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GEOTECHNICAL ENGINEERING STUDY AND PAVEMENT THICKNESS DESIGN PROPOSED MULTI-FAMILY BUILDING EAST 25TH DRIVE AND JOLIET STREET AURORA, COLORADO

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SUMMARY

1. A field exploration program consisting of drilling eleven (11) exploratory borings was conducted to obtain information on subsurface conditions. Six borings (Boring 1 through 6) were drilled within the proposed building footprint area, three borings (P-1 through P-3) were drilled within the proposed private pavement area, and two borings (I1 and I2) were drilled within Ironton Street.

   The subsurface conditions encountered in seven of the borings consisted of a thin layer of topsoil. Beneath the topsoil, where encountered, and at the ground surface in the remaining borings, pre-existing fill was encountered. In all borings but P-2 and P-3, the pre-existing fill extended to naturally deposited (natural) soils at depths ranging from about 4 to 9 feet. The borings were terminated in natural soils at depths ranging from 5 to 25 feet. The pre-existing fill encountered in Borings P-2 and P-3 extended to the maximum drilled depth of about 5 feet.

   Groundwater was not encountered in the borings during drilling. The borings were left open to measure stabilized groundwater. Groundwater was not encountered in the borings when measured 12 days after drilling. The borings were backfilled after these measurements.

2. Based on the information obtained from the exploratory borings and the results of the laboratory testing, we recommend the proposed building be founded on spread footings or post-tensioned slabs (PT-Slabs) placed on a zone of properly moisture-conditioned and compacted structural fill.

3. Slab-on-grade floors are considered feasible for the proposed construction. Slab-on-grade floors and movement-sensitive flatwork should be placed on a zone of properly moisture-conditioned and compacted structural fill, as described in the “Floor Slabs” section of this report.

4. Pavement section alternatives based on the anticipated traffic volume, on-site material properties, and local industry standards of practice for both on-site private pavement areas and for the city-owned roadway on the west side of the site are presented herein.:

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
City-Owned Roadway (Local)	N/A	6.0/9.0	8.0

PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical engineering study and pavement thickness design performed for the proposed apartment building to be constructed at the southwest corner of the intersection of East 25th Drive and Joliet Street in Aurora, Colorado. The project site is shown on Fig. 1. The geotechnical study was performed in general accordance with the scope of work presented in our Proposal No. P-24-595 to Aurora Housing Authority dated June 19, 2024.

A field exploration program consisting of drilling exploratory borings was conducted to obtain information on subsurface conditions. Representative samples of the on-site soils 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 the construction of the proposed facility are included in the report.

PROPOSED CONSTRUCTION

Based on the information provided, we understand the development will include the construction of a four-story, multi-family building with a footprint area of approximately 30,000 square feet to be constructed at the eastern portion of the site, about 20,000 square feet of at-grade parking lot at the central portion of the site, and a stormwater detention system of approximate area of 3,500 square feet at the western portion of the site. Foundation loads are expected to be moderate, consistent with this type of construction. Ironton Street will be constructed on the western end of the site.

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

SITE CONDITIONS

The site consists of an approximately 3-acre vacant lot. The site is bounded on the north by East 25th Drive, on the east by Joliet Street, and south and west by residential properties. Based on available topographic information, the site is almost level, with about 4 to 6 feet of elevation relief across the site.

SUBSURFACE CONDITIONS

Field Exploration: The subsurface conditions were explored by drilling eleven (11) exploratory borings at the approximate locations shown on Fig. 1. The borings were advanced through the pre-existing fill and into natural soils, where encountered, using 4-inch-diameter, continuous-flight, solid-stem augers and were logged by a representative of Kumar and Associates, Inc. (K+A). Samples of the fill materials and natural soils were obtained with a 2-inch-I.D. California-liner sampler driven into the various strata with blows from a 140-pound hammer falling 30 inches. Sampling with the California-liner sampler is generally similar to the standard penetration test described by ASTM International Method D1586. Penetration resistance values (blow counts), when properly evaluated, 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 the Exploratory Borings, Figs. 2 and 3. A legend and notes describing the fill materials and natural soils encountered are presented on Fig. 3.

The subsurface conditions encountered in seven of the borings consisted of a thin layer of topsoil. Beneath the topsoil, where encountered, and at the ground surface in the remaining borings, pre-existing fill was encountered. In all borings but P-2 and P-3, the pre-existing fill extended to naturally deposited (natural) soils at depths ranging from about 4 to 9 feet. The borings were terminated in natural soils at depths ranging from 5 to 25 feet. The pre-existing fill encountered in Borings P-2 and P-3 extended to the maximum drilled depth of about 5 feet.

The pre-existing fill materials generally consisted of lean clay with a variable fine- to coarse-grained sand fraction, and were moist to very moist, and light brown to brown. The exact vertical and horizontal limits of the fill were not determined in this study. The fill materials appear to be reworked on-site or locally sourced soils.

The natural soils varied between cohesive and granular soils. The cohesive soil consisted of lean clay with a variable fine- to coarse-grained sand fraction and was moist and light brown to brown.

The granular soil consisted of fine- to coarse-grained clayey sand, and was slightly moist to moist and gray to tan to light brown. Based on the blow counts, the natural granular soil was medium dense to dense, and the natural clay soil was stiff to very stiff in consistency.

Groundwater was not encountered in the borings during drilling. The borings were left open to measure stabilized groundwater. Groundwater was not encountered in the borings when measured 12 days after drilling. The borings were backfilled after these measurements.

LABORATORY TESTING

Samples obtained from the exploratory borings were visually classified in the laboratory by the project engineer. Laboratory testing was performed on representative samples to evaluate in-situ moisture content and dry unit weight, liquid and plastic limits, water-soluble sulfates, HVEEM stabilometer (R-Value), swell-consolidation behavior, remolded swell-consolidation behavior, and moisture-density relationship (standard Proctor). The above testing was performed in accordance with the applicable ASTM standard 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 Fig. 2, plotted graphically on Figs. 4 through 14, and summarized in Table I.

Swell-Consolidation: Swell-consolidation tests were conducted on five samples of the on-site pre-existing fill and two samples of the natural soils to determine the swell and/or compressibility potential under loading and when submerged in water. The samples were prepared and placed in a confining ring between porous discs, subjected to a surcharge pressure of either 200 psf or 1,000 psf, and allowed to consolidate before being submerged in water. The samples were then inundated with water. 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 3,000 psf to 5,000 psf, and the sample height was monitored until deformation practically ceased under each load increment.

Results of the swell-consolidation tests conducted on the relatively undisturbed drive samples are presented on Figs. 4 through 10 as plots of the curve of the final strain at each increment of pressure against the log of the pressure. Based on the results of the swell-consolidation test, three samples of the on-site pre-existing fill exhibited low to moderate swell potential, and one of the natural soil samples exhibited no movement under the applied surcharge pressure when wetted. The remaining two samples of the pre-existing fill and the remaining sample of the natural soil exhibited additional compression under the applied surcharge pressure when wetted. We believe the additional compression exhibited by the pre-existing fill and natural soil samples may be partially the result of sample disturbance and may not be entirely indicative of the strength characteristics of the material.

Standard Proctor (ASTM D698): A composite sample of the on-site pre-existing fill was obtained from the upper 5 feet in Borings 1 through 5 for moisture-density relationship testing (standard Proctor, ASTM D698) to determine the optimum moisture content and maximum dry density of the soil types. The standard Proctor test results are shown on Fig. 13. The maximum dry density of the on-site existing soil was determined to be 107.7 pcf, and the optimum moisture content was 13.5%. These values of the maximum dry density and the optimum moisture content are uncorrected rock values.

Remolded Swell-Consolidation: Additional swell testing was performed on the composite sample of the on-site pre-existing fill soils to determine the swell potential when remolded to approximately 95% of the maximum dry density to a moisture content near the optimum for use as fill placed beneath the building foundations, slabs-on-grade, and pavement.

The on-site pre-existing fill sample was remolded to approximately 94.2% of the maximum dry density and 0.7% above the optimum moisture content. The testing was performed under a surcharge pressure of 200 psf, indicating a swell potential of 0.5%. The result of the remolded swell-consolidation test is graphically plotted on Fig. 11.

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 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. The results of gradation testing performed on samples of the pre-existing fill are presented on Fig. 12.

GEOTECHNICAL ENGINEERING CONSIDERATIONS

Foundations: Shallow foundation systems consisting of spread footings or PT-slabs are considered feasible to support the proposed building if placed on structural fill. Recommendations for shallow foundations are presented in the following section of this report.

Soil-Supported Slabs: Subgrade preparation for areas considered movement-sensitive, such as exterior flatwork and rigid pavements adjacent to the building, should follow the recommendations presented in the "Floor Slabs for Non-PT Supported Structures" section of this report. Subgrade preparation beneath areas not considered movement-sensitive should follow the subgrade preparation recommendations presented in the "Pavement Thickness Design" section of this report.

On-site Pre-Existing Fill: As indicated previously, up to 9 feet of man-placed fill was encountered in the 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 existing fill should be considered non-engineered and unsuitable in its current condition for support of foundation elements and slabs-on-grade due to the potential for excessive settlement.

Ideally, all pre-existing fills should be completely removed from beneath spread footings, PT-slabs, floor slabs, and settlement-sensitive flatwork and replaced with properly placed and compacted structural fill. Complete removal and replacement of the pre-existing fills could result in significant costs to the project. A partial fill removal and replacement option beneath PT-slabs, floor slabs, and settlement-sensitive flatwork may be considered, provided the Owner accepts the significant risk that post-construction compression of pre-existing fill left in place may result in settlement in excess of normally accepted tolerances and associated distress. If the risk cannot be accepted by the Owner, then all pre-existing fill should be removed and replaced with engineered fill.

All pre-existing fills beneath spread footings, if selected, should be completely removed and replaced with properly placed and compacted structural fill.

FOUNDATION RECOMMENDATIONS

PT-Slabs: Due to the unpredictable post-construction movement of the pre-existing fill in its current condition, we recommend PT-Slab foundations be supported on a uniform thickness of a zone of properly moisture-conditioned and compacted structural fill. As discussed in the preceding section of this report, a partial pre-existing fill removal may be considered beneath PT-slabs, provided the significant risk of partial fill removal alternative should be considered and accepted by the Owner.

PT-Slab foundations are suitable if supported as recommended above. The subsurface soils at the site indicate that the foundations may be designed using BRAB Type II criteria; however, buildings such as the one planned for this site add a post-tensioned component to provide additional structural support while reducing materials costs. We can provide BRAB Type III PT design criteria if desired; however, the BRAB Type III design values may add cost to the overall design and are generally not used when adding a PT element to a BRAB Type II slab.

Spread Footings: Spread footing foundations, if selected, should be placed on structural fill extending to undisturbed natural soils. Material and placement criteria for structural fill are presented in the "Site Grading and Earthwork" section of this report.

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

1. Spread footings placed as described above 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.

2. Structural fill should extend down and out from the edges of the footings at a 1 (horizontal) to 1 (vertical) projection. Structural fill should meet the material type and placement recommendations, including fill subgrade preparation measures, presented in the "Site Grading and Earthwork" section of this report.

3. Based on experience, we estimate total settlement for spread footings designed and constructed as discussed herein will be approximately one inch or less for footings prepared in accordance with the previous recommendations. Differential settlements across the structure are estimated to be approximately ½ to ¾ of the total settlements.

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

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

6. The lateral resistance of a spread footing will be a combination of the sliding resistance of the footing on the foundation-bearing materials and passive earth pressure against the side of the footing. Resistance to sliding at the bottoms of the footings can be calculated based on a coefficient of friction of 0.30. Passive pressure against the sides of the footings may be calculated assuming an equivalent fluid unit weight of 185 pcf. The above values are working values.

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

FLOOR SLABS FOR NON-PT SUPPORTED STRUCTURES

Discussion of the removal of pre-existing fill to reduce potential movement is presented in the "Geotechnical Engineering Considerations" section of this report. Ideally, all pre-existing fill material should be removed from below soil-supported slabs and replaced with structural fill. However, it is our opinion a portion of the pre-existing fill may remain in place if the potential for some distress resulting from post-construction compression of the fill is recognized by the owner. As a minimum, we recommend all fill within 5 feet of floor slab subgrade level be removed and replaced with structural fill meeting the material and placement criteria presented in the "Site Grading and Earthwork" section of this report.

1. Floor slabs should be placed on at least 5 feet of structural fill. 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 100 pci is recommended for a subgrade consisting of structural fill. The modulus of the vertical subgrade reaction was correlated based on material classification and blow count values from several engineering resources, including Joseph E. Bowles Foundation Analysis and Design. The modulus value given is for a 1-foot square plate and must be corrected for slab shape and size.

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.

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

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.

SITE GRADING AND EARTHWORK

Site Preparation: Subgrade preparation should be considered beneath shallow foundations, slabs-on-grade, rigid pavement, and movement-sensitive flatwork and replaced with structural fill as recommended herein.

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

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. 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 pre-existing fills and natural granular soil will classify as OSHA Type C soils. The natural clay soil will classify as OSHA Type B soil. 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.

Fill Material Specifications: The following material specifications are presented for fills on the project site. We believe the on-site fill materials and natural soils are suitable for reuse when used according to the specifications outlined below.

1. *Structural Fill*: Structural fill may consist of on-site overburden soil, including pre-existing fill, provided it meets the material suitability and placement criteria specified in this section. Imported structural fill material, if necessary, should have a percent passing the No. 200 sieve between 30 and 70 percent, 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 swell criteria outlined in Item 4 below are satisfied.

2. *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.

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

4. *Material Suitability*: Unless otherwise defined herein, all fill material should be non- to low-swelling, free of vegetation, brush, sod, trash and debris, and other deleterious substances, and should not contain rocks or lumps having a diameter of more than 4 inches. A fill material should be considered non-expansive if the swell potential under a 200 psf surcharge pressure does not exceed 1 percent when a sample remolded to 95

percent of the standard Proctor (ASTM D698) maximum dry density at optimum moisture content is wetted.

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

Compaction Requirements: A representative of the geotechnical engineer should observe fill placement operations on a full-time basis. We recommend the following minimum compaction criteria be used on the project.

Fill Location:	Percentage of Maximum Standard Proctor Density (ASTM D698)
Beneath Spread Footing or PT-Slab Foundations	98%
Beneath Floor Slabs and Pavements[1]	95%
Utility Trenches	95%
General Site Grading and Landscape areas	95%

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

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

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.

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 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 10 feet of the building foundations during and following construction.

6. Landscaping which requires relatively heavy irrigation and lawn sprinkler heads should be located a minimum of 10 feet from foundation walls. Use of drip irrigation lines with limited irrigation quantities is generally acceptable within 10 feet of foundation walls, provided the main lines are located 10 feet outside of foundation walls.

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

SITE SEISMIC CRITERIA

The general soil profile across the site after construction will generally consist of relatively stiff to very stiff/medium dense to dense overburden soils extending to a depth of about 25 feet or more below the finished ground surface. Overburden consisting of new fill and/or existing overburden soils will generally classify as Site Class D in accordance with the International Building Code (IBC).

In the absence of measured shear wave velocities supporting 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.

INFILTRATION RATES

The approximate percolation test locations are shown on Fig. 1. The corresponding infiltration rates calculated using procedures from the Michigan LID Manual and the corresponding Hydrologic Soil Group (HSG) per USDA classification are summarized in the following table.

Test Hole	Depth of Percolation Test Hole (in.)	Calculated Infiltration Rate (cm /sec)	Calculated Infiltration Rate (in /hr.)	Hydrologic Soil Group Classification (HSG)
D1	66.5	1.84E-05	0.0261	D

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

The calculated infiltration rates provided above can be expected to diminish over time as a result of contamination by fine particles and organic material. The design team should consider the diminishment of the infiltration rate in the final design. The on-site soils generally classify as HSG-D. We recommend the infiltration rate presented in HSG-D Soils be used for design purposes.

WATER-SOLUBLE SULFATES

The concentration of water-soluble sulfates measured in a representative sample of the on-site natural soil was 0.00%. This concentration represents Class S0 exposure to sulfate attack on concrete exposed to these materials. The degree of attack is based on a range of Class S0 (not applicable), Class S1 (moderate), Class S2 (severe), and Class S3 (very severe) severity of exposure, as presented in ACI 201.2R.

Based on the laboratory test results, we believe special sulfate-resistant cement will not be required for concrete exposed to the on-site overburden soils.

PAVEMENT THICKNESS 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 exploration and laboratory testing programs, the near-surface subgrade materials at the site generally classify as A-6 soils, and isolated zones classify as A-4 soils with group indices ranging from 5 to 18 in accordance with the American Association of State Highway and Transportation Officials (AASHTO) classification. Soils classifying as A-6 are generally considered to provide poor subgrade support. Soils classifying as A-4 are generally considered to provide fair subgrade support.

Hveem stabilometer (R-value) testing was performed on a composite bulk sample of the anticipated pavement subgrade materials to determine the support characteristics for pavement thickness design purposes. Based on the result of the laboratory testing, the samples classified as an A-6 soil with an R-value of 11 at an exudation pressure of 300 psi. The R-value of 11 was correlated to a design resilient modulus value of 3,681 psi using CDOT correlations. A seasonally adjusted modulus of subgrade reaction (K-value) of 35 pci was selected for rigid pavements.

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

We understand the city-owned street on the east side of the property will classify as a local street. Accordingly, an ESAL of 600,000 was selected based on Table 5.07.1.1 of the City of Aurora Standards Specification (Standards). The roadway designer and design team should verify which traffic loads are valid for the project. If higher ESAL values are anticipated we should be informed as the pavement sections presented in this report will have to be reevaluated.

Pavement Sections: The pavement thicknesses were determined in accordance with the 1993 AASHTO pavement design procedures and the City Standards for private pavement and the city-owned roadway, respectively. For flexible pavement design of private pavements, initial and terminal serviceability indices of 4.5 and 2.0, respectively, were selected, with a reliability of 80 percent. For the City-owned roadway, initial and terminal serviceability indices of 4.5 and 2.0, respectively, were selected, with reliability values of 85 percent (flexible pavements) and 90 percent (rigid pavements) in accordance with Section 5.10.5.01 of the City Standards.

If other design parameters are preferred, we should be contacted to reevaluate the recommendations presented herein. The pavement section recommendations provided herein are based on a 20-year pavement design life.

Based on this procedure, flexible pavements should meet the minimum requirements presented in the table below.

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
City-Owned Roadway (Local)	N/A	6.0/9.0	8.0

Areas of the private pavement, truck loading dock areas, and other areas where truck turning movements are concentrated should be paved with 8.0 inches of Portland cement concrete. The concrete pavement should contain sawed or formed joints to ¼ of the depth of the slab at a maximum distance of 12 feet on center. Concrete pavement will be somewhat more sensitive to heave-related movements than asphalt pavement.

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 mixed asphalt (HMA) pavements should meet the material specifications for Class 6 ABC stated in the current Colorado Department of Transportation (CDOT) *"Standard Specifications for Road and Bridge Construction"*. The ABC material should have a minimum R-value of 78 at an exudation pressure of 300 psi. The ABC should be compacted to a minimum of 98% of the standard Proctor (ASTM D 698) 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 *"Standard Specifications for Road and Bridge Construction"*. We recommend the HMA used for this project is designed in accordance with the Super Pave gyratory mix design method. The mix should generally meet Grading S or SX specifications with a Super Pave gyratory design revolution (*NDESIGN*) of 75. The mix design for the HMA should use a performance grade PG 58-28 asphalt binder. Placement and compaction of HMA should follow current CDOT standards and specifications.

3. *Portland Cement Concrete*: Portland cement concrete pavement (PCCP) should meet Class P specifications and requirements in the current CDOT *"Standard Specifications for Road and Bridge Construction"*. Rigid PCCP is 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 ¼ of the depth of the slab at a maximum distance of 12 to 15 feet on center.

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 vehicles, 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 of 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 are critical to the performance of PCCP, especially if dowels and tie bars are not installed.

Subgrade Preparation: Pavement subgrade conditions are anticipated to include expansive clay soils, which are a problem when present beneath pavements. Expansive soils could result in potentially excessive heave when subjected to increases in moisture.

As previously stated, the overburden clay soils primarily exhibited low to moderate swell potential upon wetting. Accordingly, private on-site pavements should be supported on a minimum of 2 feet of structural fill. Prior to placing structural fill, the fill subgrade should be scarified, well-mixed, moisture-conditioned, and compacted according to the recommendations presented in the "Site Grading and Earthwork" section of this report. This will result in a 3-foot-thick zone of prepared subgrade below on-site pavements.

For the city-owned roadway, we recommend the pavement be underlain by a minimum of 2 feet of structural fill with the upper 12 inches consisting of cement-treated subgrade (CTS). Chemical stabilization through CTS should consist of blending the clayey subgrade materials with cement such that the final product provides a minimum compressive strength of 160 psi at 5 days under moist curing conditions.

There is no requirement for base course material between the chemically stabilized subgrade and the asphalt; however, providing a thin layer of base material would result in a bond breaker that would mitigate cracks in the treated subgrade from propagating through the asphalt surface.

The pavement subgrade should also 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. 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. Paving should be completed within 48 hours of the completion of subgrade preparation and a passing proof roll. Subgrades open to precipitation events should be reworked and retested for moisture and density prior to paving.

The owner should be aware sub-excavation and replacement, as well as chemical stabilization, will reduce but not eliminate the potential movement of pavements should moisture levels increase within the expansive soils beneath the replacement fill and/or CTS.

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.

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 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.

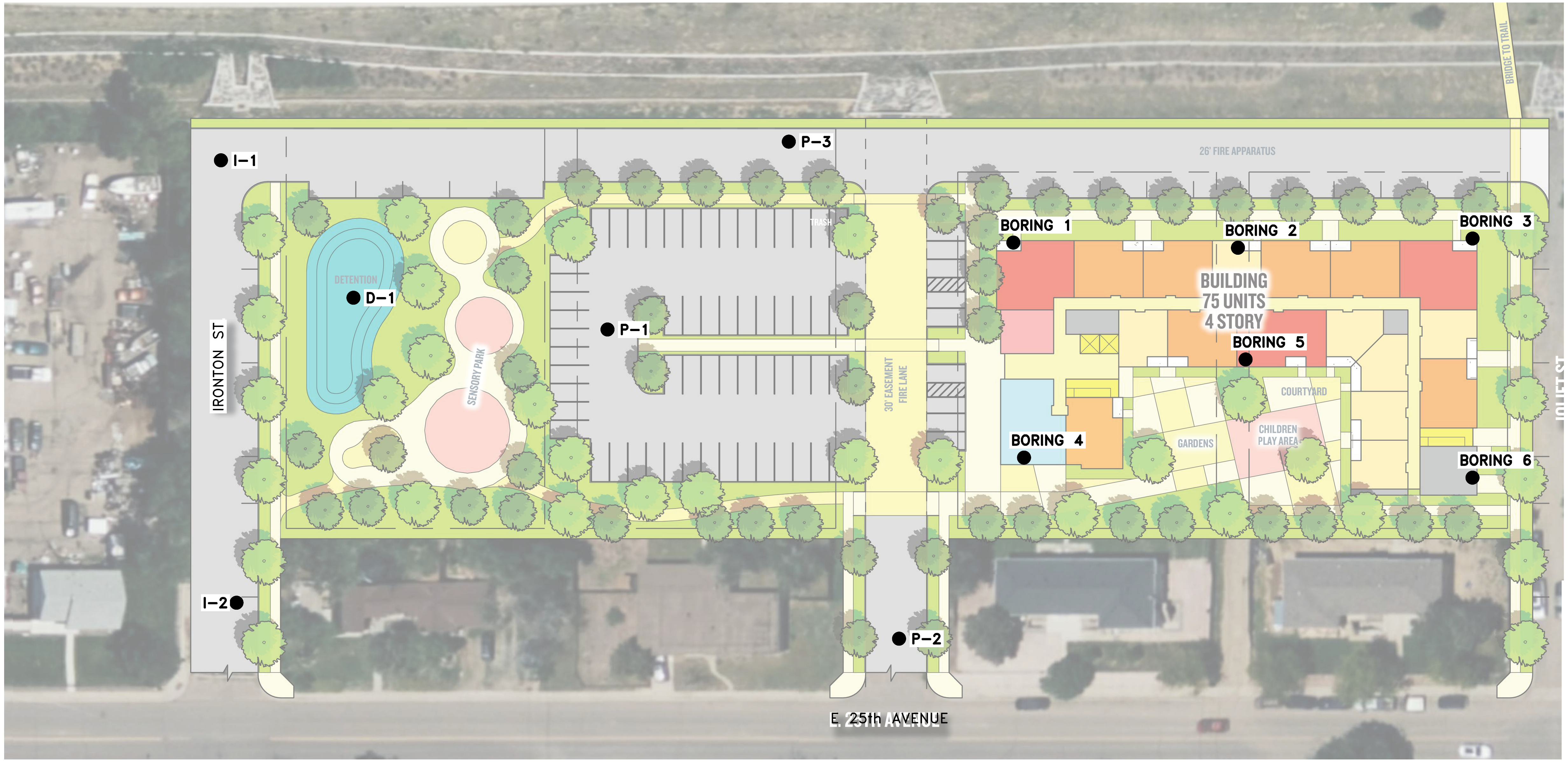
LIMITATIONS

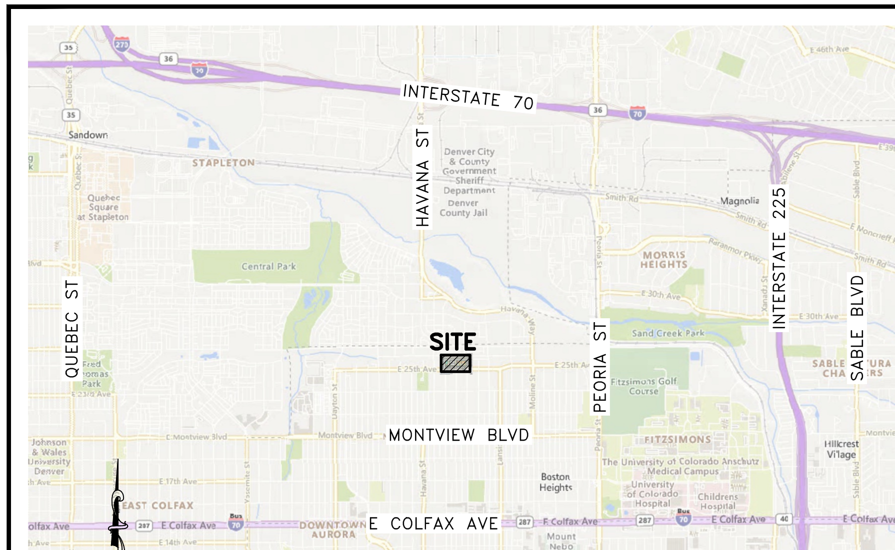
This study has been conducted for the exclusive use by the client and provides geotechnical related design and construction recommendations for the project. The conclusions and recommendations submitted in this report are based upon the data obtained from the exploratory borings at the locations indicated on Fig. 1 or as described in the report, 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, bedrock or groundwater conditions appear to be different from those described herein, K+A should be advised at once so 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 the experience of our engineers on the behavior of swelling soil in this area. 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, floor slab, and pavement movement due to expansive soils and bedrock.

KMH/mm
Rev. by: JLB
Enclosures
cc: File





VICINITY MAP
NOT TO SCALE

30 0 30 60
APPROXIMATE SCALE–FEET

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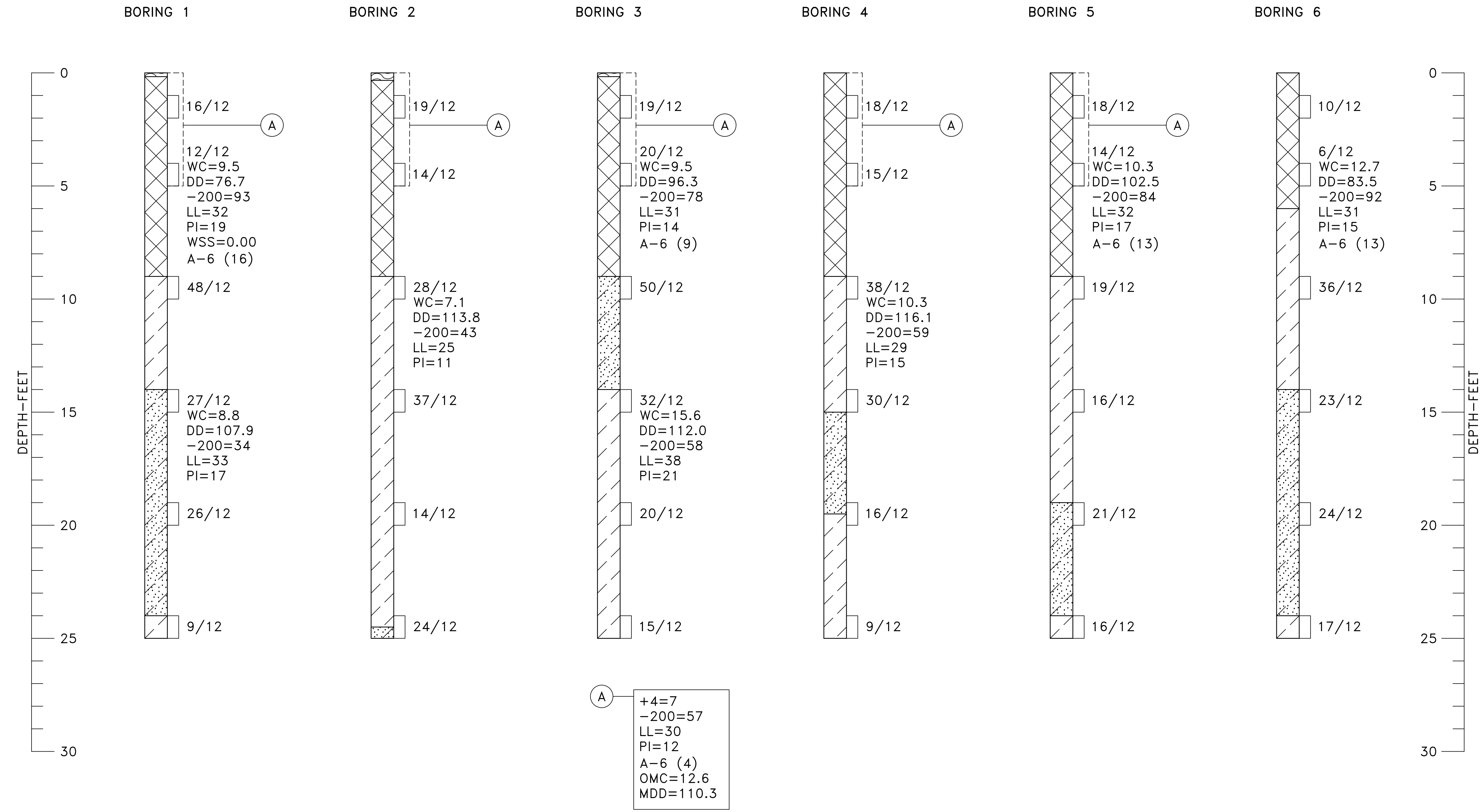
24–1–440	Kumar & Associates	STANLEY 98 AURORA – EAST 25th DRIVE & JULIET STREET, AURORA, COLORADO	LOCATION OF EXPLORATORY BORINGS	Fig. 1



24−1−440	Kumar & Associates	STANLEY 98 AURORA − EAST 25th DRIVE & JULIET STREET, AURORA, COLORADO	LOGS OF EXPLORATORY BORINGS	Fig. 2

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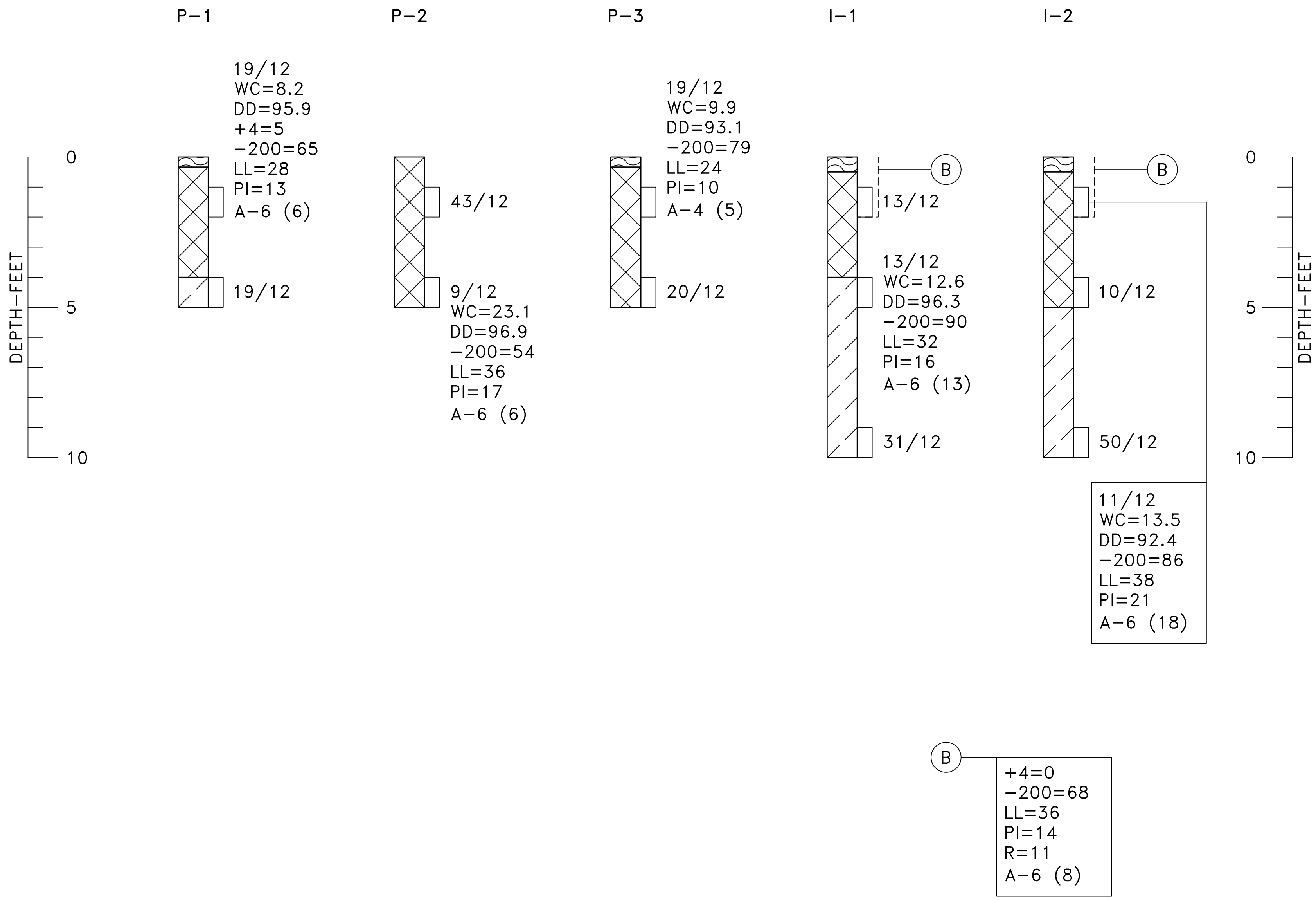


LEGEND

TOPSOIL.

FILL: LEAN CLAY (CL) WITH VARIABLE FINE- TO COARSE-GRAINED SAND FRACTION, MOIST TO VERY MOIST, LIGHT BROWN TO BROWN.

LEAN CLAY (CL) WITH VARIABLE FINE- TO COARSE-GRAINED SAND FRACTION, STIFF TO VERY STIFF, MOIST, LIGHT BROWN TO BROWN.

CLAYEY SAND (SC), FINE- TO COARSE-GRAINED, MEDIUM DENSE TO DENSE, SLIGHTLY MOIST TO MOIST, GRAY TO TAN TO LIGHT BROWN.

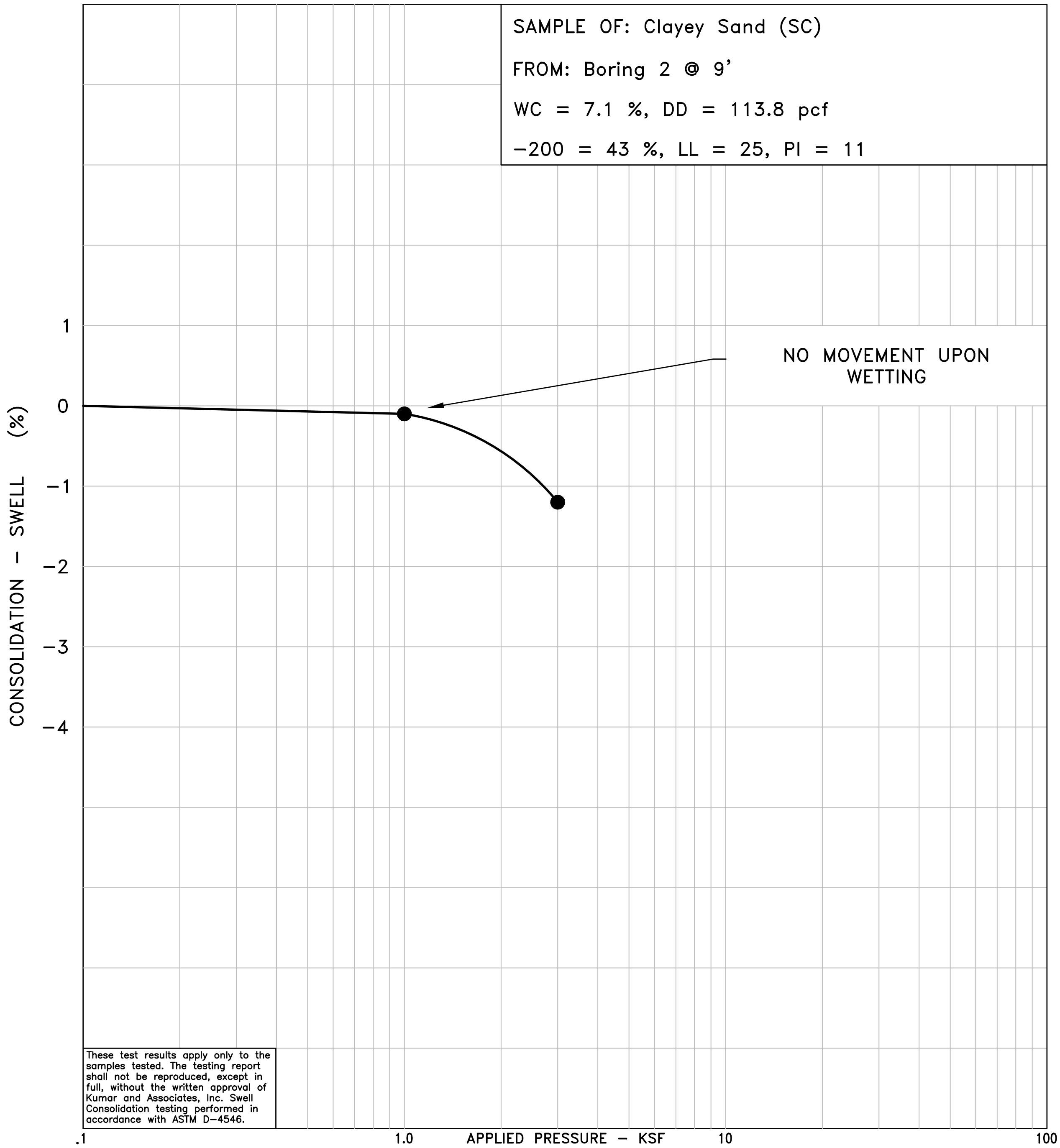
DRIVE SAMPLE, 2-INCH I.D. CALIFORNIA LINER SAMPLE.

DISTURBED BULK SAMPLE.

16/12 DRIVE SAMPLE BLOW COUNT. INDICATES THAT 16 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 JULY 10, 2024 WITH A 4-INCH-DIAMETER CONTINUOUS-FLIGHT POWER AUGER.
2. THE LOCATIONS OF THE EXPLORATORY BORINGS WERE LOCATED BY GPS COORDINATES OBTAINED FROM GOOGLE EARTH™ AND LOCATED IN THE FIELD WITH A HANDHELD GPS UNIT.
3. THE ELEVATIONS OF THE EXPLORATORY BORINGS WERE NOT MEASURED AND THE LOGS OF THE EXPLORATORY BORINGS ARE PLOTTED TO DEPTH.
4. THE EXPLORATORY BORING LOCATIONS SHOULD BE CONSIDERED ACCURATE ONLY TO THE DEGREE IMPLIED BY THE METHOD USED.
5. THE LINES BETWEEN MATERIALS SHOWN ON THE EXPLORATORY BORING LOGS REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN MATERIAL TYPES AND THE TRANSITIONS MAY BE GRADUAL.
6. GROUNDWATER WAS NOT ENCOUNTERED IN THE BORINGS AT THE TIME OF DRILLING NOR WHEN CHECKED 12 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);
   WSS = WATER SOLUBLE SULFATES (%) (CP-L 2103);
   A-6 (16) = AASHTO CLASSIFICATION (GROUP INDEX) (AASHTO M 145);
   R = HVEEM R-VALUE (AT 300 psi) (ASTM D2844).



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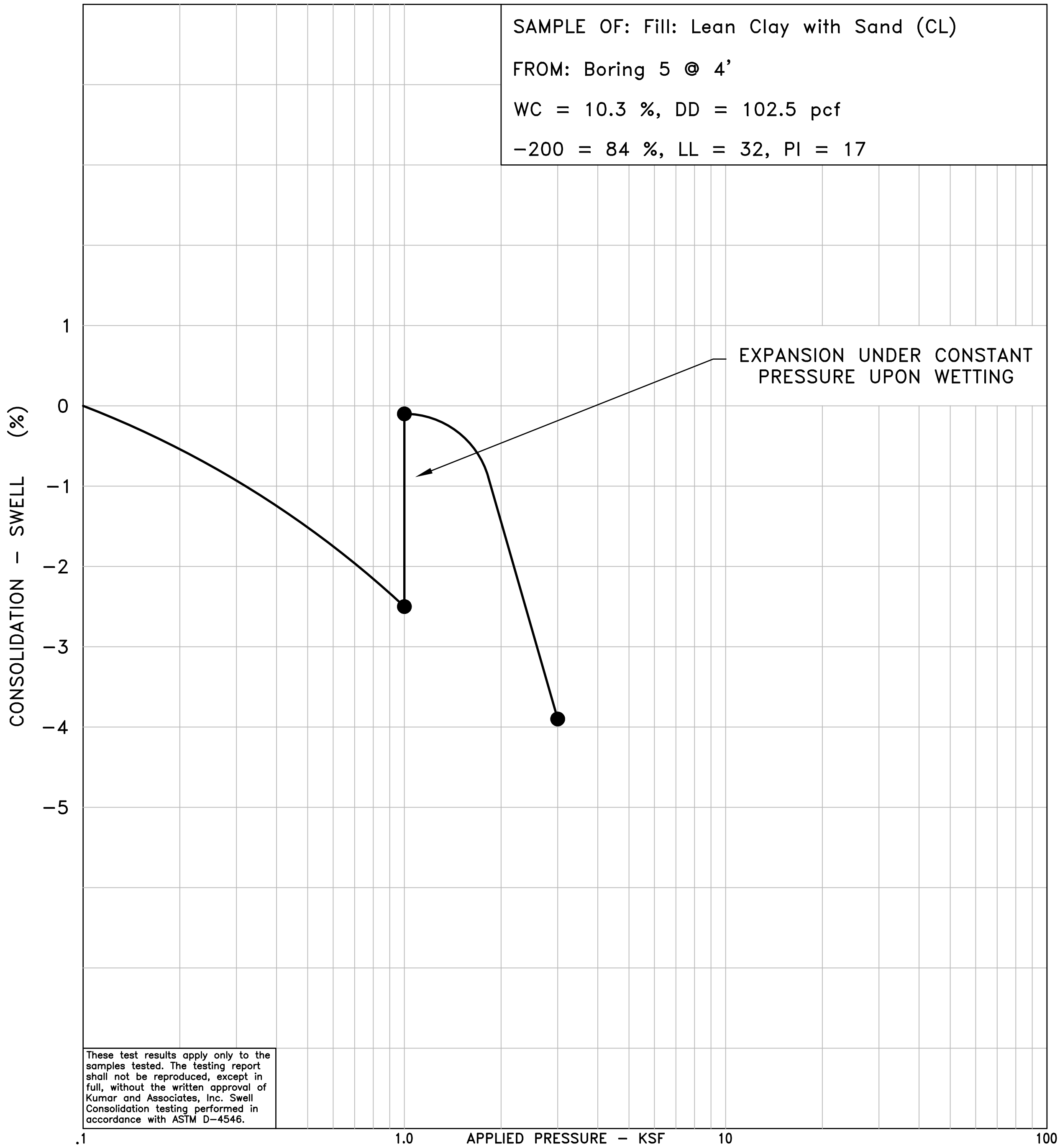
24−1−440	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULT	Fig. 4



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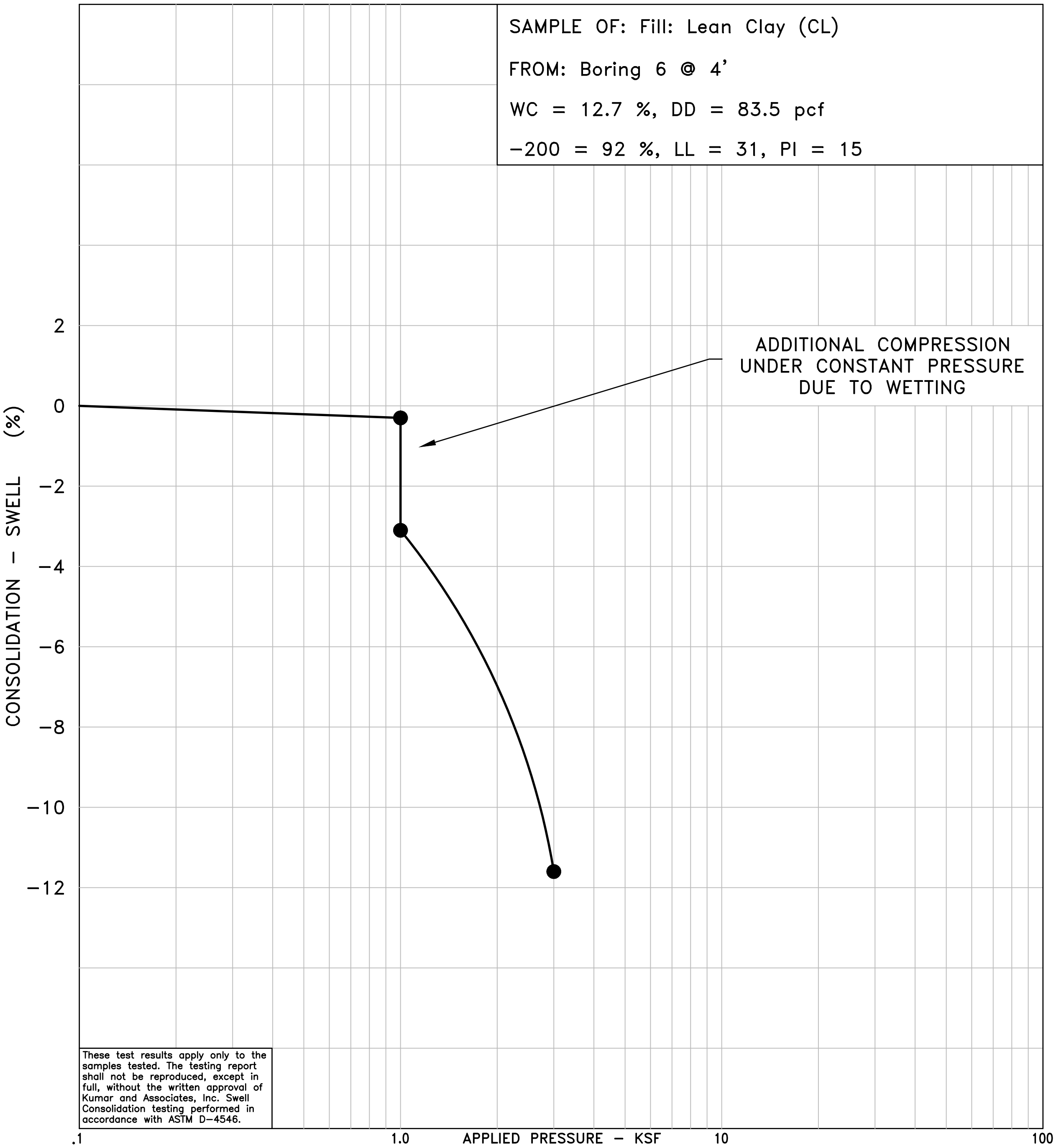
24-1-440	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULT	Fig. 5



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.

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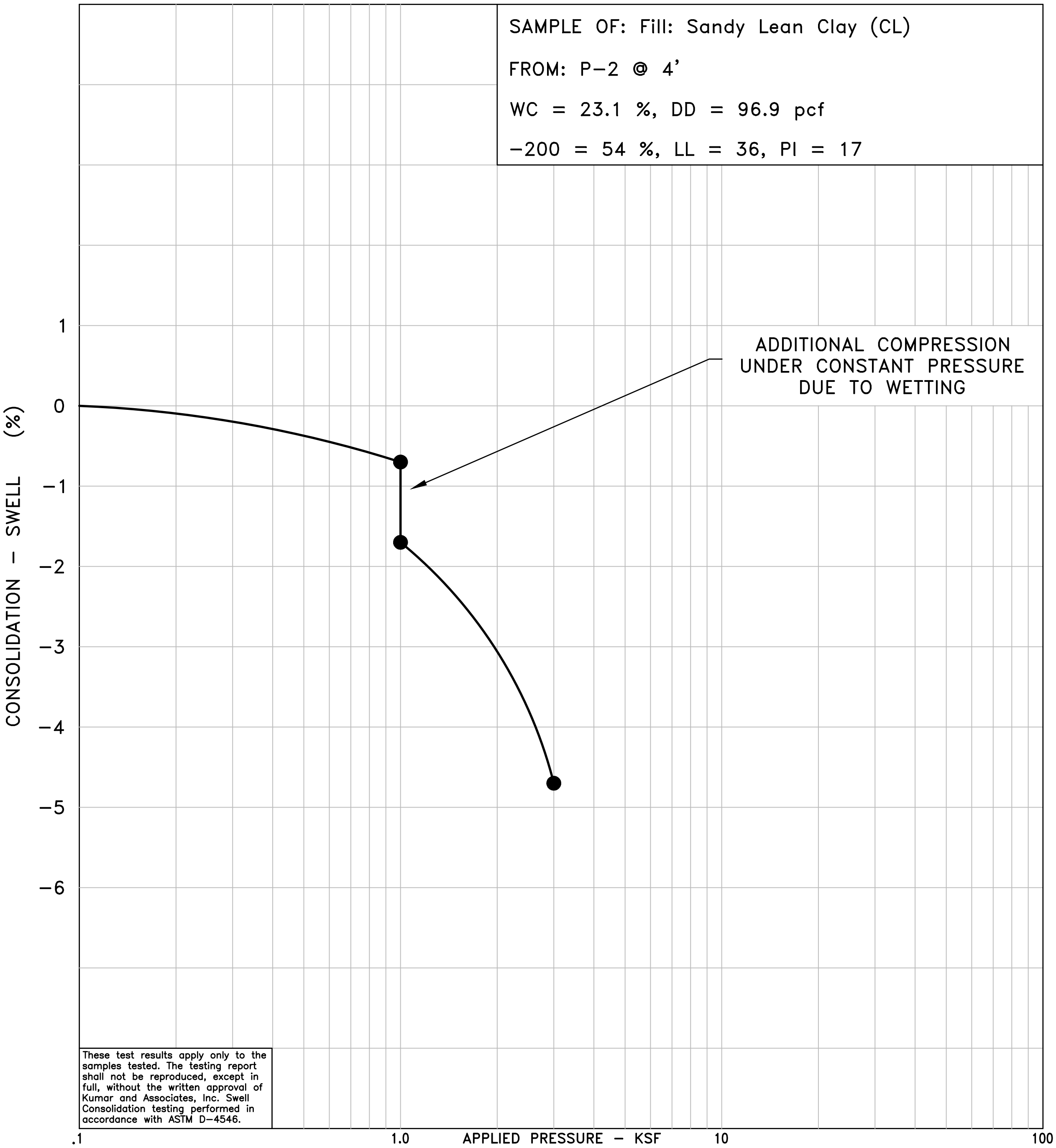
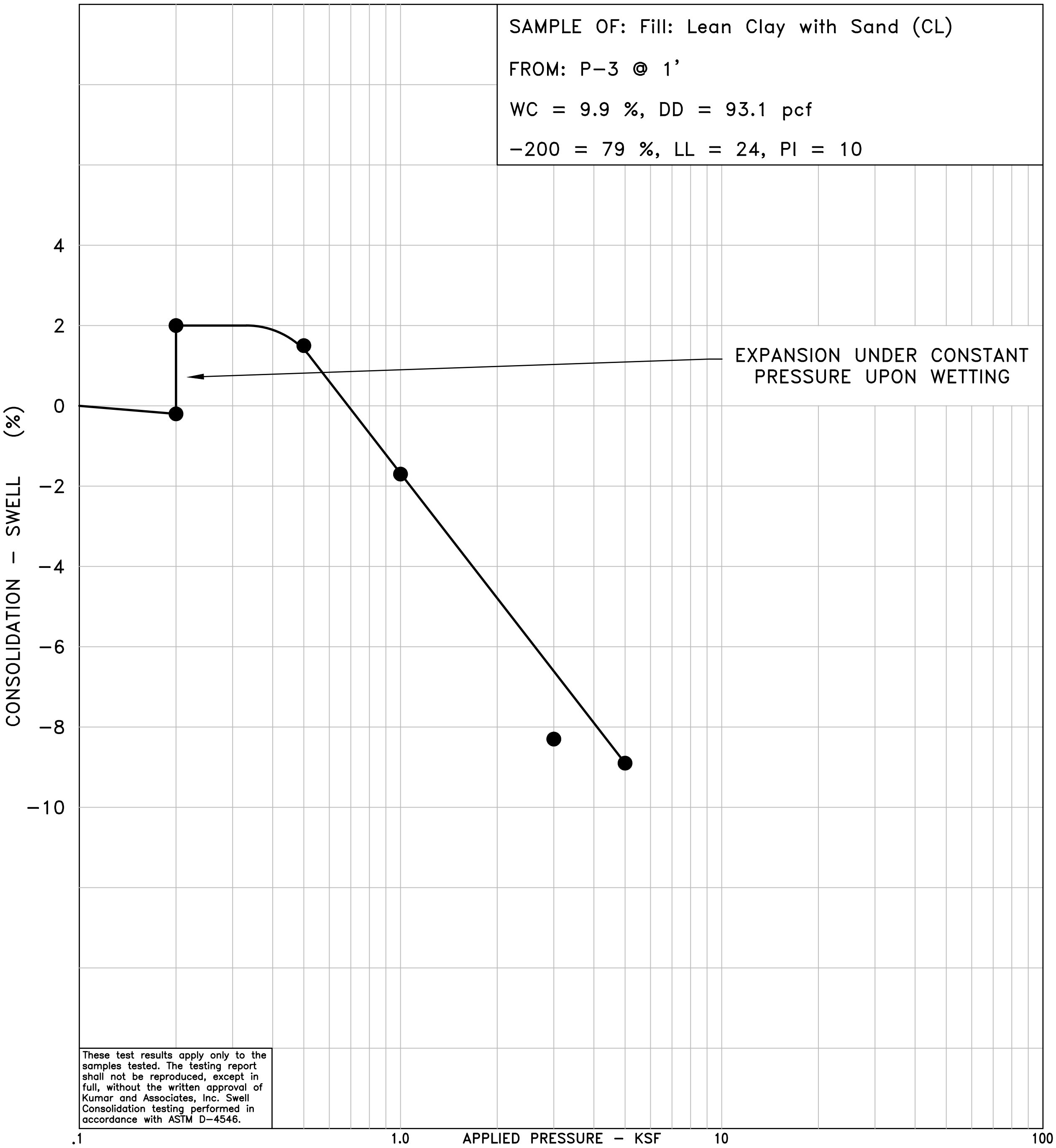
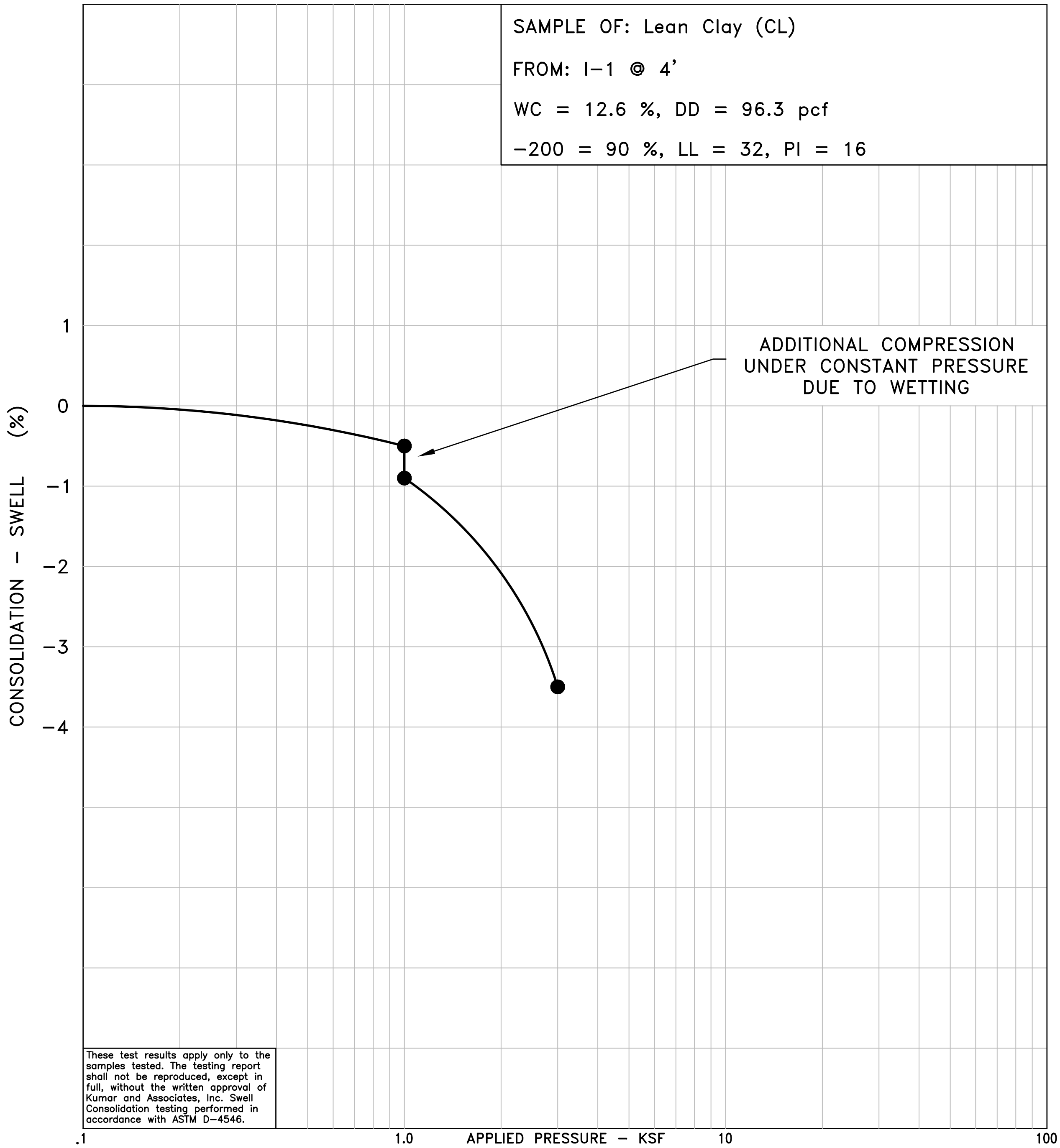
24−1−440	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULT	Fig. 6



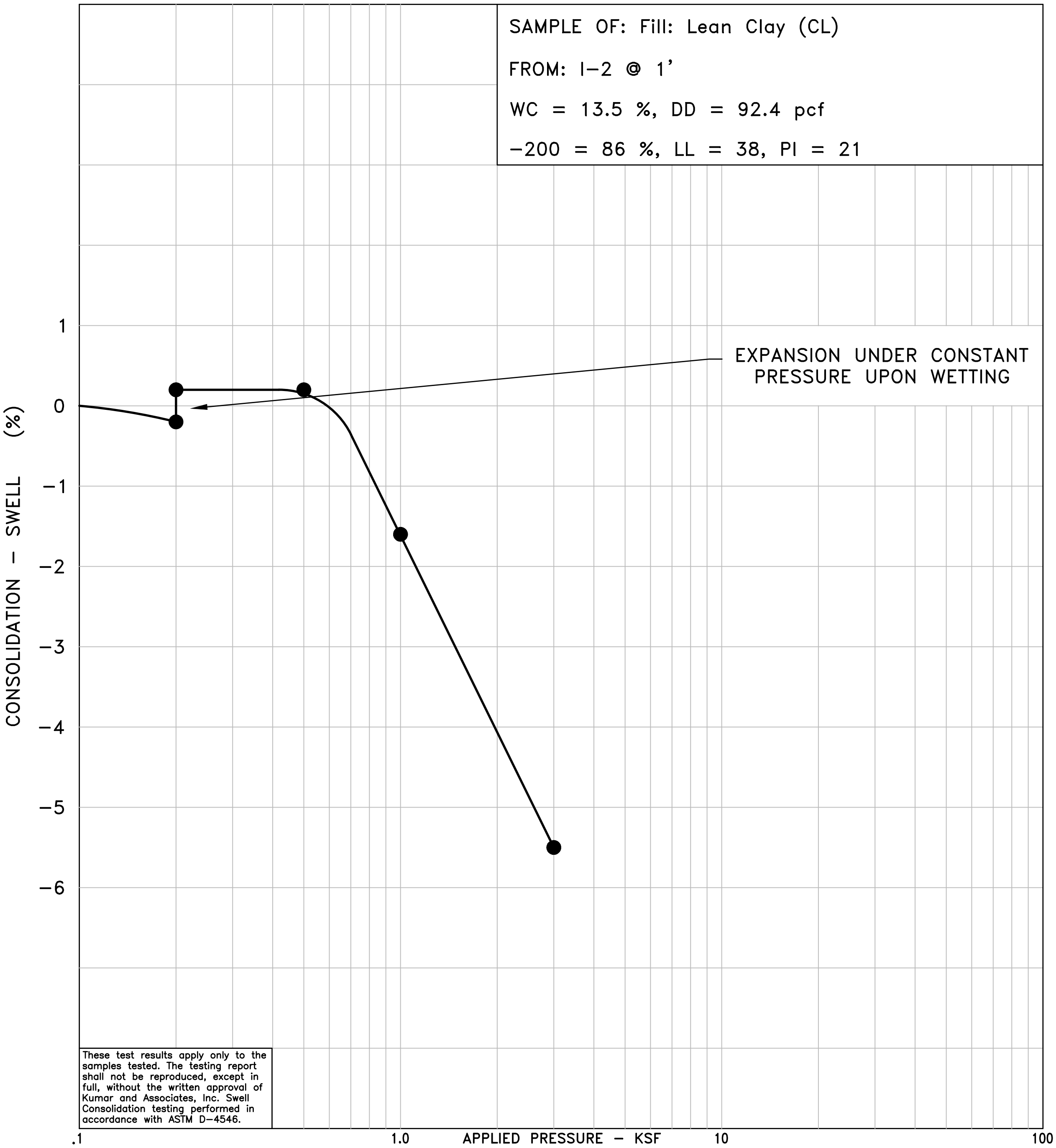
24-1-440	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULT	Fig. 7



24-1-440	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULT	Fig. 8



24-1-440	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULT	Fig. 9



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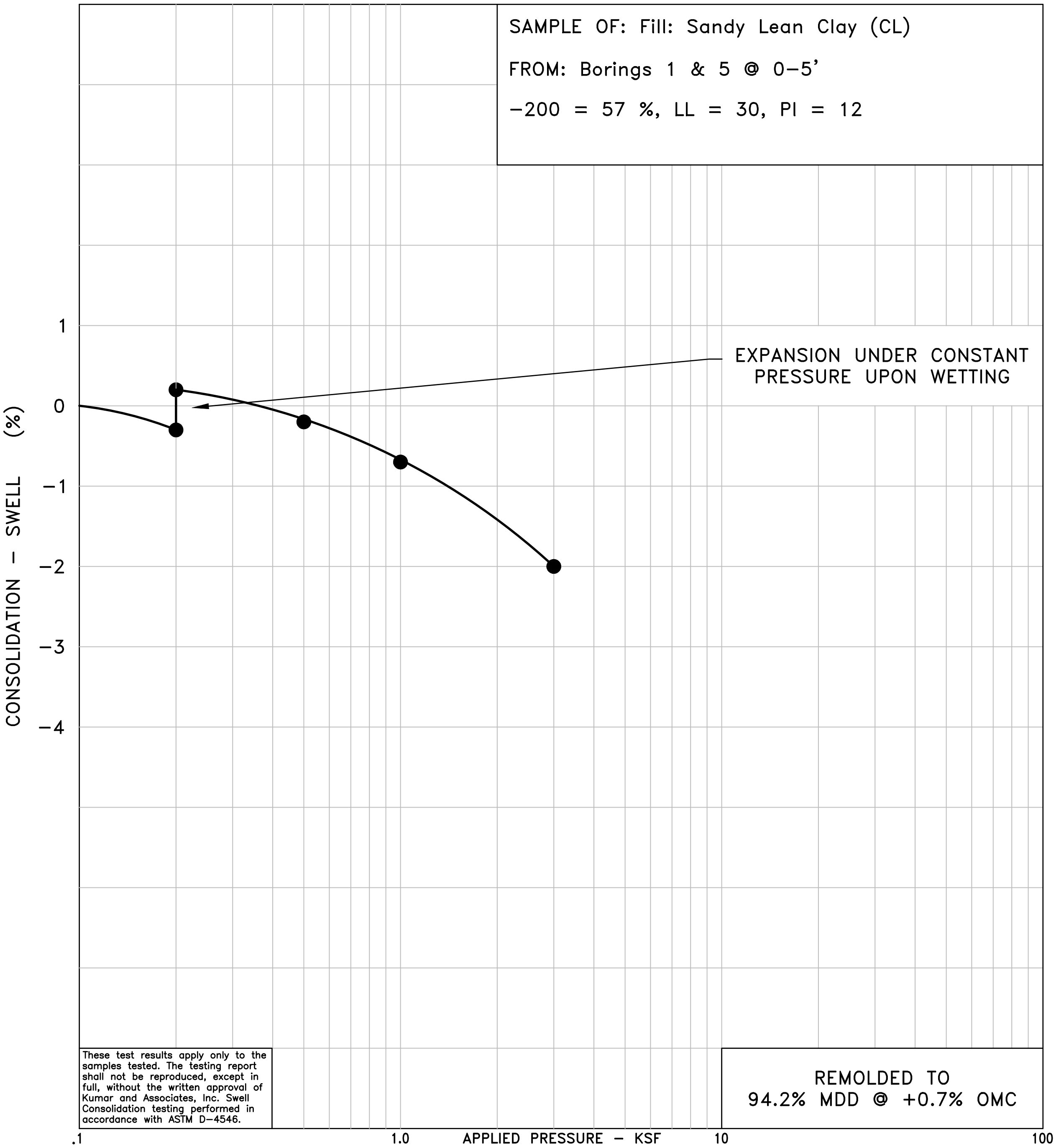
24-1-440	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULT	Fig. 10



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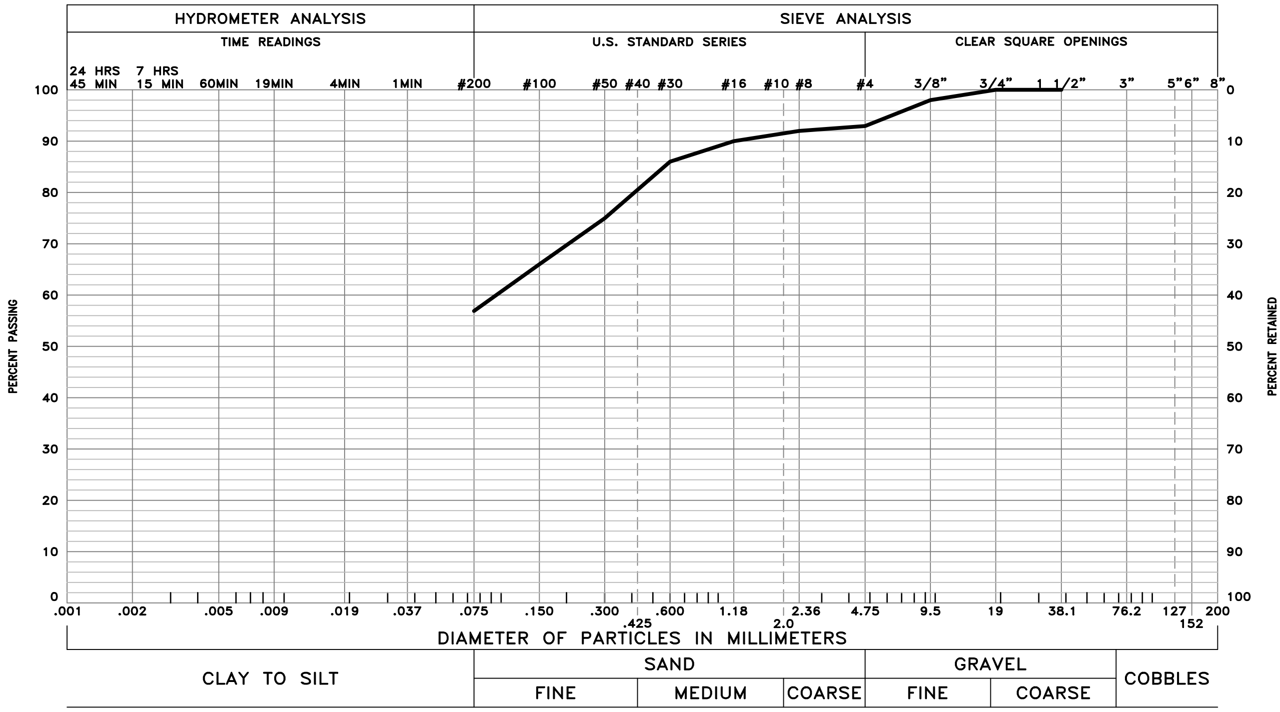
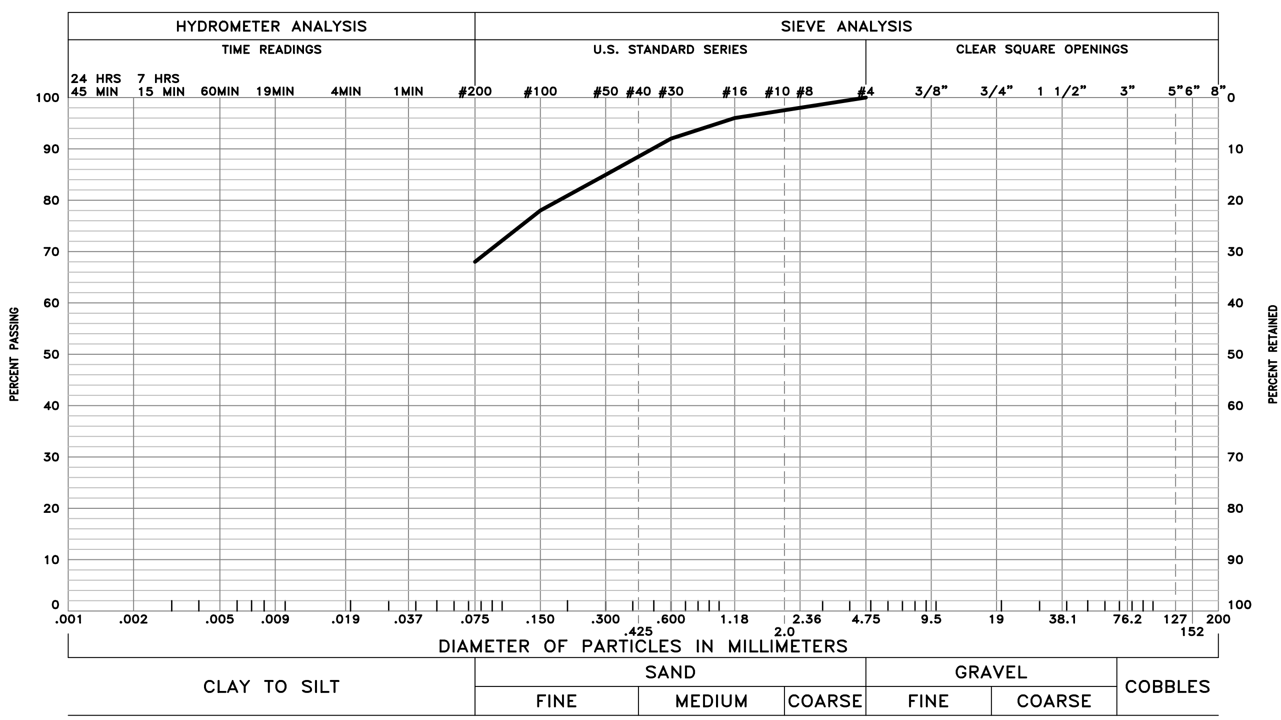
24-1-440	Kumar & Associates	REMOLDED SWELL-CONSOLIDATION TEST RESULT	Fig. 11



GRAVEL 7 % SAND 36 % SILT AND CLAY 57 %

LIQUID LIMIT 30 PLASTICITY INDEX 12

SAMPLE OF: Fill: Sandy Lean clay (CL) FROM: Borings 1-5 @ 0-5'



GRAVEL 0 % SAND 32 % SILT AND CLAY 68 %

LIQUID LIMIT 69 PLASTICITY INDEX 14

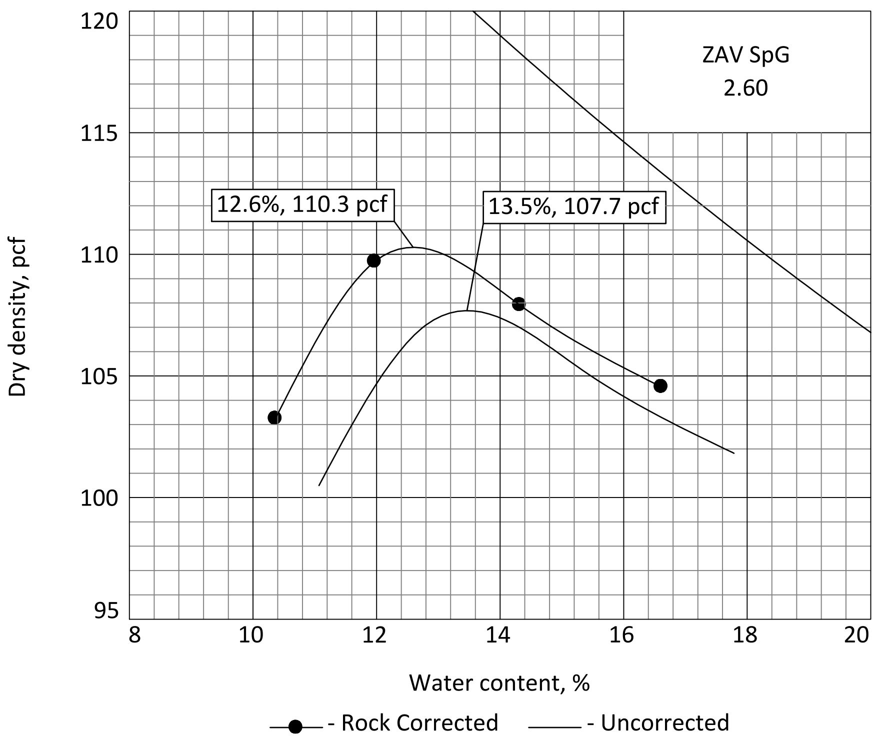
SAMPLE OF: Fill: Sandy Lean Clay (CL) FROM: Borings 11 & 12 @ 0-2'

 Sieve analysis testing is performed in accordance with ASTM D6913, ASTM D7928, ASTM C136 and/or ASTM D1140.

24-1-440	Kumar & Associates	GRADATION TEST RESULTS	Fig. 12

COMPACTION TEST REPORT

Curve No. 3522



Preparation Method	
Rammer: Wt.	5.5 lb.
Drop	12 in.
Type	Manual
Layers: No.	3
Blows per	25
Mold Size	0.03333 cu. ft.
Test Performed on Material Passing	#4 Sieve
%>#4	7
%<No.200	57
Atterberg (D 4318): LL	30
PI	12
NM (D 2216)	
Sp.G. (D 854)	2.6
USCS (D 2487)	CL
AASHTO (M 145)	A-6(4)
Date: Sampled	7/10/24
Received	7/11/24
Tested	7/12/24
Tested By	AS

COMPACTION TESTING DATA
ASTM D 698-12 Method A Standard
ASTM D4718-15 Oversize Corr. Applied to Each Test Point

	1	2	3	4	5	6
WM + WS	6050.0	6189.0	6198.0	6176.0		
WM	4359.0	4359.0	4359.0	4359.0		
WW + T #1	353.3	358.0	364.3	413.3		
WD + T #1	333.4	334.8	336.4	374.1		
TARE #1	153.6	153.5	154.3	153.7		
WW + T #2						
WD + T #2						
TARE #2						
MOIST.	10.4	12.0	14.3	16.6		
DRY DENS.	103.2	109.7	107.9	104.6		

SIEVE TEST RESULTS

Opening Size	% Passing	Specs.
1-1/2"	100	
3/4"	100	
3/8"	98	
#4	93	
#8	92	
#16	90	
#30	86	
#50	75	
#100	66	
#200	57	

ROCK CORRECTED TEST RESULTS	UNCORRECTED
Maximum dry density = 110.3 pcf	107.7 pcf
Optimum moisture = 12.6 %	13.5 %

Material Description

Sandy Lean Clay

Remarks:

These test results apply only to the samples which were tested. the testing report shall not be reproduced, except in full, without the written approval of Kumar and Associates, Inc. Moisture/density relationships performed in accordance with ASTM D698, D1557. Atterberg limits performed in accordance with ASTM D4318 sieve analysis performed in accordance with ASTM D422, D1140.

Project No. 24-1-440 **Client:**

Project: Stanley 98 Aurora

○**Location: Borings 1-5** **Depth: 0'-5'** **Sample Number: 3522**

Checked by: JJM

Title: Lab Manager

24–1–440	Kumar & Associates	MOISTURE–DENSITY RELATIONSHIPS	Fig. 13

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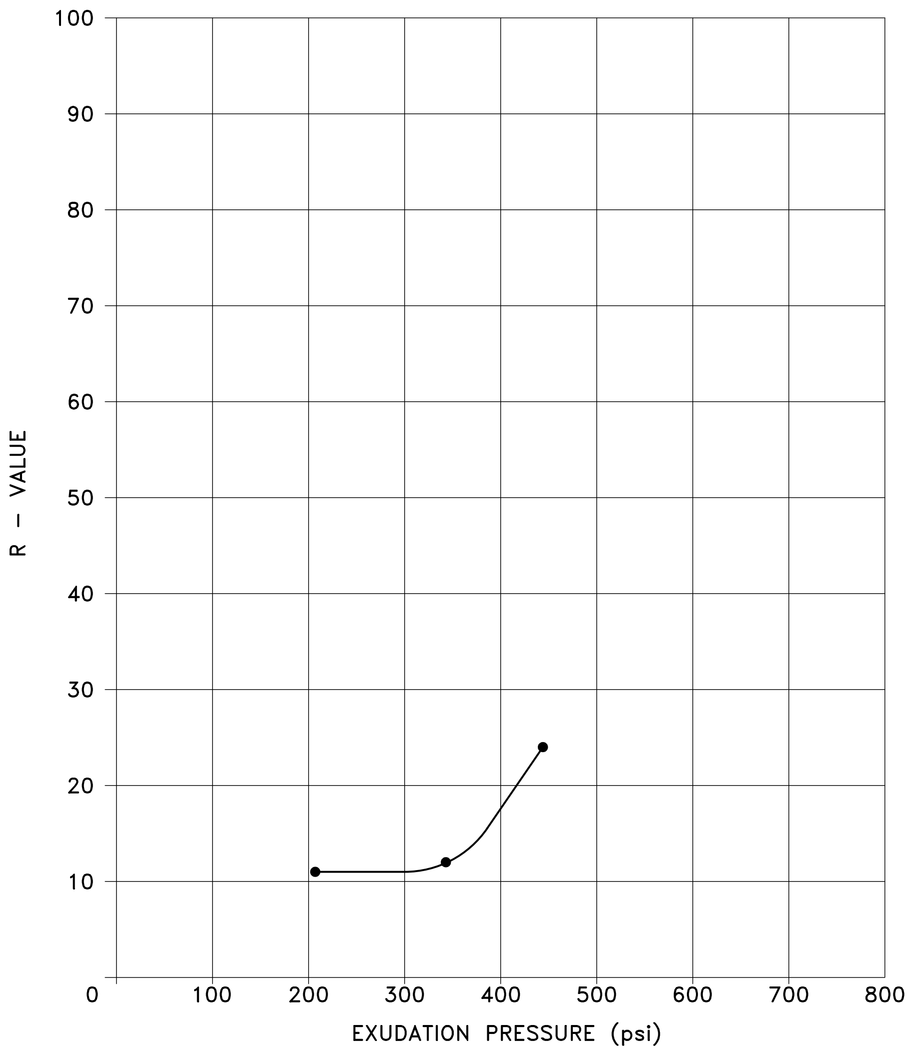
TEST SPECIMEN	1	2	3	4	R –VALUE (300 psi)
MOISTURE CONTENT (%)	24.2	22.4	109.8		
DENSITY (pcf)	100.5	104.5	106.4		
EXPANSION PRESSURE (psi)	0.000	0.000	0.000		
EXUDATION PRESSURE (psi)	207	343	444		
R VALUE	11	12	24		11



SOIL TYPE: Fill: Sandy Lean Clay

LOCATION: Borings I1 & I2 @ 0–2'

DATE SAMPLED: 07/12/24 DATE RECEIVED: 07/15/24 DATE TESTED: 07/17/24

GRAVEL: 0 % SAND: 32 % SILT AND CLAY: 68 %

LIQUID LIMIT: 36 PLASTICITY INDEX: 14

These test results apply only to the samples which were tested. The testing report shall not be reproduced, except in full, without the written approval of Kumar & Associates, Inc. R–value performed in accordance with ASTM D2844. Atterberg limits performed in accordance with ASTM D4318. Sieve analyses performed in accordance with ASTM D422, D1140.

24–1–440	Kumar & Associates	HVEEM STABILOMETER TEST RESULTS	Fig. 14

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Table I
Summary of Laboratory Test Results

Project No.: 24-1-440
Project Name: Stanley 98 Aurora - East 25th Drive and Joliet Street, Aurora, Colorado
Date Sampled: 7/10/2024 & 7/12/2024
Date Received: 7/11/2024 & 7/15/2024

Sample Location		Date Tested	Natural Moisture Content (%)	Natural Dry Density (pcf)	Gradation		Percent Passing No. 200 Sieve	Atterberg Limits		Water Soluble Sulfates (%)	R-Value	AASHTO Classification (Group Index)	Soil or Bedrock Type
Boring	Depth (Feet)				Gravel (%)	Sand (%)		Liquid Limit (%)	Plasticity Index (%)				
1	4	7/12/2024	9.5	76.7			93	32	19	0.00		A-6 (16)	Fill: Lean Clay (CL)
1	14	7/12/2024	8.8	107.9			34	33	17				Clayey Sand (SC)
2	9	7/12/2024	7.1	113.8			43	25	11				Clayey Sand (SC)
3	4	7/12/2024	9.5	96.3			78	31	14			A-6 (9)	Fill: Lean Clay with Sand (CL)
3	14	7/12/2024	15.6	112.0			58	38	21				Sandy Lean Clay (CL)
4	9	7/12/2024	10.3	116.1			59	29	15				Sandy Lean Clay (CL)
5	4	7/12/2024	10.3	102.5			84	32	17			A-6 (13)	Fill: Lean Clay with Sand (CL)
1-5	0-5	7/12/2024	12.6*	110.3*	7	36	57	30	12			A-6 (4)	Fill: Sandy Lean Clay (CL)
6	4	7/12/2024	12.7	83.5			92	31	15			A-6 (13)	Fill: Lean Clay (CL)
P-1	1	7/12/2024	8.2	95.9	5	30	65	28	13			A-6 (6)	Fill: Sandy Lean Clay (CL)
P-2	4	7/12/2024	23.1	96.9			54	36	17			A-6 (6)	Fill: Sandy Lean Clay (CL)
P-3	1	7/12/2024	9.9	93.1			79	24	10			A-4 (5)	Fill: Lean Clay with Sand (CL)
I1	4	7/12/2024	12.6	96.3			90	32	16			A-6 (13)	Lean Clay (CL)
I2	1	7/12/2024	13.5	92.4			86	38	21			A-6 (18)	Fill: Lean Clay (CL)
I1 & I2	0-2	7/17/2024			0	32	68	36	14		11	A-6 (8)	Fill: Lean Clay (CL)

* - Optimum moisture content and maximum dry density as determined by standard Proctor (ASTM D 698)