclusions and recommendations submitted in this report are based upon the data obtained from the exploratory borings at the locations indicated on Fig. 1, data from previous site studies, and the proposed type of construction. This report may not reflect subsurface variations that occur between the exploratory borings, 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.

The scope of services for this project does not include any environmental assessment of the site or identification of contaminated or hazardous materials or conditions. If the owner is concerned about the potential for such contamination, other studies should be undertaken.

Swelling and compressible soils occur on this site. Such soils are stable at their natural moisture content but will undergo moderate to potentially high volume changes with changes in moisture content. The extent and amount of perched water beneath the 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 and compressible soils 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 and bedrock material 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.

KMH/ad
Rev. by: JWG
Enclosures
cc: file



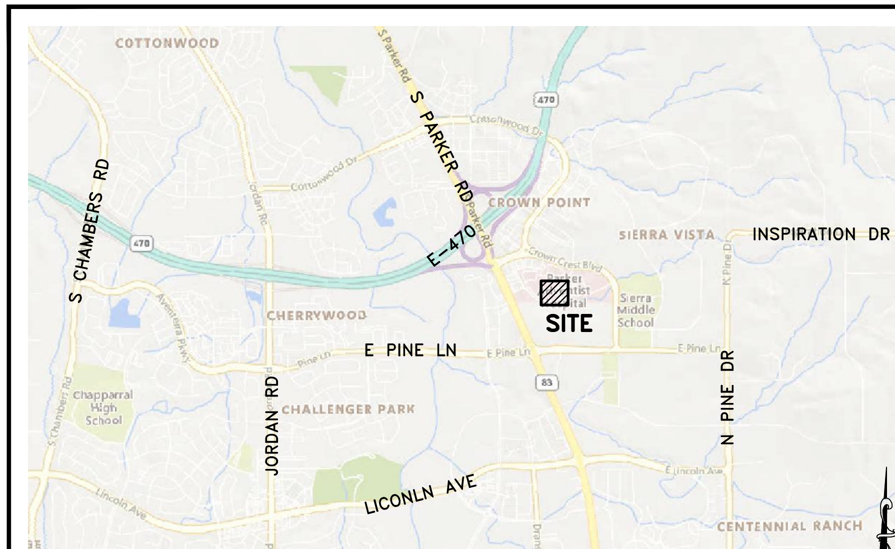
30 0 30 60

APPROXIMATE SCALE–FEET

LEGEND:

● CURRENT BORING

○ BORING DRILLED FOR KUMAR & ASSOCIATES PROJECT NO. 14–1–178.



VICINITY MAP
NOT TO SCALE

23–1–737	Kumar & Associates	HOSPITAL TOWER ADDITION, ADVENT HEALTH PARKER CAMPUS – 9395 CROWN CREST BOULEVARD, PARKER, COLORADO	LOCATION OF EXPLORATORY BORINGS	Fig. 1

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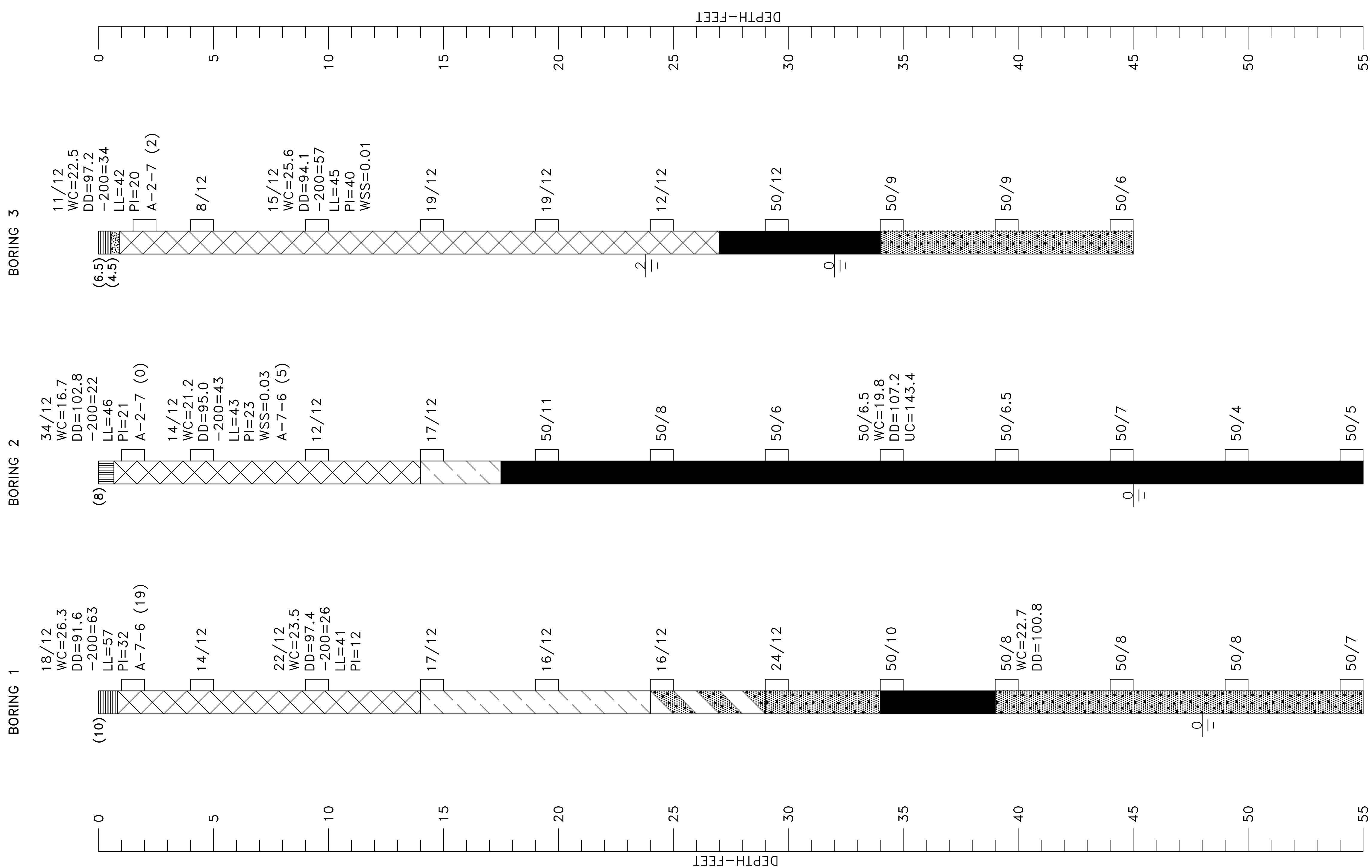

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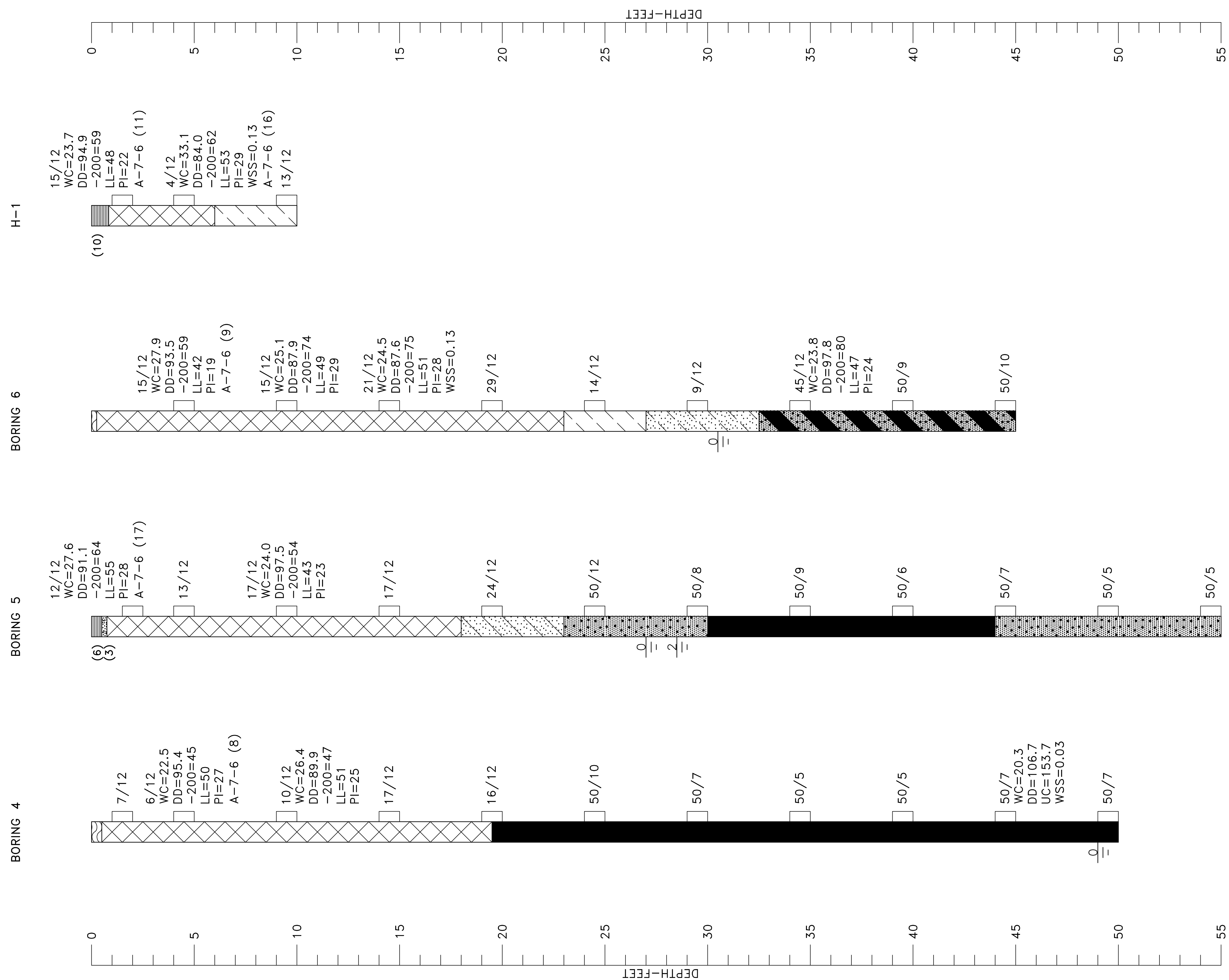
23–1–737	Kumar & Associates	HOSPITAL TOWER ADDITION, ADVENT HEALTH PARKER CAMPUS – 9395 CROWN CREST BOULEVARD, PARKER, COLORADO	LOGS OF EXPLORATORY BORINGS	Fig. 2



23-1-737	Kumar & Associates	HOSPITAL TOWER ADDITION, ADVENT HEALTH PARKER CAMPUS – 9395 CROWN CREST BOULEVARD, PARKER, COLORADO	LOGS OF EXPLORATORY BORINGS	Fig. 3

LEGEND

(10) ASPHALT, THICKNESS IN INCHES SHOWN IN PARENTHESES TO LEFT OF THE LOG.

(8) CONCRETE, THICKNESS IN INCHES SHOWN IN PARENTHESES TO LEFT OF THE LOG.

TOPSOIL.

(4.5) BASE COURSE, THICKNESS IN INCHES SHOWN IN PARENTHESES TO LEFT OF THE LOG.

FILL: LEAN CLAY (CL) AND OCCASIONAL FAT CLAY (CH) ZONES WITH VARIABLE SAND AND CLAYSTONE CONTENT, VERY MOIST, GRAY TO DARK GRAY TO BROWN TO DARK BROWN.

LEAN CLAY (CL) WITH VARIABLE SAND CONTENT, STIFF TO VERY STIFF, MOIST, DARK GRAY TO BROWN.

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

CLAYEY SAND (SC), FINE– TO COARSE–GRAINED, LOOSE, MOIST TO WET (BELOW GROUNDWATER), BROWN.

WEATHERED SANDSTONE BEDROCK, MOIST, BROWN.

SANDSTONE BEDROCK, FIRM TO VERY HARD, MOIST TO VERY MOIST, DARK GRAY TO BROWN.

CLAYSTONE BEDROCK, HARD TO VERY HARD, VERY MOIST, GRAY TO BROWN.

INTERBEDDED CLAYSTONE SANDSTONE BEDROCK, MEDIUM HARD TO HARD, VERY MOIST, GRAY TO BROWN.

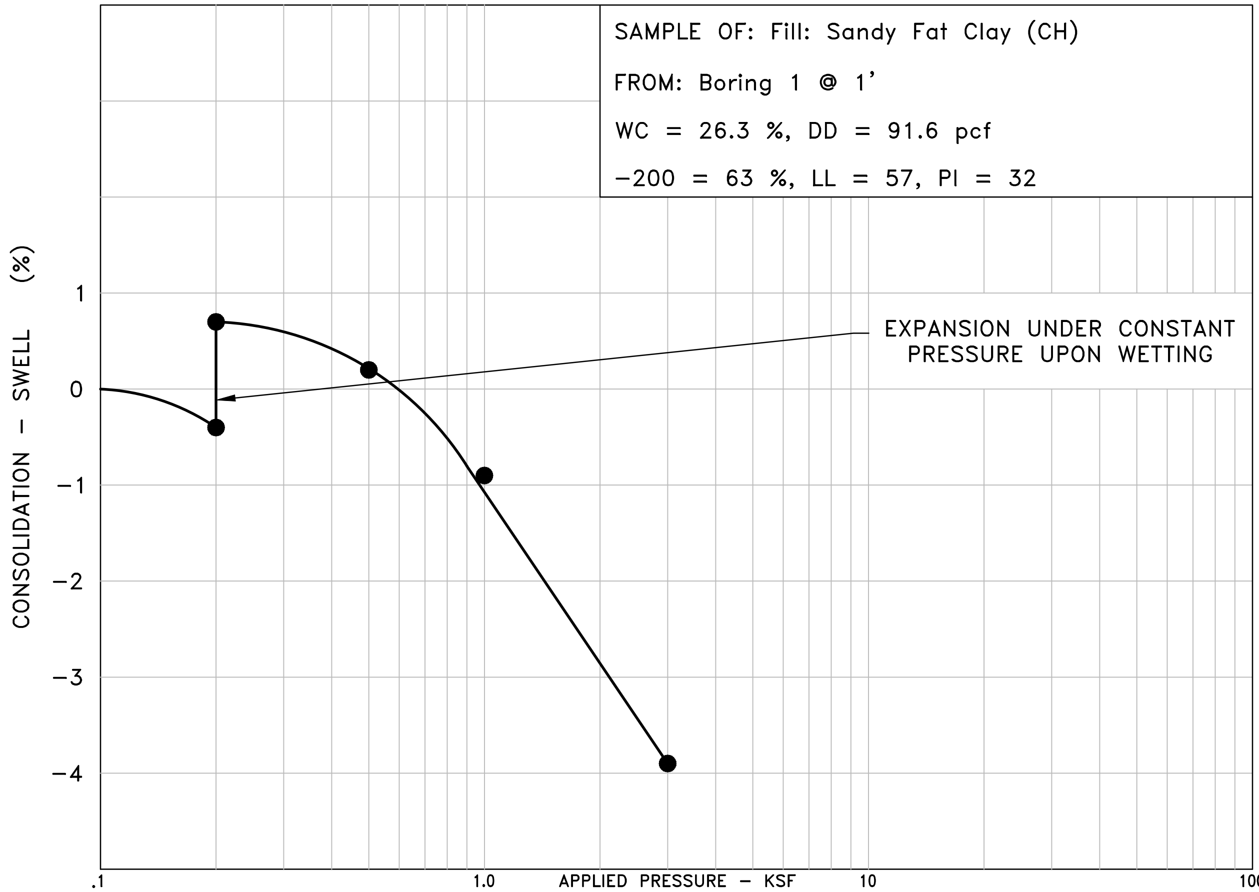
DRIVE SAMPLE, 2–INCH I.D. CALIFORNIA LINER SAMPLE.

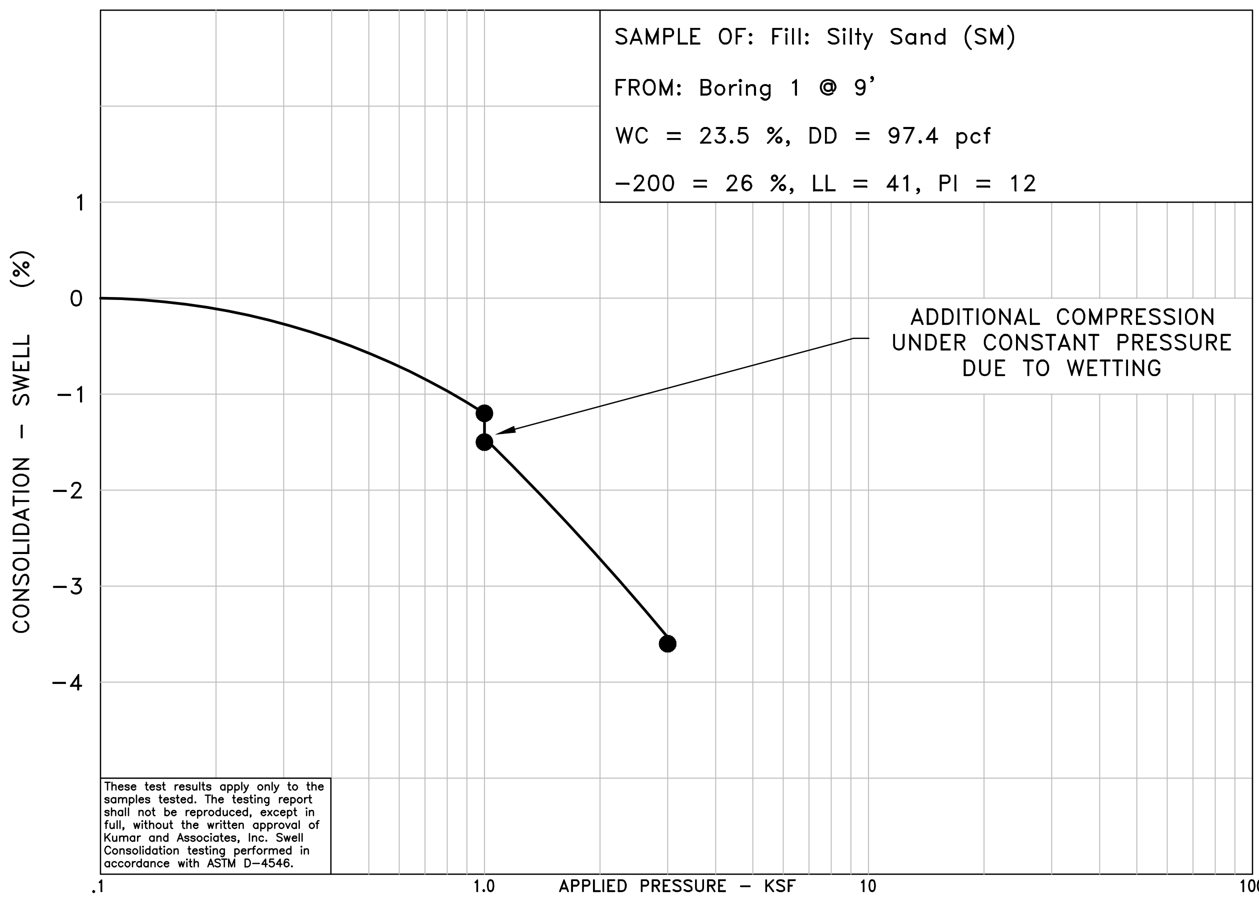
18/12 DRIVE SAMPLE BLOW COUNT. INDICATES THAT 18 BLOWS OF A 140–POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE THE SAMPLER 12 INCHES.

2 DEPTH TO WATER LEVEL AND NUMBER OF DAYS AFTER DRILLING MEASUREMENT WAS MADE.

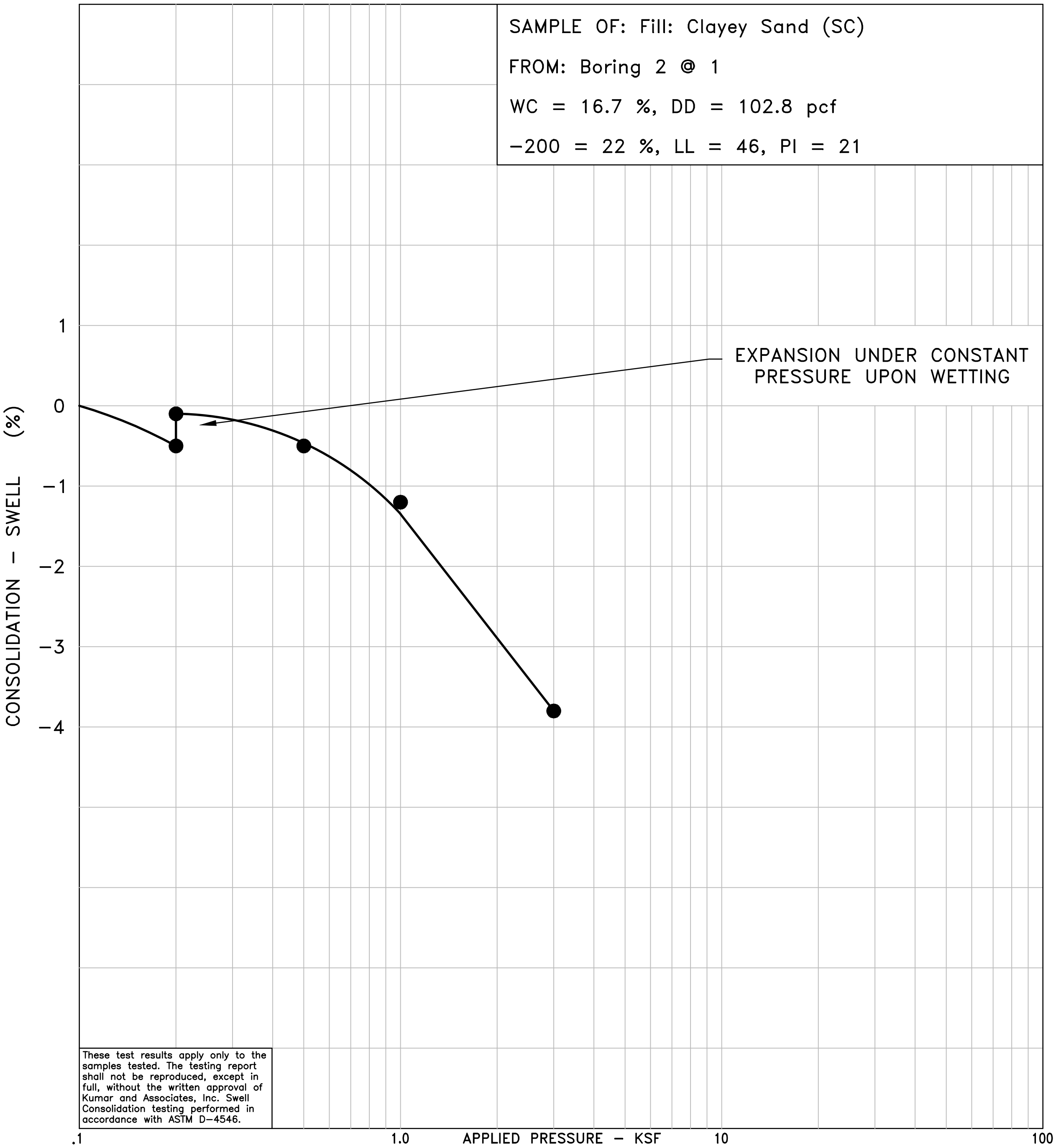
NOTES

1. THE EXPLORATORY BORINGS WERE DRILLED BETWEEN DECEMBER 1 AND 15, 2023 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 LEVELS SHOWN ON THE LOGS WERE MEASURED AT THE TIME AND UNDER CONDITIONS INDICATED. FLUCTUATIONS IN THE WATER LEVEL MAY OCCUR WITH TIME.
7. LABORATORY TEST RESULTS:
   WC = WATER CONTENT (%) (ASTM D2216);
   DD = DRY DENSITY (pcf) (ASTM D2216);
   –200= PERCENTAGE PASSING NO. 200 SIEVE (ASTM D1140);
   LL = LIQUID LIMIT (ASTM D4318);
   PI = PLASTICITY INDEX (ASTM D4318);
   WSS = WATER SOLUBLE SULFATES (%) (CP–L 2103);
   UC = UNCONFINED COMPRESSIVE STRENGTH (psi) (ASTM D 2166);
   A–7–6 (16) = AASHTO CLASSIFICATION (GROUP INDEX) (AASHTO M 145).





23−1−737	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULTS	Fig. 5



23−1−737	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULT	Fig. 6

February 02, 2024 − 01:27pm
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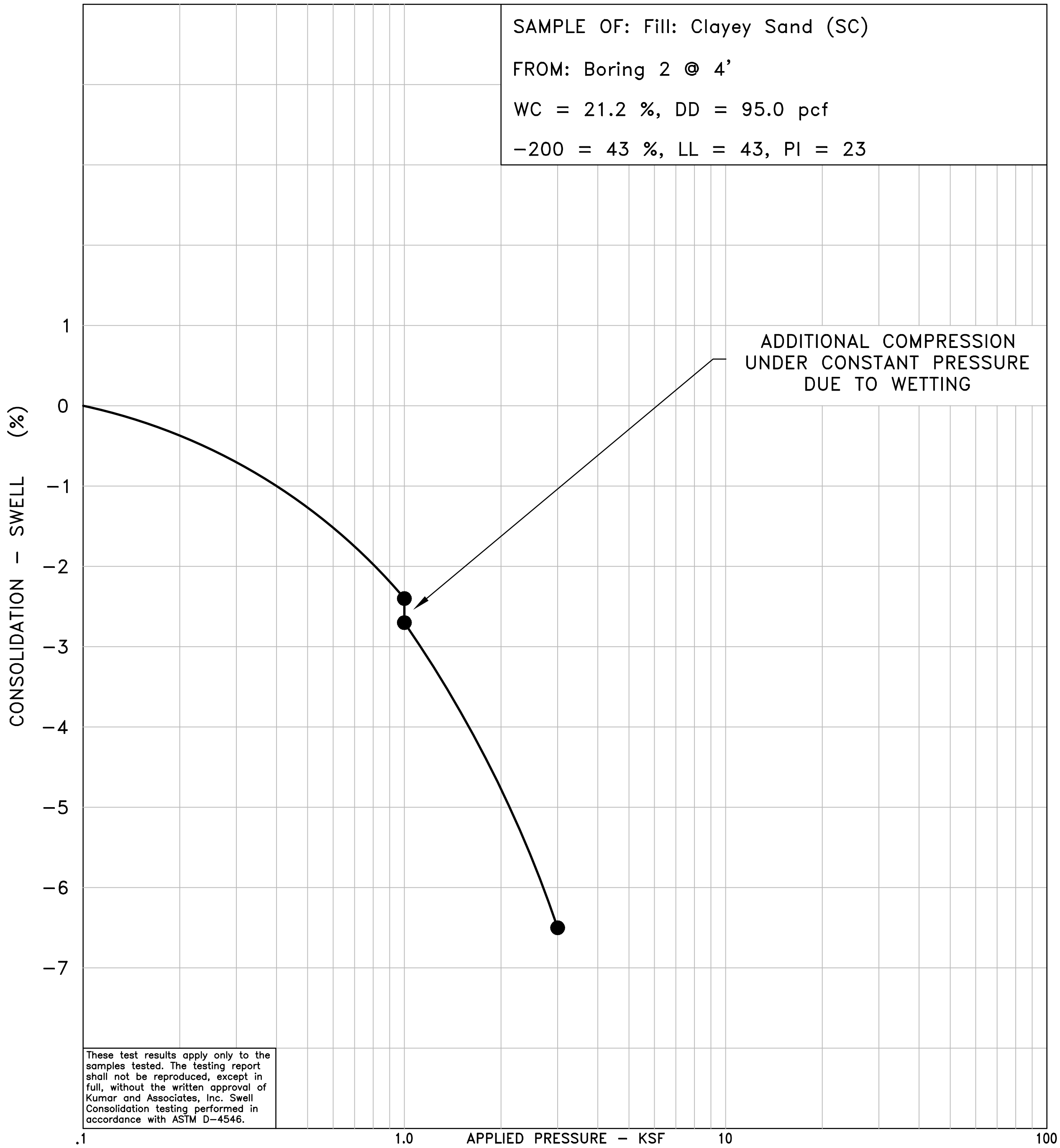

23−1−737	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULT	Fig. 7

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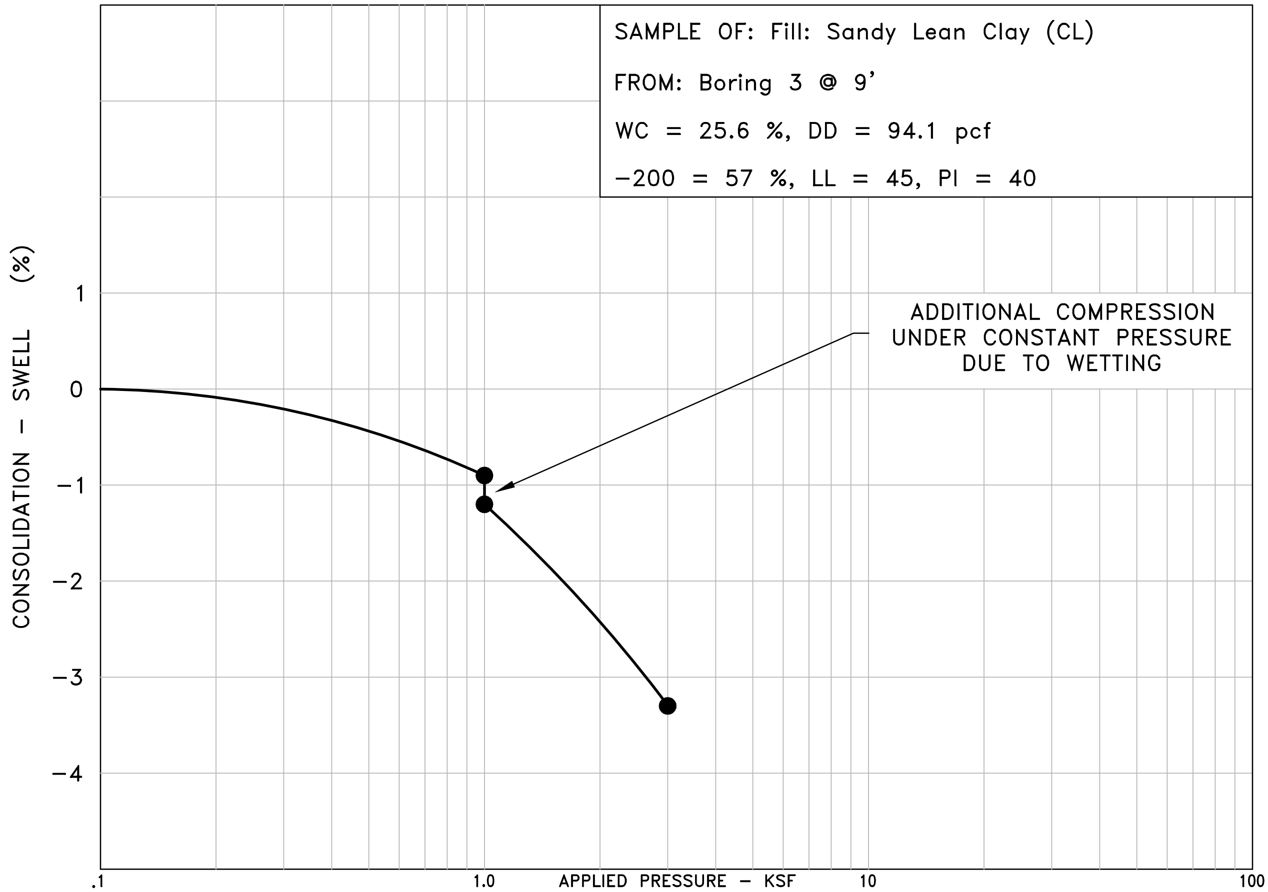
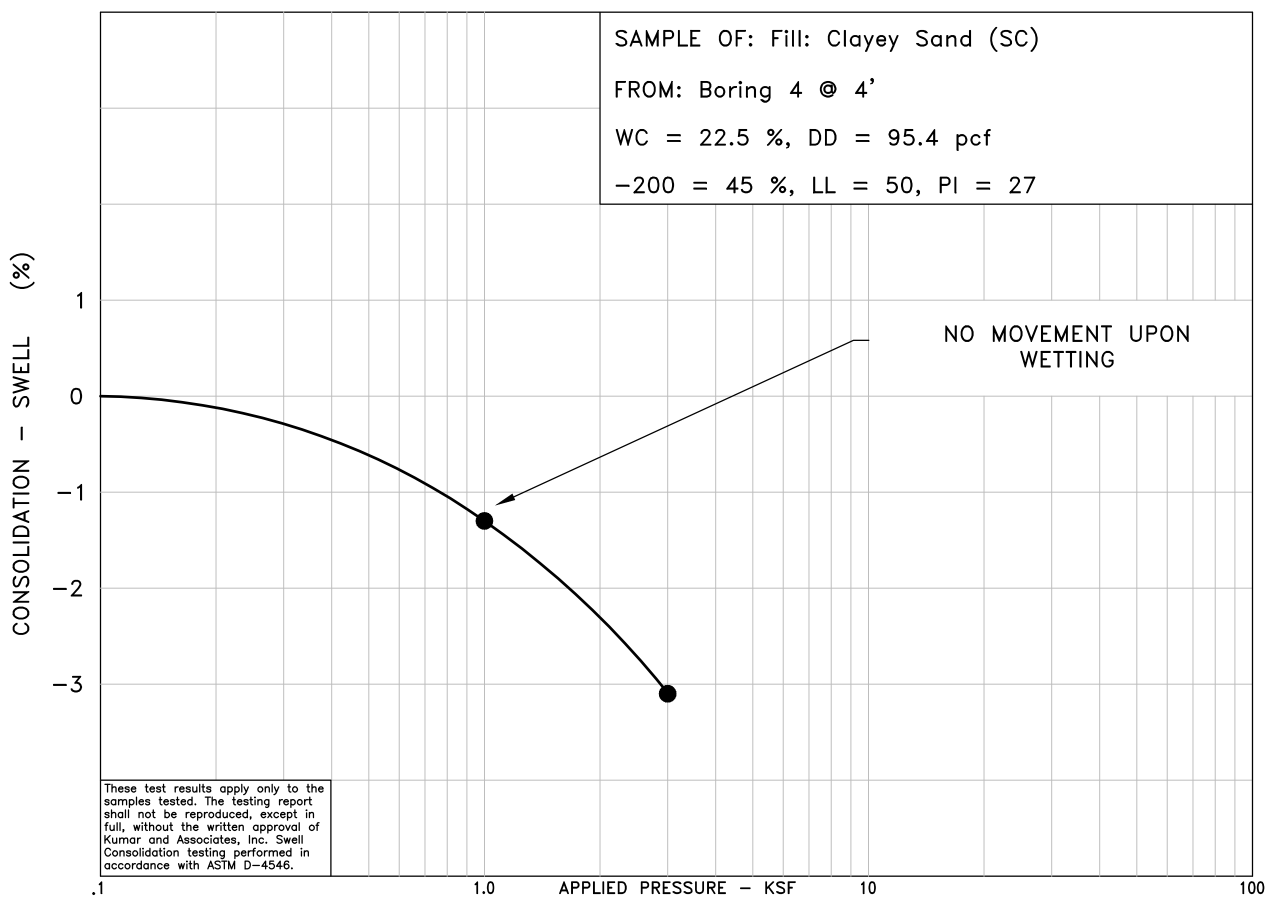
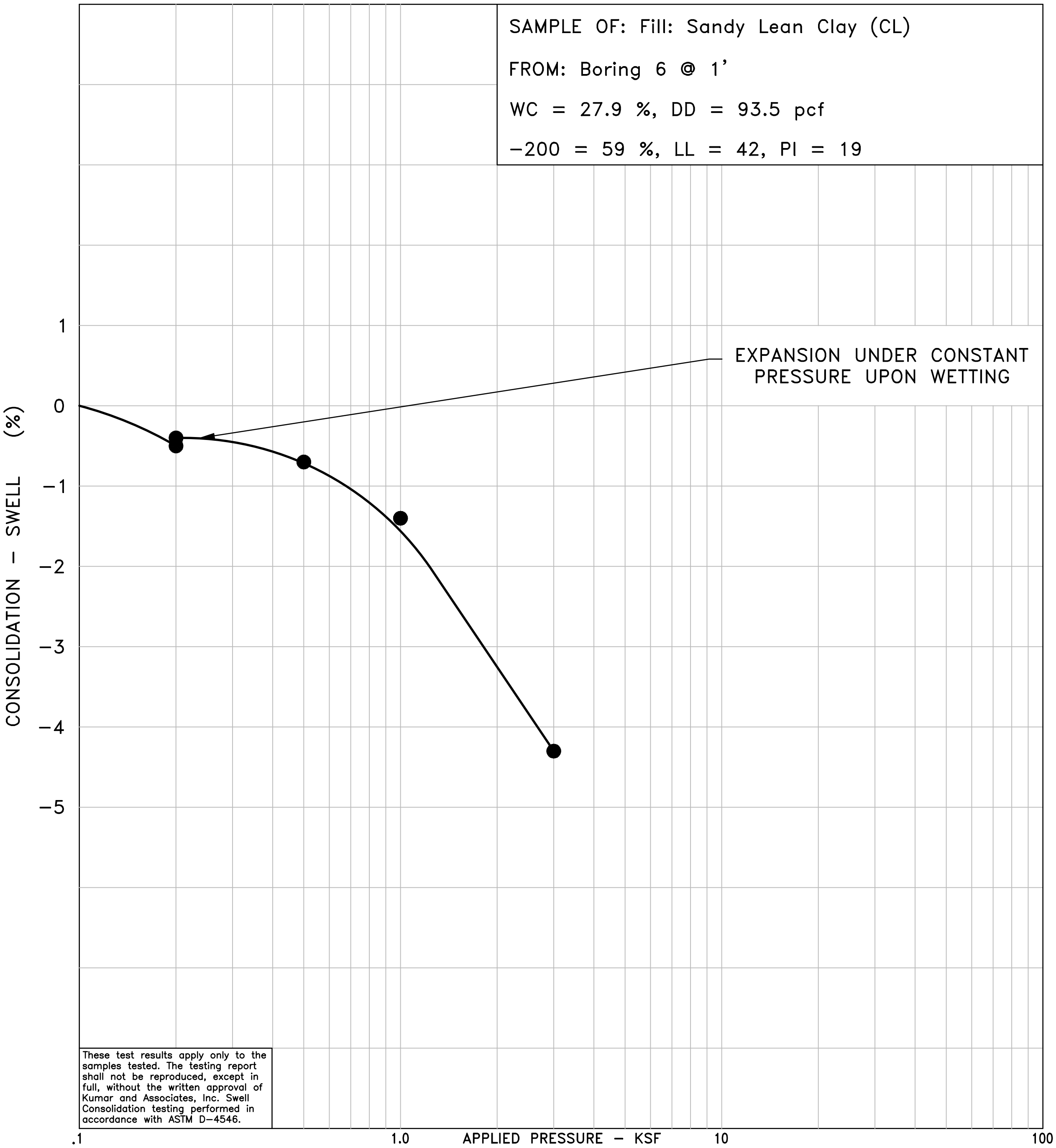




23−1−737	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULTS	Fig. 8



23−1−737	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULT	Fig. 9

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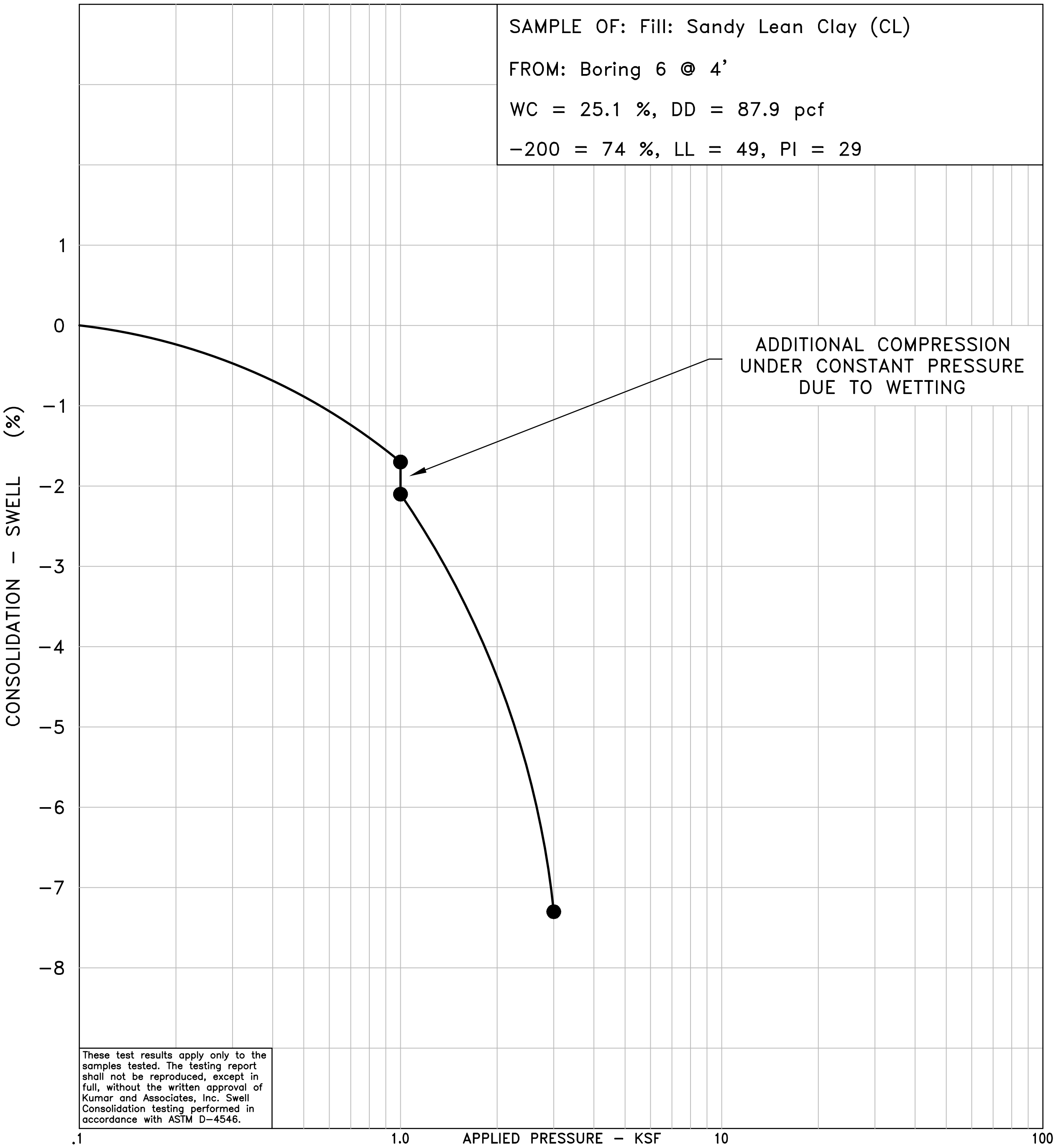

23−1−737	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULT	Fig. 10

February 02, 2024 − 01:27pm
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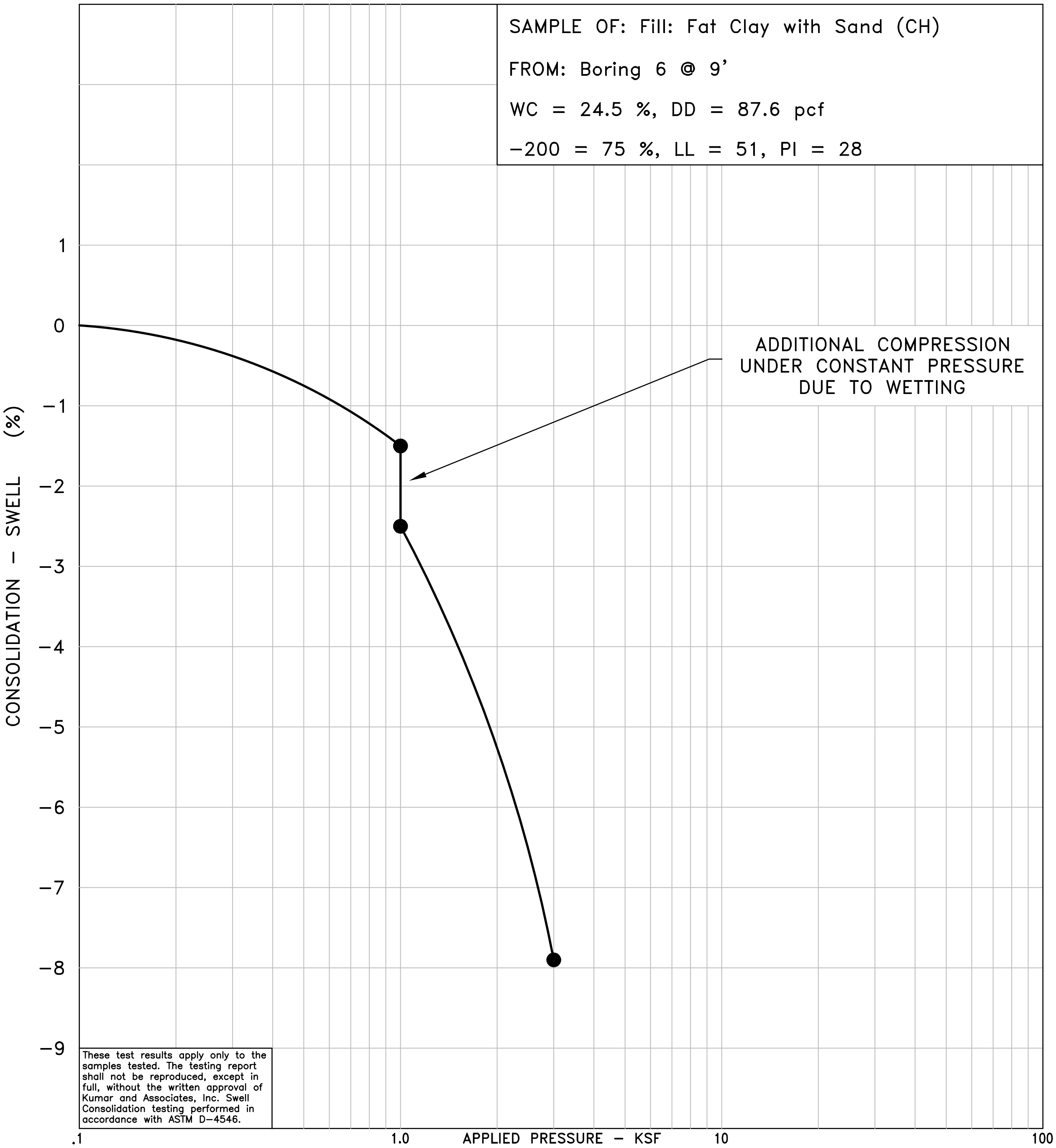

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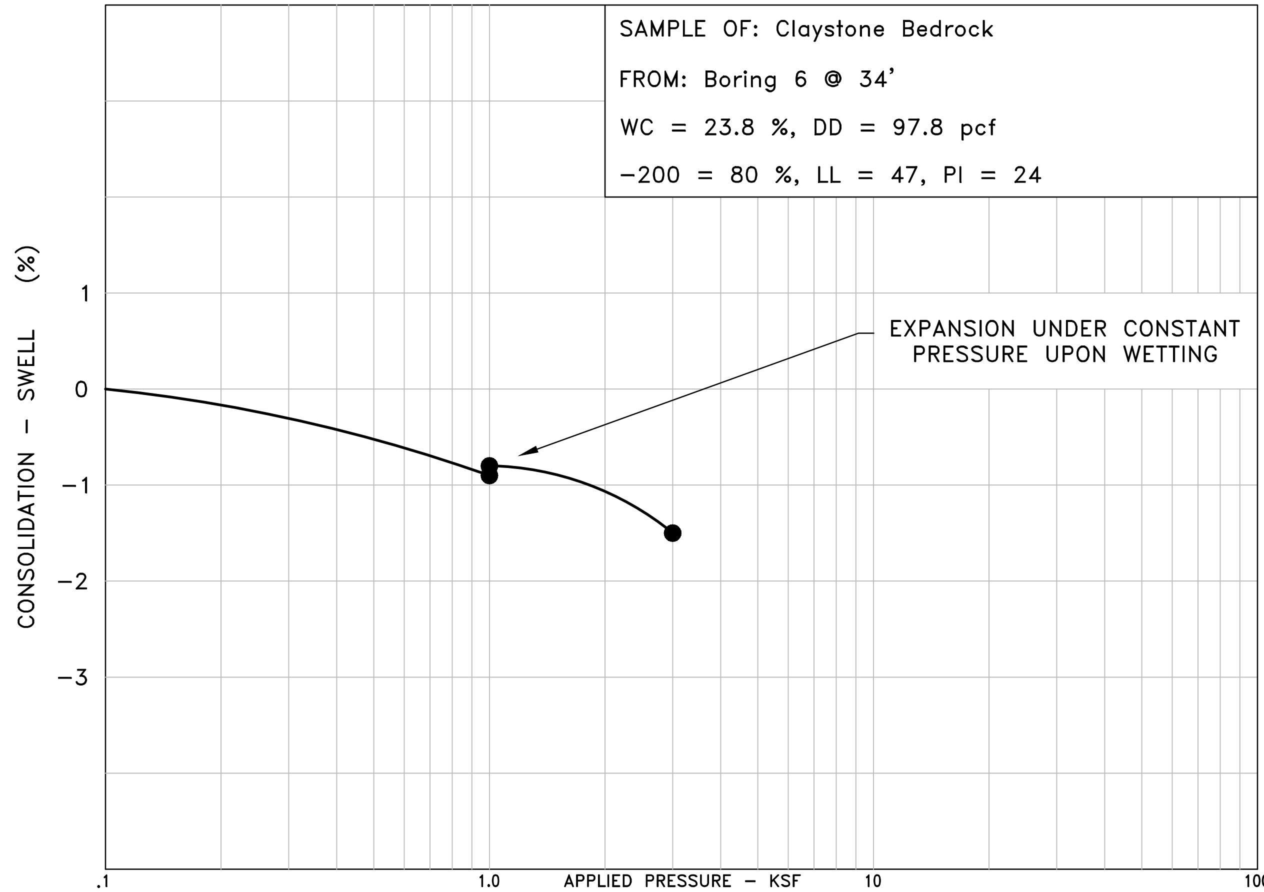
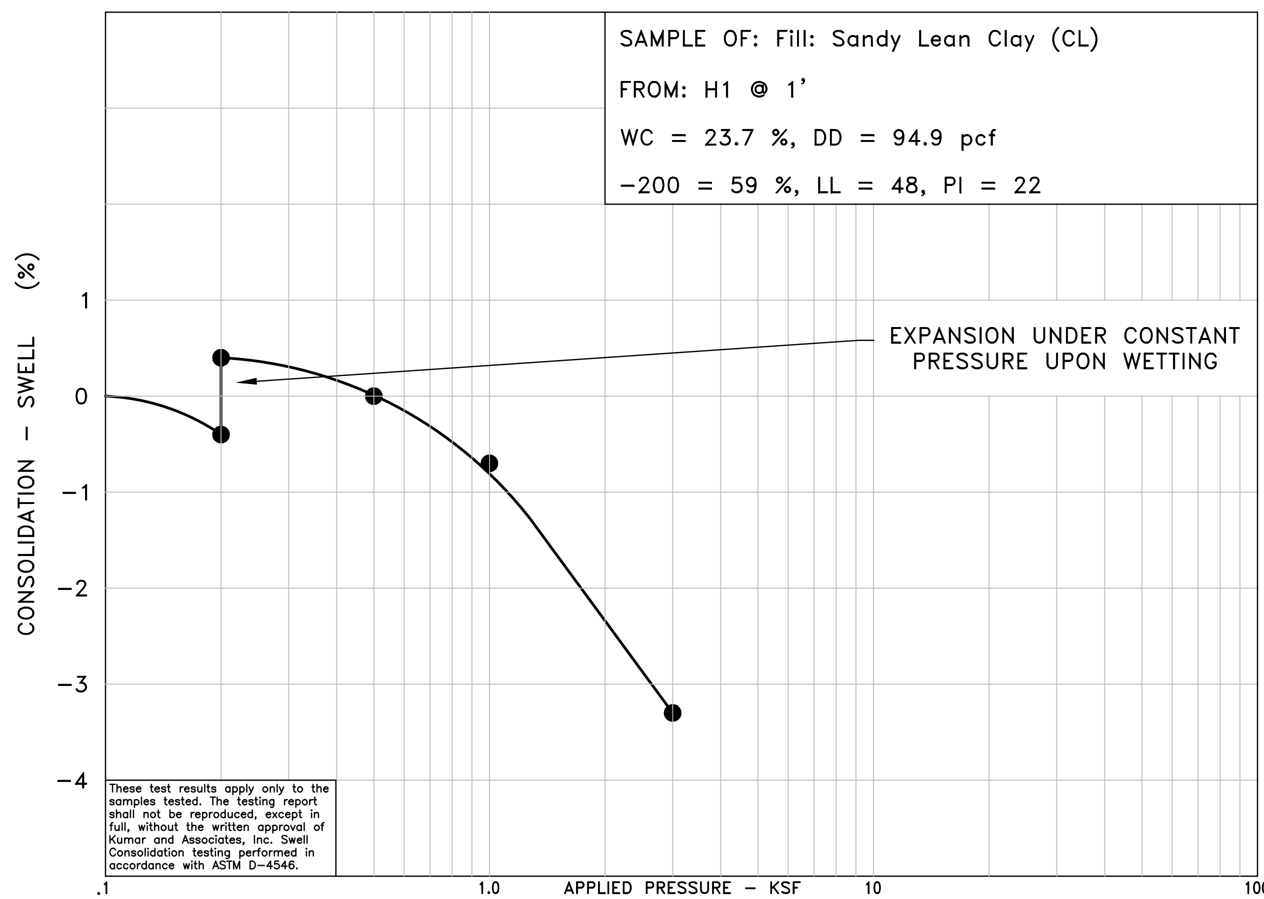
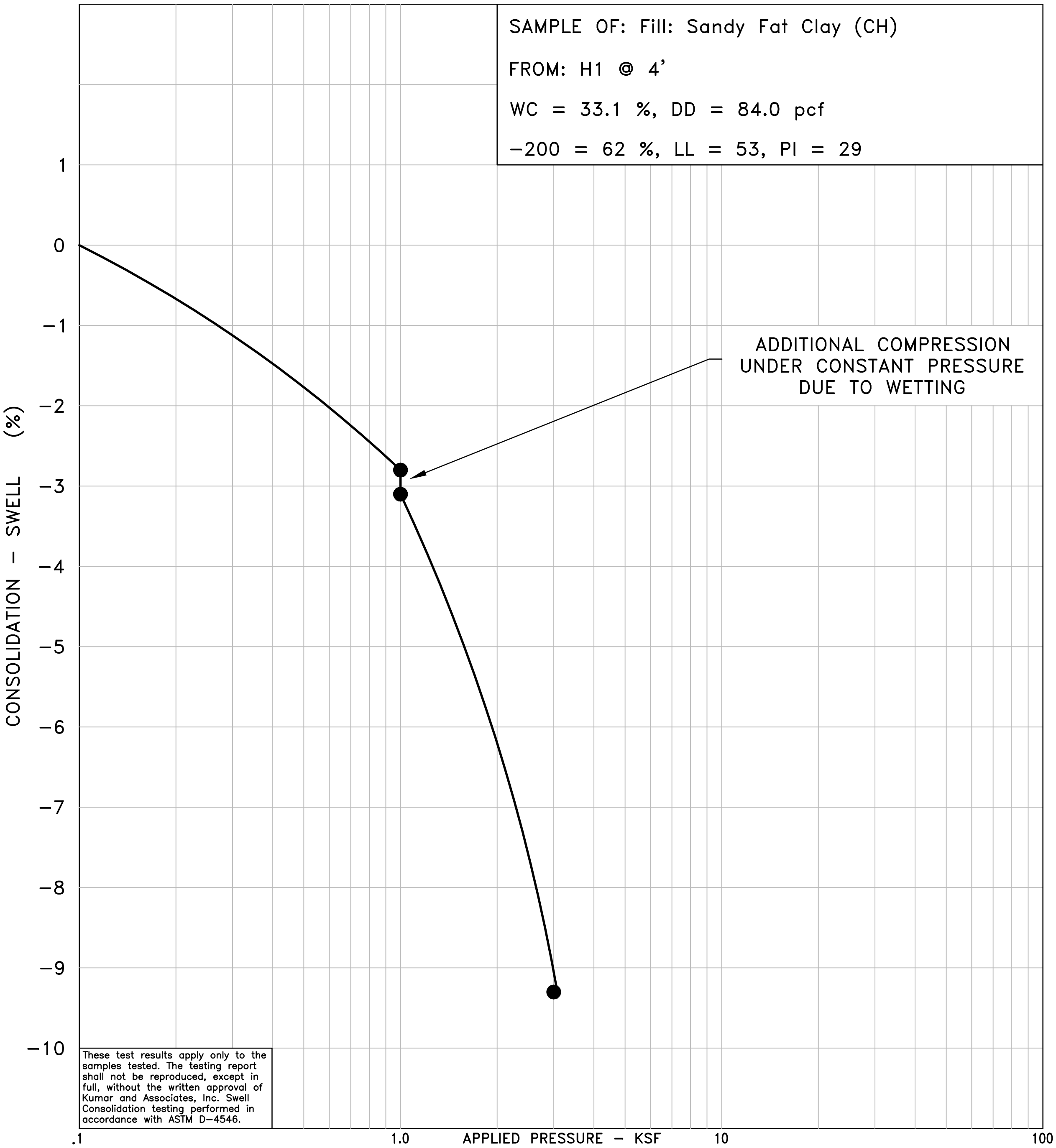
23-1-737	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULT	Fig. 11





23−1−737	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULS	Fig. 12



23-1-737	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULT	Fig. 13

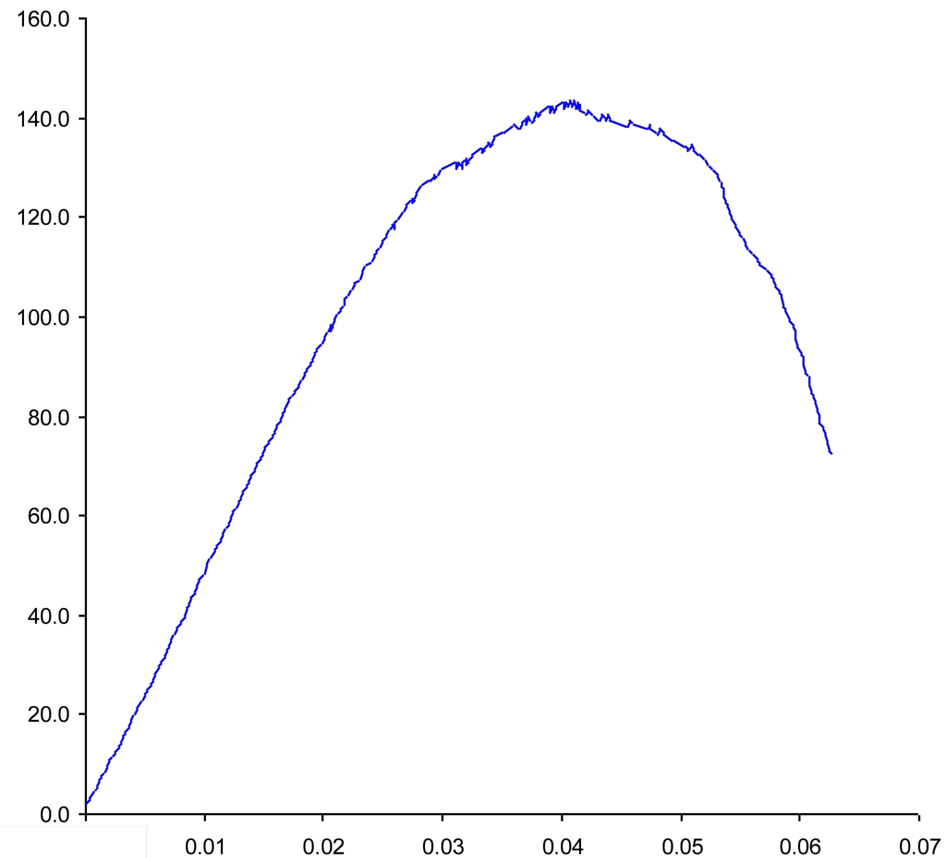
NEW Unconfined Comp (20000lbs)

Specimen ID B2-34
Test Number 10348
Report Number 4081
Test Date 12/14/2023 9:24:26 AM

Test Results	
Sample Dia.-inches	1.94
Sample Area.-inches^2	2.96
Stress (PSI)	143.4
Peak Load (lbs.)	424
Comp. Str. (lbs./sf)	20,645
% Strain at Failure	0.04

Testing Machine STM-20K 1105588	
Load Cell S/N (TVI123194), Units (LBS)	20000
Crosshead Speed (Inches / min) or Rate	0.03
Extension or Position Measured by	XHD_100 (XHD100)

Stress (-Lbs / Inches ²) vs Extension (-%)



By : ______________________ Date : ____________

Job Name	Avent Tower Addition	Job Number	23-1-737	Operator	TC
Hole Number	2	Depth	34	Descr:	Claystone
Load Rate= 0.5-2.0%/min					
				Part Number	

Template No 29	14-Dec-23	Length	3.9	Min/Max Load Rate	0.25
Kumar & Associates		Len/Dia Ratio	2.01		

23–1–737	Kumar & Associates	UNCONFINED COMPRESSION RESULTS	Fig. 14

NEW Unconfined Comp (20000lbs)

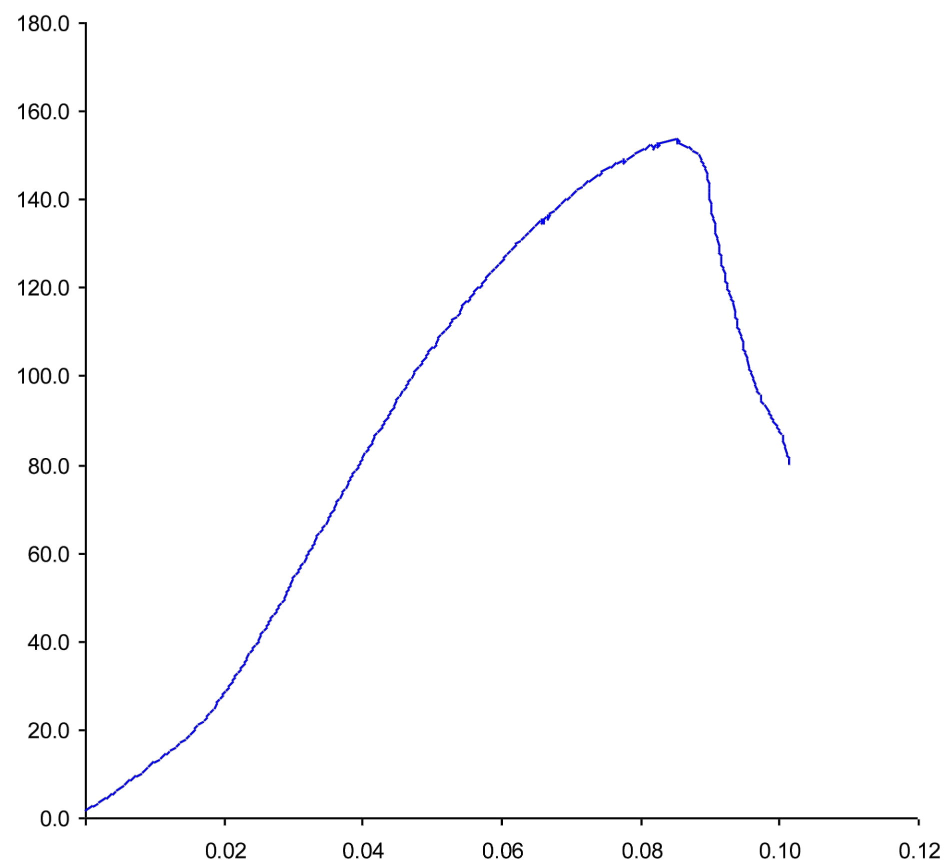
Specimen ID B4-45'
Test Number 10349
Report Number 4082
Test Date 12/14/2023 9:40:52 AM

	Test Results
Sample Dia.-inches	1.94
Sample Area.-inches^2	2.96
Stress (PSI)	153.7
Peak Load (lbs.)	455
Comp. Str. (lbs./sf)	22,133
% Strain at Failure	0.09

Testing Machine STM-20K 1105588
Load Cell S/N (TVI123194), Units (LBS) 20000

Crosshead Speed (Inches / min) or Rate 0.03
Extension or Position Measured by XHD_100 (XHD100)

Stress (-Lbs / Inches ²) vs Extension (-%)



By : ____________________ Date : __________

Job Name	Avent Tower Addition	Job Number	23-1-737	Operator	TC
Hole Number	4	Depth	45'	Descr:	Claystone
Load Rate= 0.5-2.0%/min					
				Part Number	

Template No 29	14-Dec-23	Length	4	Min/Max Load Rate	0.25
Kumar & Associates		Len/Dia Ratio	2.06		

23-1-737	Kumar & Associates	UNCONFINED COMPRESSION RESULTS	Fig. 15

February 02, 2024 - 01:27pm
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TABLE I
SUMMARY OF LABORATORY TEST RESULTS

PROJECT NO.: 23-1-737
PROJECT NAME: Advent Health Parker Addition - 9395 Crown Crest Boulevard, Parker, Colorado
DATE SAMPLED: 12/1/2023, 12/5/2023, & 12/15/2023
DATE RECEIVED: 12/6/2023 & 12/12/2023

SAMPLE LOCATION		DATE TESTED	NATURAL MOISTURE CONTENT (%)	NATURAL DRY DENSITY (pcf)	PERCENT PASSING NO. 200 SIEVE	ATTERBERG LIMITS		UNCONFINED COMPRESSIVE STRENGTH (psi)	WATER SOLUBLE SULFATES (%)	AASHTO CLASSIFICATION (group index)	SOIL OR BEDROCK TYPE
BORING	DEPTH (feet)					LIQUID LIMIT (%)	PLASTICITY INDEX (%)				
1	1	12/12/23	26.3	91.6	63	57	32			A-7-6 (19)	Fill: Sandy Fat Clay (CH)
1	9	12/12/23	23.5	97.4	26	41	12				Fill: Silty Sand (SM)
1	39	12/12/23	22.7	100.8							Sandstone Bedrock*
2	1	12/12/23	16.7	102.8	22	46	21			A-2-7 (0)	Fill: Clayey Sand (SC)
2	4	12/12/23	21.2	95.0	43	43	23		0.03	A-7-6 (5)	Fill: Clayey Sand (SC)
2	34	12/12/23	19.8	107.2				143.4			Claystone Bedrock*
3	1.5	12/12/23	22.5	97.2	34	42	20			A-2-7 (2)	Fill: Clayey Sand (SC)
3	9	12/12/23	25.6	94.1	57	45	40		0.01		Fill: Sandy Lean Clay (CL)
4	4	12/12/23	22.5	95.4	45	50	27			A-7-6 (8)	Fill: Clayey Sand (SC)
4	9	12/12/23	26.4	89.9	47	51	25				Fill: Clayey Sand (SC)
4	45	12/12/23	20.3	106.7				153.7	0.03		Claystone Bedrock*
5	1.5	12/12/23	27.6	91.1	64	55	28			A-7-6 (17)	Fill: Sandy Fat Clay (CH)
5	9	12/12/23	24.0	97.5	54	43	23				Fill: Sandy Lean Clay (CL)
6	1	12/12/23	27.9	93.5	59	42	19			A-7-6 (9)	Fill: Sandy Lean Clay (CL)
6	4	12/22/23	25.1	87.9	74	49	29				Fill: Sandy Lean Clay (CL)
6	9	12/22/23	24.5	87.6	75	51	28		0.13		Fill: Fat Clay with Sand (CH)
6	34	12/22/23	23.8	97.8	80	47	24				Claystone Bedrock
H1	1	12/12/23	23.7	94.9	59	48	22			A-7-6 (11)	Fill: Sandy Lean Clay (CL)
H1	4	12/12/23	33.1	84.0	62	53	29		0.13	A-7-6 (16)	Fill: Sandy Fat Clay (CH)

* Classification based on visual observation