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GEOTECHNICAL ENGINEERING STUDY
AND PAVEMENT THICKNESS DESIGN
DOVE VALLEY INDUSTRIAL BUILDING
8001 SOUTH CHAMBERS ROAD
ENGLEWOOD, COLORADO

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SUMMARY

1. The subsurface conditions were explored by drilling a total of 8 exploratory borings at the approximate locations shown on Fig.1. The borings encountered a relatively thin layer of topsoil overlying either pre-existing fill or naturally deposited (native) clayey soil. The fill, where encountered, extended approximately 2 feet below the existing ground surface. The native clay soil extended about 8 to 9.5 feet below existing grades. In Borings 2 and P-2, the native clay soil was underlain by native silty clayey sand to a depth of about 18 and 9 feet, respectively. Bedrock was generally encountered in the borings drilled for the building at depths of approximately 9 to 9.5 feet, with the exception of Boring 2 where bedrock was encountered at a depth of about 18 feet.

   Groundwater was encountered in Boring 6 at the time of drilling at a depth of about 14 feet below the ground surface. At the completion of drilling, Boring 1 was left open for a follow-up, stabilized groundwater measurement. The follow-up measurement, made 12 days later indicated no groundwater within the boring. At the request of the land owner, the remaining borings were immediately backfilled after drilling due to cattle being placed on the property. As a result, we were not able to obtain follow-up groundwater measurements in those borings.

2. Due to the moderate to very high swell potential of the on-site soils, the safest approach for building support on the site would be to use deep foundations such as drilled piers extending into bedrock, or helical piers bearing in dense sand or bedrock. As an alternative, the building may be founded on shallow spread footings placed on a minimum of 7 feet zone of structural fill extending to undisturbed native soils provided the risk of distress resulting from potential additional movement can be accepted by the Owner. Recommendations for drilled pier and spread footing foundations are presented in this report.

3. Highly expansive soils are present at the site. The most positive method for to avoid damage as a result of floor slab movement is to construct a structural floor above a well-ventilated crawl space. Based on the moisture-volume change characteristics of the materials encountered, we believe slab-on-grade floors are acceptable for the proposed building provided the risk of distress resulting from slab movement is accepted by the Owner. We believe the swelling potential of the expansive overburden soils and movement of slab-on-grade floor slabs can be reduced by placing the slab on a minimum of 10 feet of prepared subgrade.

   If the potential for floor slab settlement or uplift is not acceptable, a structural floor should be used. Kumar & Associates, Inc. can provide detailed recommendations for structural floors, if desired.

4. The recommended pavement sections are as follows:

Area	**Full-depth HMA *(in.)***	**Composite HMA over ABC *(in.)***	**PCCP *(in.)***
Auto Parking	6.0	4.0 over 8	6.0
Auto Drive & Fire Lanes	6.5	4.5 over 8	7.0
Truck Traffic and Loading	8.0	5.0 over 10	7.0

   Dumpster pads and any areas subjected to concentrated truck turning movements should be paved using a minimum of 7.0 inches of Portland cement concrete.

PURPOSE AND SCOPE OF STUDY

This report presents the results of a geotechnical engineering study and pavement thickness design for the proposed Dove Valley Industrial Building to be constructed at 8001 South Chambers Road in Englewood, Colorado. The project site is shown on Fig. 1. This study has been performed in general accordance with our Proposal No. P-22-215 to Brennan Investment Group, dated February 10, 2022.

A field exploration program consisting of exploratory borings was conducted to obtain information on the subsurface conditions. Samples of the soils and bedrock obtained during the field exploration program were tested in the laboratory to determine their classification properties. Results of the field exploration and laboratory testing program were evaluated to develop recommendations for foundation types, depths and allowable pressures for the proposed building foundation, floor slabs, and site pavements. The results of the field exploration and laboratory testing program are presented herein.

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

PROPOSED CONSTRUCTION

Based on the information provided to us, we understand the project will include the construction of a proposed single-story, high-bay industrial warehouse/distribution building to be constructed on approximately 8.2 acres of vacant land located at 8001 South Chambers Road. At the time of this report, there are two building layouts under consideration. The preferred alternative includes a 102,960 square foot building with automobile parking on the north, east, and west sides of the building. Truck docks and loading areas will be constructed on the south side of the building. The second alternative includes a 79,560 square foot building with similar parking and truck pavements. The second alternative includes 33 trailer parking stalls further south of the truck dock and loading areas.

We assume the proposed development will be consistent with typical warehouse/industrial building construction, utilizing pre-cast tilt-up construction surrounding a large floor area. The building is assumed to have a relatively long roof span and a high ceiling. The building will not have any below ground construction. We anticipate the building will have moderate foundation loads, generally consistent with the proposed type of construction.

At the time of this report, a grading plan showing the finished floor elevations was not available. Based on the site topography, we assume the project will require cuts and fills of at least a few feet to establish final building and pavement grades.

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

SITE CONDITIONS

The site is located on approximately 8.2 acres of vacant land and is bounded on the north by South Chambers Road, to the east by a similar unoccupied lot, to the west and south by existing warehouse buildings. At the time of drilling, the site was vegetated with grasses and weeds. Topography at the site was sloping down toward the northeast with an overall elevation difference of about 24 feet.

SUBSURFACE CONDITIONS

Field Exploration Program: The subsurface conditions were explored by drilling a total of 8 exploratory borings at the site. Six (6) of the borings were drilled within the limits of the proposed building footprint, and two (2) borings were drilled within the limits of proposed truck dock pavement area on the south side of the building. The approximate locations of the exploratory borings are shown on Fig. 1.

The exploratory borings were advanced into the overburden soils and into the bedrock, where encountered, using 4-inch diameter continuous flight augers. Samples were obtained using a 2-inch-I.D. California barrel sampler. The sampler was driven with blows from a 140-pound hammer falling 30 inches. The split sampling procedure is similar to the standard penetration test described by ASTM International (ASTM) D1586. Penetration resistance values indicate the relative density or consistency of the soils.

Depths at which samples were taken and the associated penetration resistance values are shown on the Logs of Exploratory Borings on Figs. 2 and 3. A legend and explanatory notes associated with the graphic logs and describing the soils encountered are presented on Fig. 4.

Subsurface Conditions: The borings encountered a relatively thin layer of topsoil overlying either pre-existing fill or naturally deposited (native) clayey soil. The fill, where encountered, extended approximately 2 feet below the existing ground surface. The native clay soil extended about 8 to 9.5 feet below existing grades. In Borings 2 and P-2, the native clay soil was underlain by native silty clayey sand to a depth of about 18 and 9 feet, respectively. Bedrock was generally encountered in the borings drilled for the building at depths of approximately 9 to 9.5 feet, with the exception of Boring 2 where bedrock was encountered at a depth of about 18 feet. The borings drilled for the building were terminated in the bedrock at depths of 20 to 25 feet and the borings drilled for the pavement were terminated in the native clays and granular soils at depths of 5 to 10 feet.

The fill material appears to be reworked native material from a source on or near the project site and consisted of lean clay with varying sand content, and occasional fat clay. The exact lateral and vertical extents of the fill, and the degree of compaction were not evaluated as part of this study. The native clay soils consisted of lean clays with varying sand content with occasional sandy fat clays. The clay soils were slightly moist to moist and very stiff to hard based on sampler penetration resistance values (blow counts) obtained during drilling. The granular clayey silty sand soils were fine- to coarse-grained, slightly moist to moist and medium dense to dense in consistency. The bedrock encountered consisted primarily of claystone. Sandstone bedrock was encountered in two of the borings drilled for the building. The claystone bedrock was slightly moist to moist and medium hard to hard. The sandstone bedrock was slightly moist and hard to very hard.

Groundwater was encountered in Boring 6 at the time of drilling at a depth of about 14 feet below the ground surface. At the completion of drilling, Boring 1 was left open for a follow-up, stabilized groundwater measurement. The follow-up measurement, made 12 days later indicated no groundwater within the boring. At the request of the land owner, the remaining borings were immediately backfilled after drilling due to cattle being placed on the property. As a result, we were not able to obtain follow-up groundwater measurements in those borings.

Groundwater levels may fluctuate upward in response to precipitation events and landscape irrigation.

LABORATORY TESTING

Selected samples obtained from the exploratory borings were visually classified by the project engineer. Laboratory testing was performed on selected samples to determine in-situ soil moisture content and dry density, liquid and plastic limits, swell-consolidation characteristics, moisture-density relationship (standard Proctor), and concentration of water-soluble sulfates. The results of the laboratory testing program are shown adjacent to the boring logs on Figs. 2 and 3, plotted graphically on Figs. 5 through 11, and summarized in Table I. The testing was conducted in general accordance with recognized ASTM and CDOT test procedures.

Swell-Consolidation Testing: Swell-consolidation testing was conducted on samples of the on-site clay soils and claystone bedrock in order to determine their compressibility and/or swell characteristics under loading and when wetted. Additionally, swell-consolidation testing was conducted on clayey samples remolded to near the maximum dry density at, and slightly above the optimum moisture content to help determine the suitability of the on-site material for use as structural fill below building areas.

Each sample was prepared and placed in a confining ring between porous discs. A surcharge pressure of 1,000 psf was applied to each sample, and the samples were allowed to compress to a stabilized height before being submerged in water. The sample height was monitored until deformation practically ceased under each load increment. Results of the swell-consolidation tests are plotted as a curve of the final strain at each increment of pressure against the log of the pressure.

Based on the results of the laboratory swell-consolidation testing, a sample of the claystone bedrock exhibited very high swell potential (6.1%) upon wetting under a 1,000-psf surcharge pressure. Tested samples of the on-site clays yielded high to very high potential (5.2% to 8.2%) under the same surcharge pressure. The high swell potential exhibited by the samples is considered in part due to the relatively high in-situ densities and low in-situ moisture contents of the samples. The tested remolded sample of the on-site clay near optimum moisture content exhibited low swell potential under a surcharge pressure of 200 psf. A sample remolded to 2 percent above optimum moisture content exhibited additional consolidation when submerged upon loading, which we believe to be the result of sample disturbance during handling. Results of the swell-consolidation tests are presented on Figs. 5 through 10.

Water-Soluble Sulfates: The concentrations of water-soluble sulfates measured in samples obtained from the exploratory borings ranged from 0.04% to 0.13%. These concentrations of water-soluble sulfates represent a Class S0 and Class S1 severity exposure of sulfate attack on concrete exposed to these materials. These 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-16.

Based on the laboratory data and our experience, we recommend all concrete exposed to the on-site materials meet the cement requirements for Class S1 exposure as presented in ACI 201.2R.16. Alternatively, the concrete could meet the Colorado Department of Transportation's (CDOT) cement requirements for Class 1 exposure as presented in Section 601.04 of the latest CDOT Standard Specifications for Road and Bridge Construction.

GEOTECHNICAL ENGINEERING CONSIDERATIONS

Expansive Soils and Bedrock: At the time of this report, a grading plan for the building had not been generated for the project. Based on conditions encountered in the borings, we anticipate subsurface conditions at foundation and floor slab levels to consist primarily of highly- to very-highly swelling clays above relatively shallow expansive bedrock.

Foundations and Floor Slabs: Foundations placed on expansive material such as the on-site clays soils are likely to experience movement in excess of normally accepted tolerances should the soils become subject to moisture changes. The safest approach to limit potentially excessive foundation movement due to potential moisture-related expansion is to support the building on a deep foundation system using straight-shaft piers drilled into bedrock. Using a deep foundation system has the advantage of bottoming the piers in a zone of relatively stable moisture content and concentrating the loads to help offset uplift forces from expansive soil and bedrock. Based on our project experience of facilities of this nature in the Dove Valley area, drilled piers are often selected as the foundation of choice over spread footings.

Floor slabs present a problem where expansive materials are present near floor slab elevation because sufficient dead load cannot be imposed on them to resist the uplift generated when the materials are wetted and expand. The most positive method to avoid slab damage as a result of ground heave is to construct a structural floor above a well-vented crawl space. The structural floor would be supported on grade beams and piers the same as the main structure. Alternatively, a “slab-on-void” construction approach may be used by constructing a structurally supported reinforced slab on void form material.

Given the size of the building a structural floor system may be cost prohibitive to the project. Based on our experience, we believe slab-on-grade floors supported on a zone of compacted fill should be a practical and cost-effective alternative to structural floors for the proposed building. Additionally, the relatively deep over-excavation required for slab-on-grade construction would also allow the use of a shallow foundation system bearing on the compacted fill. Recommendations for shallow spread footing and drilled pier foundations are presented in the following section of this report.

The use of shallow foundations and slab-on-grade floors will require significant over-excavation beneath the foundation and slab subgrade elevation, and backfilling with a zone of moisture-treated and compacted fill. Acceptable performance will also rely on minimizing water infiltration into the expansive soils by providing good surface and subsurface drainage and implementing sensible landscaping and irrigation practices.

Potential Slab-on-Grade and Foundation Movement: The following discussion presents estimates of ground heave to aid in the decision-making process for foundation and floor support systems. The risk of ground heave can be mitigated to a certain degree by providing a zone of compacted fill with nil to low swell potential directly beneath foundations and floor slabs. Heave estimate calculations can be useful in evaluating the relative effectiveness of varying the thickness of this prepared fill zone. However, such calculations cannot address the uncertainty in the potential depth and degree of wetting that may occur under beneath the building or the likelihood of a highly variable swell potential across a site.

We have performed calculations for a range of scenarios of depth of wetting and over-excavation and backfill combinations to demonstrate the potential for ground heave if the soils beneath the building should be thoroughly wetted to significant depth, including below the depth of the prepared fill zone. Fills consisting of the on-site clays placed at moisture contents at or above optimum (ASTM D698) will likely have reduced swell potentials. The following table presents estimates of potential heave based on the results of swell-consolidation tests assuming different combinations of non- to low-swelling clay fills with a low swell potential using test and analysis methods generally accepted in the Colorado Front Range. Both depth of wetting and depth of the prepared fill were considered as variables in the analysis.

Treatment Option	Calculated Ground Heave (in.)		
	10 ft. Depth of Wetting	15 ft. Depth of Wetting	20 ft. Depth of Wetting
No over-excavation or moisture treatment	9.6	12.6	15.3
8 ft. over-excavation: Backfill with moisture-treated on-site soil with <2% swell*	1.9	4.7	7.1
10 ft. over-excavation: Backfill with moisture-treated on-site soil with <2% swell*	0.8	3.2	5.9
10 ft. over-excavation: Backfill with moisture-treated on-site soil with <1% swell*	0.4	2.8	5.4
10 ft. over-excavation: Backfill with 5 ft. of non-expansive fill over 5 ft. of moisture-treated on-site soil with 2% swell*	0.2	2.6	5.3
*Percent swell when wetted under a 200 psf surcharge pressure.			

It should be noted the heave estimates presented above are for floor slabs, which are generally lightly loaded; the amount of heave that would be experienced by shallow foundations would be somewhat less because the footing dead load would provide some resistance to swelling.

The heave estimate calculations demonstrate significant heave should be expected if thorough wetting of the native clay soils below the bottom of the prepared fill zone occurs, particularly if wetting extends to significant depths. However, our experience indicates the native soils underlying the slabs on the large majority of sites with similar subsurface conditions do not experience extreme moisture increases to significant depth provided good surface and subsurface drainage is designed, constructed and maintained, and good irrigation practices are followed. The risk could be further reduced by eliminating landscape irrigation within about 15 to 20 feet of the building and limiting irrigation elsewhere on site. Wetting can also occur as a result of unforeseeable influences such as plumbing leaks or breaks, or in some cases even due to off-site influences depending on geologic conditions.

Considering the above discussion, we believe spread footing foundations and soil-supported floor slabs may be considered for the project, provided the potential for foundation or floor slab movement due to ground heave and associated possible distress is recognized and understood by the Owner. The intent of our recommendations for spread footing foundations and soil-supported floor slabs is to provide for conditions where there is a good chance ground heave beneath the building will not exceed amounts acceptable to the Owner. The recommendations should result in heave movements that do not exceed 1 inch and are unlikely to significantly exceed 2 inches unless extreme wetting is allowed. Barring unforeseen events, we do not believe extreme wetting is likely to occur if the surface drainage and irrigation recommendations presented in this report are followed.

The Owner should understand and accept the risk of distress resulting from some foundation and slab movement even though mitigation measures are used to reduce the potential for building and slab distress resulting from ground heave. If the potential for foundation settlement or uplift is not acceptable, a deep foundation system in combination with a structural floor should be used. Kumar & Associates can provide detailed recommendations for structural floors, if desired.

FOUNDATION RECOMMENDATIONS

As indicated above, the safest foundation system to mitigate potential foundation movement due to expansive soils is to support the building on straight-shaft piers drilled into bedrock. Using a straight-shaft pier type of foundation, each column is supported on a single drilled pier and the building walls are founded on grade beams supported by series of piers. Load applied to the piers is transmitted to the bedrock partially through peripheral shear stresses which develop on the sides of the pier and partially through end bearing pressure. Straight-shaft piers have the advantage of providing high supporting capacity.

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 be designed for allowable end-bearing soil pressure of 25,000 psf and an allowable skin friction of 2,500 psf for the portion of pier penetrating bedrock. Uplift due to structural loadings on the piers can be resisted by using 75% of the allowable skin friction value plus an allowance for pier weight.

2. Piers should also be designed for a minimum dead load pressure of 25,000 psf based on pier end area only. Application of dead load pressure is the most effective way to resist foundation movement due to swelling soils and bedrock. However, if the minimum dead load requirement cannot be achieved and the piers are spaced as far apart as practical, the pier length should be extended beyond the minimum bedrock penetration and minimum length to mitigate the dead load deficit. This can be accomplished by assuming one-half of the skin friction value given above acts in the direction to resist uplift caused by swelling soil or bedrock near the top of the pier. The owner should be aware of an

increased potential for foundation movement if the recommended minimum dead load pressure is not met.

3. Piers should penetrate at least 10 feet into the bedrock and have a minimum pier length of 20 feet. Both requirements for minimum bedrock penetration and minimum pier length should be met.

4. Piers should be designed to resist lateral loads using a modulus of horizontal subgrade reaction in the fill of 50 tcf and a modulus of horizontal subgrade reaction of 250 tcf in the bedrock. The modulus values given are for a long, one-foot-wide pier and must be corrected for pier size.

5. The lateral capacity of the piers may also 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 usages.

Material	C (psf)	Ø	γ_T	k_s	k_c	ϵ_{50}	Soil Type
Imported Non-expansive Structural Fill	0	32	130	225	225	-	1
On-site clay soils	500	0	120	500	200	0.007	2
Bedrock	8,000	0	125	2,000	800	0.004	2

c Cohesion intercept (pounds per square foot)
Ø Angle of internal friction (degrees)
γ_T Total unit weight (pounds per cubic foot)
k_s Initial static modulus of horizontal subgrade reaction (pounds per cubic inch)
k_c Initial cyclic modulus of horizontal subgrade reaction (pounds per cubic inch)
ϵ_{50} Strain at 50 percent of peak shear strength

Soil Types:
1. Sand above the water table (Reese)
2. Stiff clay without free water (Reese)

6. Closely-spaced piers and pier groups 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 3 pier diameters center-to-center for axial loading, and 5 pier diameters center-to-center in the direction both parallel and perpendicular to lateral loading. More closely spaced piles should be studied on an

individual basis to determine the appropriate reduction in axial and lateral load design parameters.

7. If the minimum pier spacing recommended above for lateral loading cannot be achieved, we recommend that the lateral 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 pile capacity. The modified p-y curve should then be reentered into the L-Pile software to calculate the pile 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 spacing of piers in the group in the direction of loading expressed in multiples of the pier diameter, we recommend p-multipliers of 0.8 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, 0.8 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 generally consistent with Table 10.7.2.4-1 of the 2017 AASHTO LRFD Bridge Design Specifications (8^{th} Edition). 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.

8. Piers should be reinforced their full length to resist an un-factored net tensile force from swelling soil pressure of at least 75 kips. The recommended tensile force is for a 1-foot diameter pier and should be increased in proportion to the pier diameter for larger piers. If the design dead load greater than or less than the recommended dead load, the requirement for tension reinforcement should be decreased or increased accordingly to account for the difference.

9. A minimum 6-inch void should be provided beneath the grade beams to concentrate pier loadings. Absence of a void space will result in a reduction in dead load pressure, which

could result in upward movement of the foundation system. A similar void should also be provided beneath necessary pier caps.

10 The pier length-to-diameter ratio should not exceed 30 to facilitate proper cleaning and observation of the pier hole.

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

12. 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.

13. Care should be taken so that the pier shafts are not oversized at the top. Mushroomed pier tops can reduce the effective dead load on the piers.

14. The general lack of water in the exploratory borings indicates the use of temporary casing or dewatering equipment in the pier holes will likely not be required. However, if water infiltration does occur, the requirements for casing can sometimes be reduced by placing concrete immediately upon cleaning and observing the pier hole. In no case should concrete be placed in more than 3 inches of water unless placed by an approved tremie method.

15. Concrete should be placed in piers the same day they are drilled. Failure to place concrete 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 identification of adequate bedrock strata and monitor pier construction procedures.

Spread Footings: Considering the discussion presented in the “Geotechnical Considerations” section of this report and the subsurface conditions encountered in the exploratory borings, a feasible alternative to drilled pier foundations would be to support the building on shallow spread footings. If this alternative is selected, we recommend footings be placed on a minimum of 7 feet of properly compacted structural fill extending to undisturbed natural soils.

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

1. Footings supported as recommended herein should be designed for a net allowable soil bearing pressure of 3,000 psf.

2. Structural fill placed beneath footings should meet the material and placement requirements presented in the "Site Grading and Earthwork" section of this report.

3. Based on experience, we estimate total settlement for footings designed and constructed as discussed in this section will be less than 1-inch. Differential settlements across the building are estimated to be approximately ½ to ¾ of the total settlement.

4. Spread footings placed on properly compacted structural fill should have a minimum footing width of 24 inches for isolated pads and 16 inches for continuous footings.

5. Exterior footings 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 placed on properly compacted structural fill material as described will be a combination of the sliding resistance of the footing on the foundation materials and passive earth pressure against the side of the footing. Resistance to sliding at the bottoms of the footings can be calculated based on a coefficient of friction of 0.30. Passive pressure against the sides of the footings can be calculated using an equivalent fluid unit weight of 185 pcf, which is a working value. Compacted fill placed against the sides of the footings to resist lateral loads should consist of on-site or imported non- to low-swelling material placed and compacted in accordance with the criteria in the "Site Grading and Earthwork" section of this report.

7. Excessive wetting or drying of the foundation excavations should be avoided during construction. Care should be taken to provide adequate surface drainage during the excavation of footings, and the contractor should have equipment available for removing water from excavations following precipitation, if needed. Footing excavations that are

inundated as a result of uncontrolled surface runoff may soften, requiring possible additional moisture conditioning and re-compaction of the exposed subgrade soils, or removal of soft subgrade soils and replacement with new compacted structural fill.

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

9. A representative of the geotechnical engineer should observe all grading operations and footing excavations prior to concrete placement to determine adequate removal of the existing fill beneath foundations.

FLOOR SLABS

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

1. Floor slabs should be supported on a minimum 10-foot-thick layer of properly compacted structural fill meeting the material and placement criteria in the "Site Grading and Earthwork" section of this report.

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

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

   If wood or metal stud partition walls are used, the slip joints should preferably be placed at the bottoms of the walls so differential slab movement 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 pad-supported grade beams and to construct the slabs independently of the foundation. If slab-bearing partition walls are required, distress may be reduced by connecting the partition walls to the exterior walls using slip channels.

4. Floor slab control joints should be used to reduce damage due to shrinkage cracking. Joint spacing is dependent on slab thickness, concrete aggregate size, and slump, and should be consistent with recognized guidelines such as those of the Portland Cement Association (PCA) or American Concrete Institute (ACI). The joint spacing and slab reinforcement should be established by the designer based on experience and the intended slab use.

5. Floor slabs should not extend beneath exterior doors or over foundation grade beams, unless saw cut at the beam after construction.

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

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

8. The geotechnical engineer should evaluate the suitability of proposed under-slab fill material.

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.

SITE GRADING AND EARTHWORK

Temporary Excavations: We assume temporary excavations will be constructed by over-excavating the slopes to a stable configuration where enough space is available. All excavations should be constructed in accordance with OSHA requirements, as well as state, local and other applicable requirements. The clayey overburden soils will generally classify as Type B soils, and the silty clayey sands classify as Type C soils. The claystone and sandstone bedrock generally classify as Type A or Type B soils depending on the amount of fracturing and cementitious nature of the materials. Excavations encountering perched groundwater could require much flatter side slopes than those allowed by OSHA.

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

<u>Site and Building Subgrade Preparation</u>: In order to limit uplift of foundations and floor slabs caused by expansive soils and to reduce potential for resulting distress, the entire building should be supported on a layer of structural fill extending to a depth of 10 feet below slab subgrade level. The fill should consist of moisture-conditioned on-site lean clays, or imported structural fill material meeting the criteria presented in this section. The full-depth over-excavation for the compacted fill zone should extend beyond the outside of the building footprint a minimum distance equal to one-half of the depth of the compacted fill zone.

Site preparation beneath exterior flatwork and areas considered sensitive to movement should be done in accordance with the “Floor Slabs” section of this report. Subgrade preparation in less sensitive areas may be done in accordance with the “Pavement Design” section of this report.

All areas receiving new fill should be scarified to a depth of 12 inches and re-compacted to at least 95% of the standard Proctor (ASTM D698) maximum dry density at a moisture content recommended in this section. Excessive wetting and drying of excavations and prepared subgrade areas should be avoided during construction.

<u>Fill Material</u>: Unless specifically modified in the other sections of this report, the following recommended material and compaction requirements are presented for fill materials on the project site. A representative of the geotechnical engineer should evaluate the suitability of all proposed fill materials for the project prior to placement.

1. *Structural Fill*: Structural fill below building areas may consist of the moisture-conditioned, on-site clay soils. Claystone bedrock should not be reused beneath shallow foundations.

   Imported, non-expansive material should consist of soils with a maximum liquid limit of 30 and maximum plasticity index of 15. Imported fill source materials not meeting the above liquid limit and plasticity index criteria may be acceptable if the swell potential when remolded to 95% of the standard Proctor (ASTM D698) maximum dry density at optimum moisture content and wetted under a 200 psf surcharge pressure does not exceed 0.5%. Additional remolded samples of on-site soils should be tested to determine if they meet the swell criteria outlined for the selected floor slab treatment option.

2. *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.

3. *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. Based on the results of laboratory testing, the on-site natural clay soils should be suitable for reuse as compacted site grading fill and as structural fill beneath foundations and floor slabs provided, they do not contain organic material or other deleterious materials. Fat clays may be used with caution below floor slabs, but should not be reused beneath shallow foundations.

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

1. *Moisture Content*: Predominantly clayey fill materials should be compacted at uniform moisture contents between optimum and 3 percentage points above optimum moisture content. Considerable processing will likely be required to achieve a uniform moisture content in the on-site clays, which have relatively low natural moisture contents. Fill materials consisting of predominantly granular soils or imported materials should be compacted at moisture contents within 2 percentage points of optimum. The contractor should be aware the clay soils, including on-site and imported materials may become somewhat unstable and deform under wheel loads if placed near the upper end of the moisture range.

2. *Placement and Degree of Compaction*: Structural fill beneath foundations and exterior flatwork, and adjacent to foundations should be placed in maximum 8-inch lifts. The following compaction criteria should be followed during construction:

AREA	MINIMUM PERCENTAGE OF STANDARD PROCTOR (ASTM D698) MAXIMUM DRY DENSITY
Beneath spread footing foundations and under-slab fill more than 8 feet below slab subgrade	100%
Upper 8 feet of fill beneath building floor slabs	95%
Fill beneath pavements and exterior hardscape, utility trenches, foundation backfill, and landscape areas	95%

Construction Monitoring: A representative of the geotechnical engineer should observe prepared fill subgrades and fill placement on a full-time basis.

SEISMIC DESIGN CRITERIA

The soil profile is expected to generally consist of up to about 10 to 15 feet of overburden clays with occasional sands underlain by relatively medium hard to very hard bedrock. The bedrock is considered to extend to a depth of at least 100 feet below ground surface. Overburden soils consisting of the natural soils and new structural fills will generally classify as International Building Code (IBC) Site Class D. The underlying bedrock generally classifies as IBC Site Class C. Based on the depth to bedrock and our experience with similar profiles, we recommend a design soil profile of IBC Site Class C. Based on the subsurface profile, site seismicity, and the depth of ground water, liquefaction is not a design consideration.

LATERAL EARTH PRESSURES

Loading dock walls and other earth retaining structures should be designed for the lateral earth pressure based on the degree of rigidity of the retaining structure and the type of backfill material used. Retaining structures such as loading dock walls that are laterally supported and can be expected to undergo only a moderate amount of deflection should be designed for earth pressures based on the following equivalent fluid unit weights:

CDOT Class 1 (<20% passing No. 200 Sieve).. 50 pcf
On-site, moisture-conditioned cohesive soil backfill .... 65 pcf

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 unit weights:

CDOT Class 1 (<20% passing No. 200 Sieve).. 40 pcf
On-site, moisture-conditioned cohesive soil backfill .... 55 pcf

The pressures recommended above assume drained conditions behind the walls and a horizontal backfill surface. The buildup of water behind a wall or an upward sloping backfill surface will increase the lateral pressure imposed on a retaining structure.

EXTERIOR FLATWORK

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

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

SURFACE DRAINAGE

Proper surface drainage is very important for acceptable performance of structures during construction and after the construction has been completed. Drainage recommendations provided by local, state and national entities should be followed based on the intended use of the structure. 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 the foundation and slab subgrades should be avoided during construction.

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

3. The ground surface surrounding the exterior of the structure should be sloped to drain away from the structure or foundation in all directions. We recommend a minimum slope of 12 inches in the first 10 feet in unpaved areas. Site drainage beyond the 10-foot zone should be designed to promote runoff and reduce infiltration. A minimum slope of 3 inches in the first 10 feet is recommended in paved or flatwork areas. These slopes may be changed as required for handicap access points in accordance with the Americans with Disabilities Act.

4. To promote runoff, the upper 1 to 2 feet of the backfill adjacent to buildings should be a relatively impervious on-site soil or be covered by flatwork or a pavement structure.

5. Ponding of water should not be allowed in foundation backfill material or in a zone within 20 feet of the building, whichever is greater.

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

7. Landscaping adjacent to the building underlain by moisture-sensitive soils and/or bedrock should be designed to avoid irrigation requirements that would significantly increase soil moisture and potential infiltration of water within at least ten feet of the building. Landscaping located within 10 feet of the building should be designed for irrigation rates that do not significantly exceed evapotranspiration rates. Use of vegetation with low water demand and/or drip irrigation systems are frequently used methods for limiting irrigation quantities.

   Lawn sprinkler heads and landscape vegetation that requires relatively heavy irrigation should be located at least 10 feet from the building. Even in areas away from the building, it is important to provide good drainage to promote runoff and reduce infiltration. Main pressurized zone supply lines, including those supplying drip systems, should be located more than 10 feet from the building in the event that leaks occur. All irrigation systems, including zone supply lines, drip lines, and sprinkler heads should be routinely inspected for leaks, damage, and improper operation.

UNDERDRAINS

Site grading information was not available at the time of this report. If site excavations will interrupt the bedrock surface at building locations, underdrains will be required.

An underdrain system should consist of drain lines extending along the perimeter of the over-excavated zone. The alignment of the drain system should preferably be just outside of the building perimeter. The drains should consist of 4-inch diameter, rigid, perforated PVC pipe placed in trenches excavated at the base of the over-excavated zone.

The drain pipes should be surrounded above the invert level with free-draining granular material extending to the bottom-of-slab level or to the base of a sub slab gravel zone, if provided. The free-draining aggregate should conform to the requirements of CDOT Class B or Class C Filter Material. Alternatively, the pipes can be wrapped with a geotextile fabric to prevent migration of fines from the surrounding soil into the drainage material; in that case a coarser free-draining gravel not necessarily meeting graded filter criteria, such as AASHTO No. 57 or No. 67 Aggregate, may be used. Pipe slots or perforations should be sized in accordance with the type of free-draining material surrounding the pipe.

The base of the over-excavation should be graded to slope towards the drain lines with a minimum slope of 0.5%. The overall underdrain system should be sloped at a minimum slope of 0.5% to a place where water can be removed by pumping or gravity drainage. In the event that sumps are used, they should be provided with alarms in the event the pumping equipment malfunctions.

PAVEMENT DESIGN

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

Subgrade Materials: Based on the results of the field exploration and laboratory test data, the on-site soils at the site generally classify as A-6 and A-7-6 soils with group indices ranging from 13 to 31 in accordance with the American Association of State Highway and Transportation Officials (AASHTO) classification system. Soils classifying as such are generally be considered to provide poor subgrade support. For design of the flexible pavement sections, a subgrade resilient modulus, M_R, of 3,025 psi was used.

Design Traffic: It appears daily traffic at the site will include automobiles that will utilize certain portions of the facility on a routine basis along with occasional trash trucks. It is also anticipated medium to large delivery trucks will utilize certain portions of the facility on a regular basis. Since anticipated traffic loading information was not available at the time of report preparation, an equivalent 18-kip daily equivalent single axle loading application (ESAL) of 36,500 was assumed for automobile parking areas, an ESAL of 73,000 was assumed for automobile drive and fire lanes, and an ESAL of 219,000 was assumed for drive areas that will be subjected to routine truck traffic. If traffic is anticipated to be different that the assumed values above, we should be notified to reevaluate the pavement thickness sections provided below.

Pavement Sections: The pavement sections were developed from the DARWin™ computer software program that solves the AASHTO pavement design equations. The recommended pavement sections are presented below.

Recommended Pavement Section Thicknesses

Area	**Full-depth HMA *(in.)***	**Composite HMA over ABC *(in.)***	**PCCP *(in.)***
Auto Parking	6.0	4.0 over 8	6.0
Auto Drive & Fire Lanes	6.5	4.5 over 8	7.0
Truck Traffic and Loading	8.0	5.0 over 10	7.0

PCCP = Portland Cement Concrete Pavement, HMA = Hot Mix Asphalt, ABC = Aggregate Base Course

Dumpster pads, and any other areas that will be subjected to concentrated truck turning movements should be paved using a minimum of 7.0 inches of Portland cement concrete. Concrete pavement should contain sawed or formed joints to ¼ of the depth of the slab at a maximum distance of 12 to 14 feet on center.

The concrete sections presented above are assumed to be un-reinforced. Providing dowels at construction joints would help reduce the risk of differential movements between panel sections. Providing a grid mat of deformed rebar within the concrete pavement section would assist in mitigating corner breaks and differential panel movements. If a rebar mat is installed, we recommend that the bars be placed in the lower half of the pavement section. On projects that elect to install rebar mats, we have commonly seen No. 4 rebar placed at 24-inch centers in each direction, however we recommend that a structural engineer evaluate the placement and spacing of rebar if needed.

Pavement Materials: Hot mix asphalt and Portland cement concrete pavement should meet the latest applicable requirements, including the CDOT Standard Specifications for Road and Bridge Construction. We recommend the asphalt placed for the project is designed in accordance with the SuperPave gyratory mix design method. The mix should generally meet Grading S or SX requirements with a SuperPave gyratory design revolution (N_{DESIGN}) of 75. A PG 58-28 asphalt binder should be used for the mix. In the event that a PG 64-22 asphalt binder is used in the mix, the asphalt section will provide adequate structural support but will be more susceptible to low temperature related transverse cracking.

Subgrade Preparation: As mentioned in the "Site Grading and Earthwork" section of the report, existing fill and swelling soils occur at the site. Non-engineered fills and expansive subgrade conditions are a problem where present beneath pavements and could result in potentially excessive settlement or heave when subjected to increases in moisture. In order to reduce the potential for pavement distress associated with excessive post-construction movement due to settlement of the non-engineered fill or heave due to swelling subgrade materials, we recommend that the pavement section be underlain by a minimum of 2 feet of moisture-treated compacted fill.

Just prior to placing compacted fill, the entire fill subgrade area should be scarified to a depth of at least 12 inches, moisture conditioned to within a range of optimum to 3 percentage points above optimum, and re-compacted to 95% of the standard Proctor (ASTM D698) maximum dry density. The contractor should be aware that the clay soils may become somewhat unstable and deform under wheel loads if placed near the upper end of the moisture range.

The pavement subgrade should be proof rolled with heavy rubber-tired equipment 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.

Maintenance: Routine maintenance of paved areas is critical to achieve the design life of the pavement. Crack sealing should be performed annually as new cracks appear. Chip seals, fog seals, or slurry seals applied at approximate intervals of 3 to 5 years are usually necessary for asphalt pavement. As conditions warrant, it may be necessary to perform patching and structural overlays at approximate 10-year intervals.

For concrete pavement, joint sealing should be performed as needed, which generally occurs every 8 to 10 years. A quarter-inch diamond grind to remove worn concrete surfacing is generally performed after approximately 20 years for pavement in generally good condition.

Drainage: The collection and diversion of surface drainage away from paved areas is extremely important to the satisfactory performance of pavement. Surface 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

Kumar & Associates, Inc. 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 Kumar & Associates, Inc. be retained to provide observation and testing services to document the intent of this report and the requirements of the plans and specifications are being followed during construction, and to identify possible variations in subsurface conditions from those encountered in this study so that we can re-evaluate our recommendations, if needed.

LIMITATIONS

This study has been conducted in accordance with generally accepted geotechnical engineering practices in this area for exclusive use by the client for design purposes. The conclusions and recommendations submitted in this report are based upon data obtained from the explorations performed by Kumar & Associates at the locations indicated on Fig. 1, and the proposed construction. This report may not reflect subsurface variations that occur between the explorations, and the nature and extent of variations across the site may not become evident until site grading and excavations are performed. If during construction, fill, soil, rock or water conditions appear to be different from those described herein, Kumar & Associates, Inc. should be advised at once so a re-evaluation of the recommendations presented in this report can be made. Kumar & Associates, Inc. is not responsible for liability associated with interpretation of subsurface data by others.

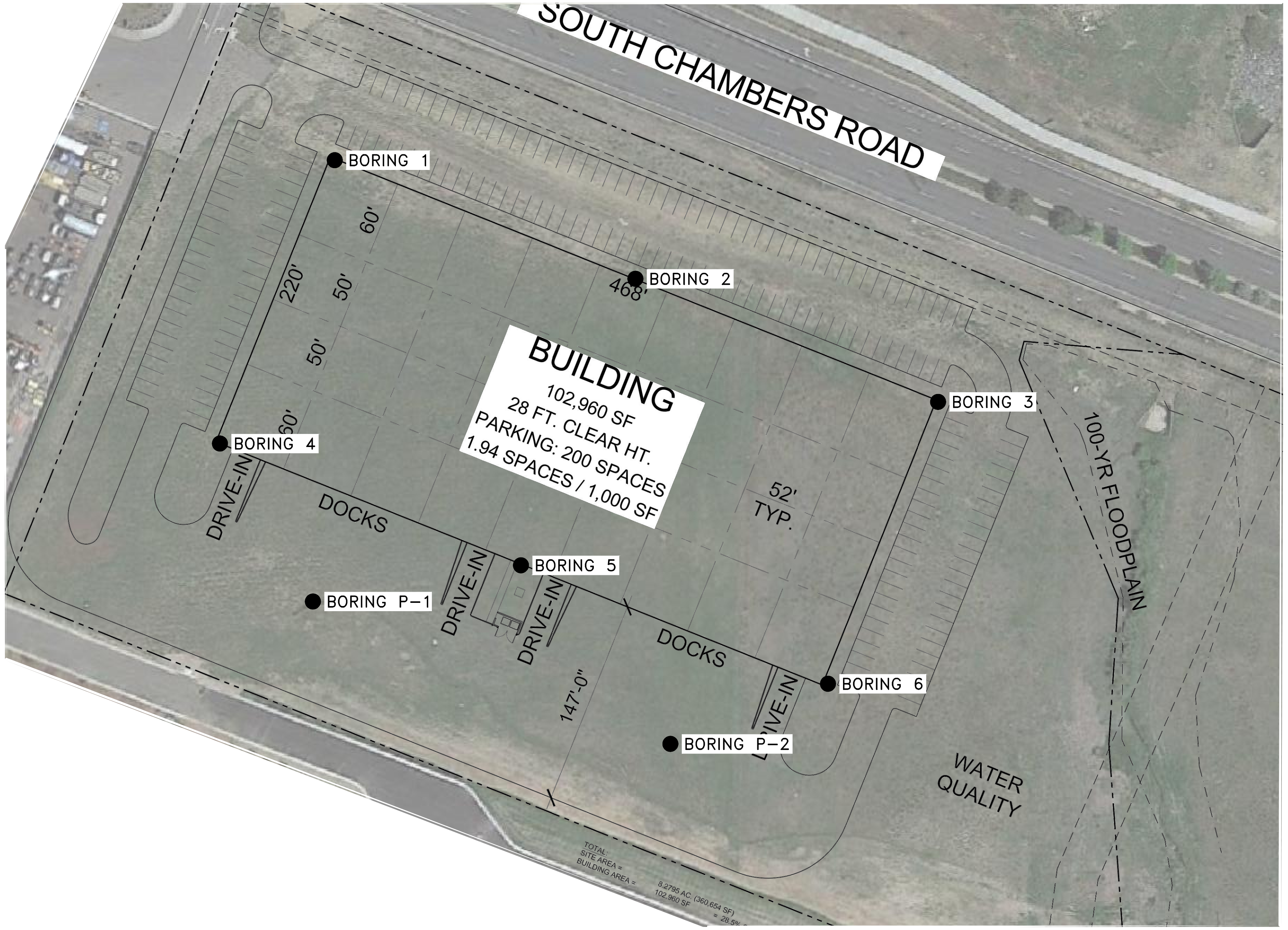
The scope of services for this project does not include any environmental assessment of the site or identification of contaminated or hazardous materials or conditions.

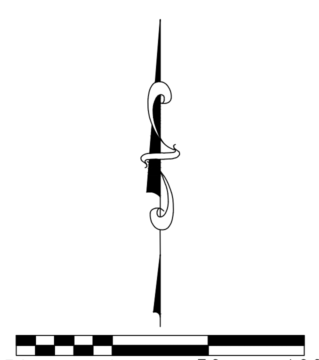
Swelling soils and bedrock is present at this site. Such materials are stable at their natural moisture content but will undergo high volume changes with changes in moisture content. The extent and amount of perched water beneath the building site as a result of area precipitation and irrigation, and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

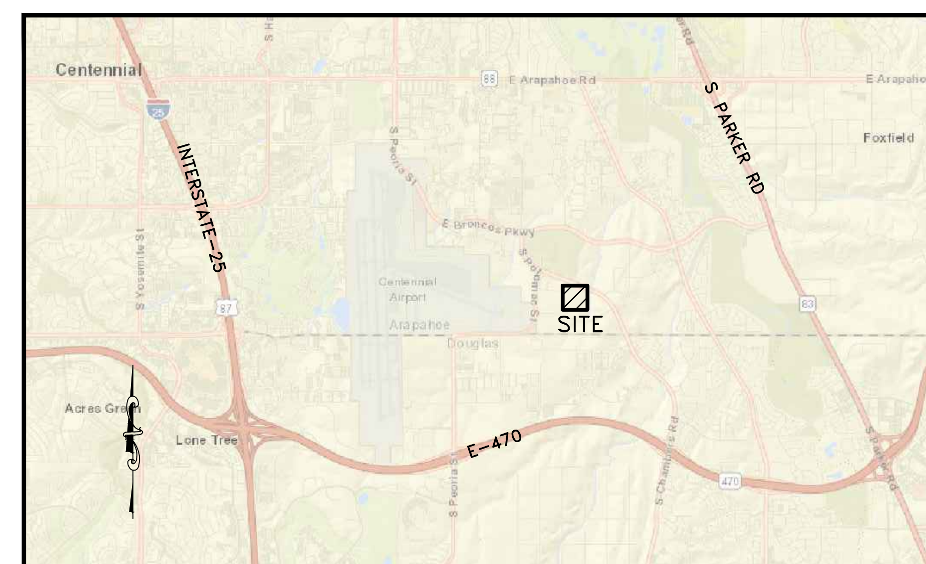
The recommendations presented in this report are based on current theories and experience of our engineers on the behavior of swelling soil and bedrock in this area. The owner should be aware there is a risk in constructing buildings and pavements in an area of highly expansive soil and bedrock. Following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

Once site grading plans including finished floor elevations have been finalized, they should be made available to us for our review to determine if the recommendations presented herein remain valid.

RRK/as
Rev. by: JDC
cc: book, file







VICINITY MAP
NOT TO SCALE

22-1-186	Kumar & Associates	DOVE VALLEY INDUSTRIAL, 8001 SOUTH CHAMBERS ROAD, ENGLEWOOD, COLORADO	LOCATION OF EXPLORATORY BORINGS	Fig. 1

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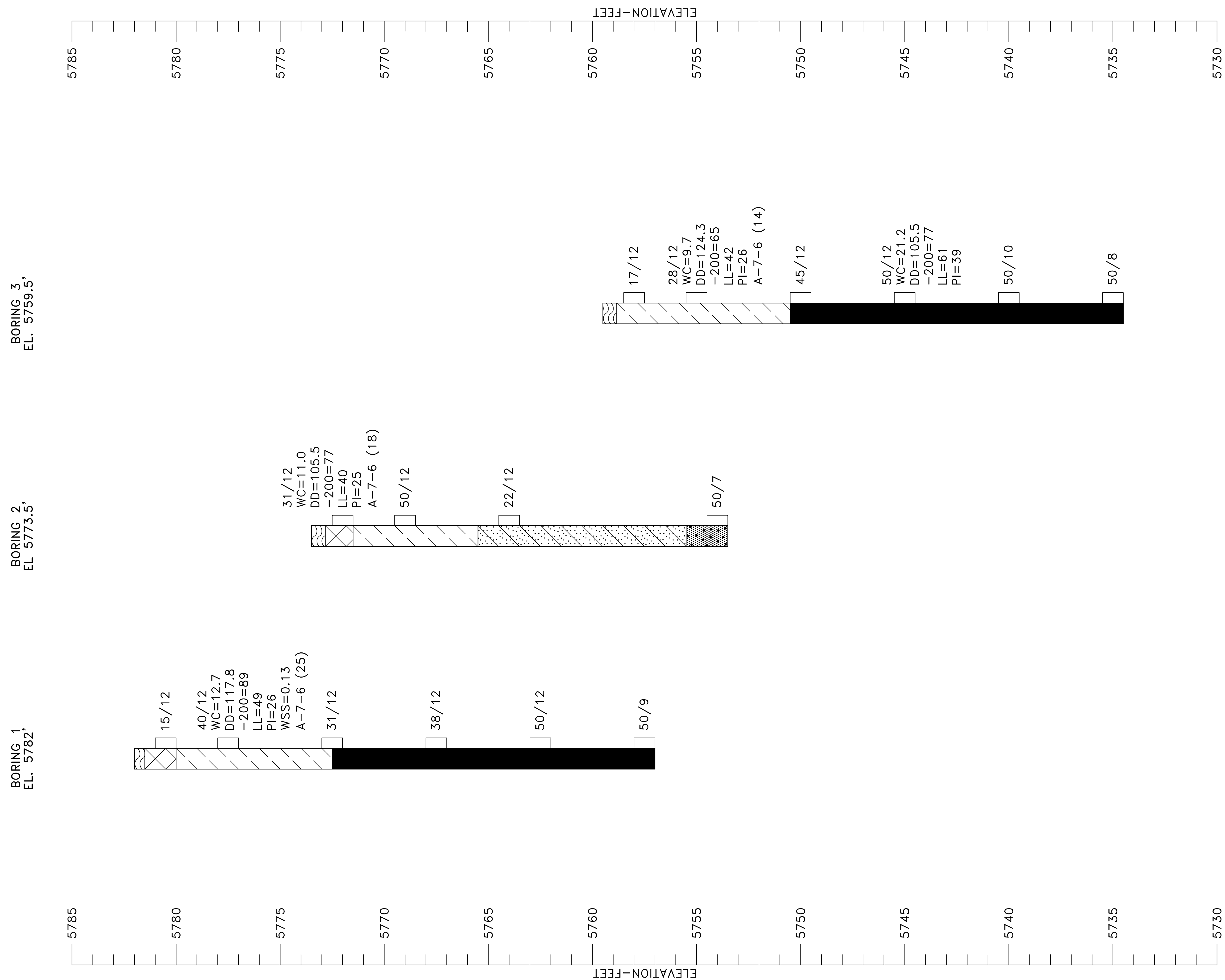
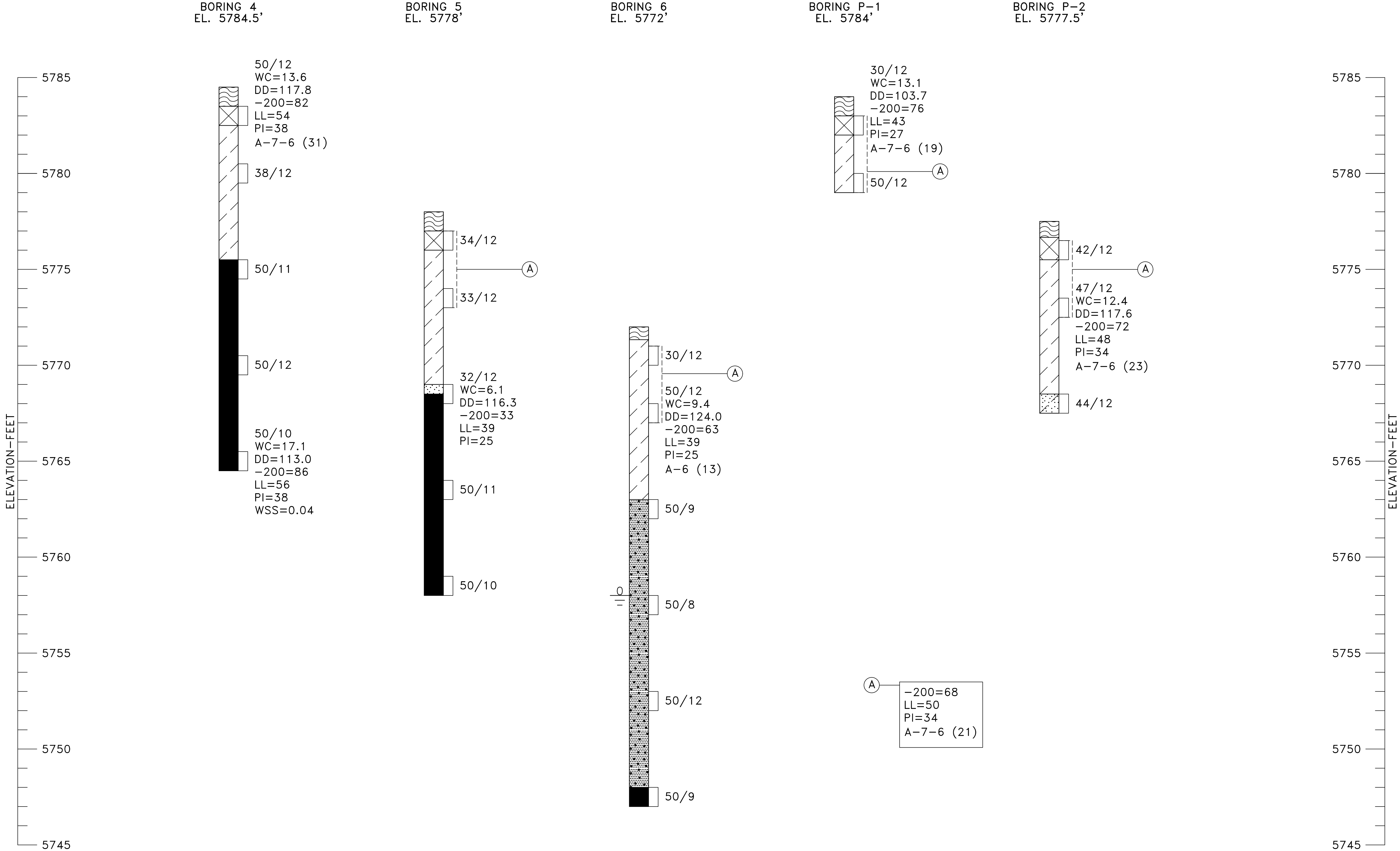


22-1-186	Kumar & Associates	DOVE VALLEY INDUSTRIAL, 8001 SOUTH CHAMBERS ROAD, ENGLEWOOD, COLORADO	LOGS OF EXPLORATORY BORINGS	Fig. 2



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22-1-186	Kumar & Associates	DOVE VALLEY INDUSTRIAL, 8001 SOUTH CHAMBERS ROAD, ENGLEWOOD, COLORADO	LOGS OF EXPLORATORY BORINGS	Fig. 3

LEGEND

TOPSOIL.

FILL: LEAN CLAY (CL) WITH VARIABLE FINE– TO COARSE–GRAINED SAND CONTENT, SLIGHTLY MOIST TO MOIST, TAN TO BROWN.

LEAN CLAY (CL) WITH VARIABLE SAND CONTENT AND ISOLATED GRAVELS, VERY STIFF TO HARD, SLIGHTLY MOIST TO MOIST, CALCAREOUS IN PLACES, TAN TO BROWN TO RED–BROWN.

SILTY SAND (SM) WITH OCCASIONALLY POORLY–GRADED SAND WITH SILT (SP–SM) LENSES, MEDIUM DENSE, FINE– TO COARSE–GRAINED, SLIGHTLY MOIST TO MOIST, TAN TO BROWN TO YELLOW–BROWN.

CLAYEY SAND (SC), FINE– TO COARSE–GRAINED, DENSE, SLIGHTLY MOIST, TAN TO BROWN.

CLAYSTONE BEDROCK, OCCASIONALLY SILTY, MEDIUM HARD TO HARD, SLIGHTLY MOIST TO MOIST, GRAY TO DARK GRAY TO GRAY–BROWN WITH OCCASIONAL IRON OXIDATION STAINING.

SANDSTONE BEDROCK, HARD TO VERY HARD, FINE– TO COARSE–GRAINED, OCCASIONALLY SILTY, SLIGHTLY MOIST, TAN TO GOLD–BROWN.

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

DISTURBED BULK SAMPLE.

15/12 DRIVE SAMPLE BLOW COUNT. INDICATES THAT 15 BLOWS OF A 140–POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE THE SAMPLER 12 INCHES.

DEPTH TO WATER LEVEL ENCOUNTERED AT THE TIME OF DRILLING.

NOTES

1. THE EXPLORATORY BORINGS WERE DRILLED ON MARCH 11 AND 23, 2022 WITH 4–INCH–DIAMETER CONTINUOUS–FLIGHT POWER AUGERS.

2. THE LOCATIONS OF THE EXPLORATORY BORINGS WERE MEASURED APPROXIMATELY BY PACING FROM FEATURES SHOWN ON THE SITE PLAN PROVIDED.

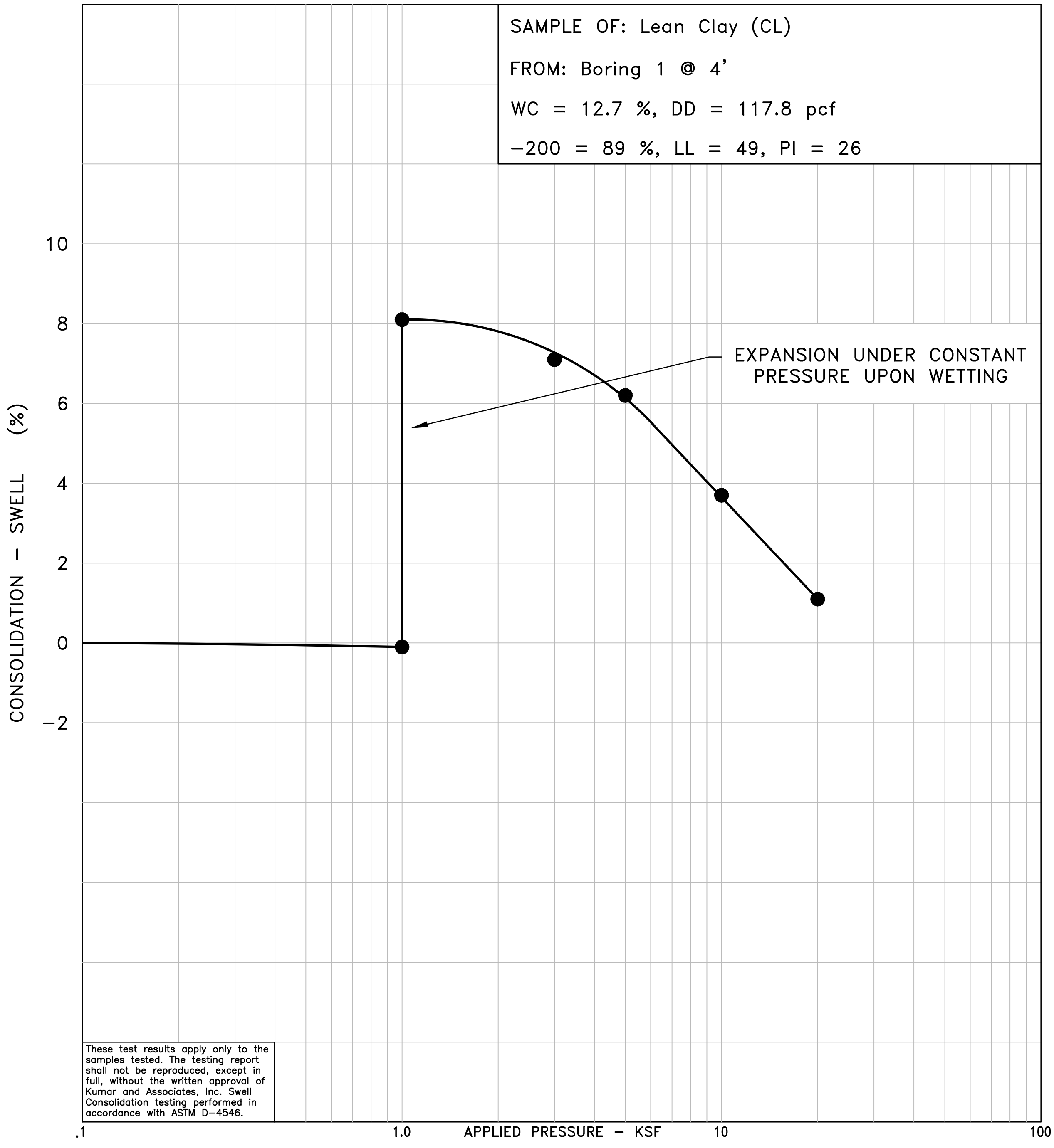
3. THE ELEVATIONS OF THE EXPLORATORY BORINGS WERE NOT MEASURED AND THE LOGS OF THE EXPLORATORY BORINGS ARE PLOTTED TO DEPTH.

4. THE EXPLORATORY BORING LOCATIONS SHOULD BE CONSIDERED ACCURATE ONLY TO THE DEGREE IMPLIED BY THE METHOD USED.

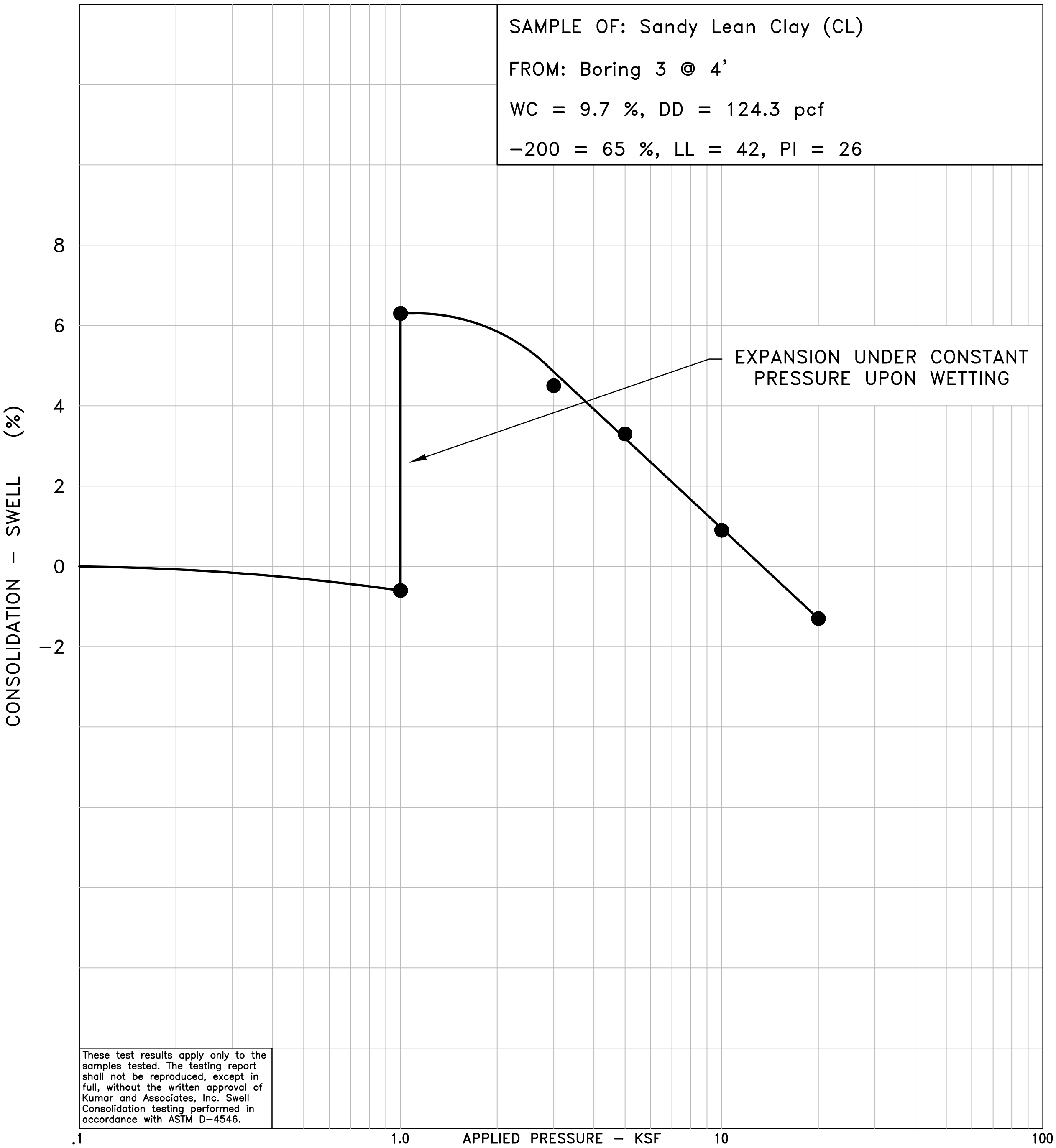
5. THE LINES BETWEEN MATERIALS SHOWN ON THE EXPLORATORY BORING LOGS REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN MATERIAL TYPES AND THE TRANSITIONS MAY BE GRADUAL.

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);
   +4 = PERCENTAGE RETAINED ON NO. 4 SIEVE (ASTM D6913);
   –200= PERCENTAGE PASSING NO. 200 SIEVE (ASTM D1140);
   LL = LIQUID LIMIT (ASTM D4318);
   PI = PLASTICITY INDEX (ASTM D4318);
   NV = NO LIQUID LIMIT VALUE (ASTM D4318);
   NP = NON–PLASTIC (ASTM D 4318);
   WSS = WATER SOLUBLE SULFATES (%) (CP–L 2103);
   A–2–6 (0) = AASHTO CLASSIFICATION (GROUP INDEX) (AASHTO M145).



22−1−186	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULTS	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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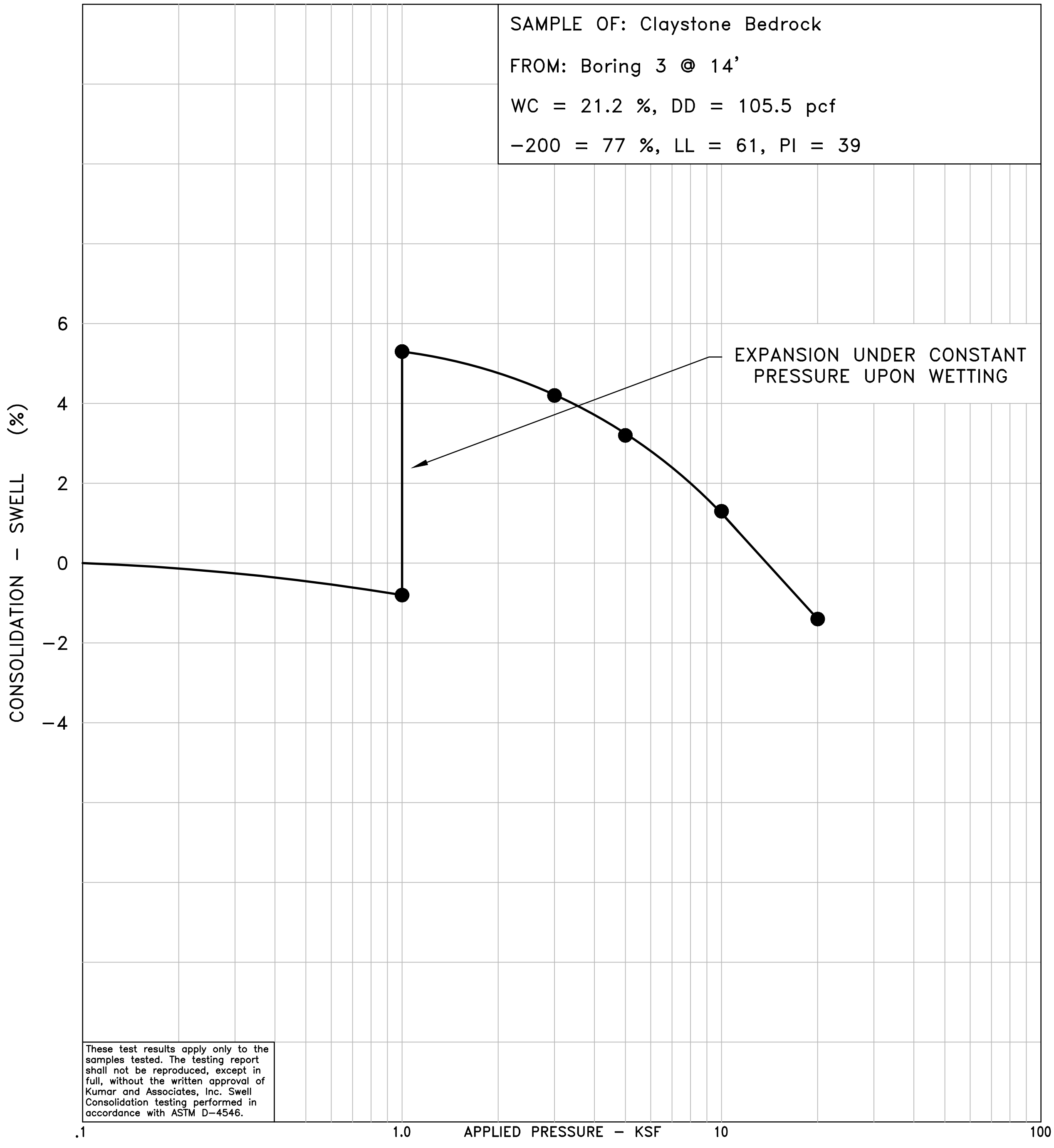
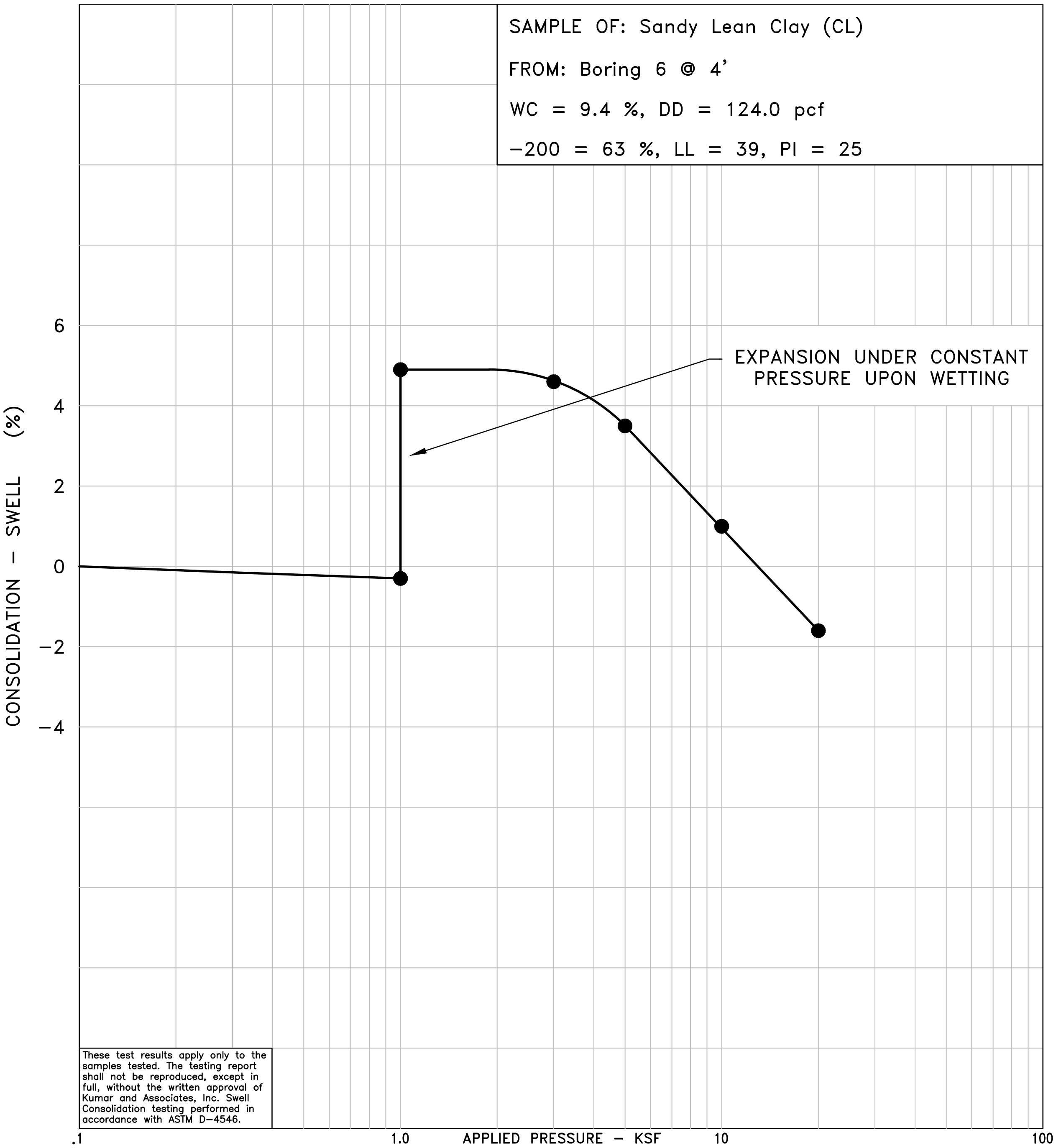
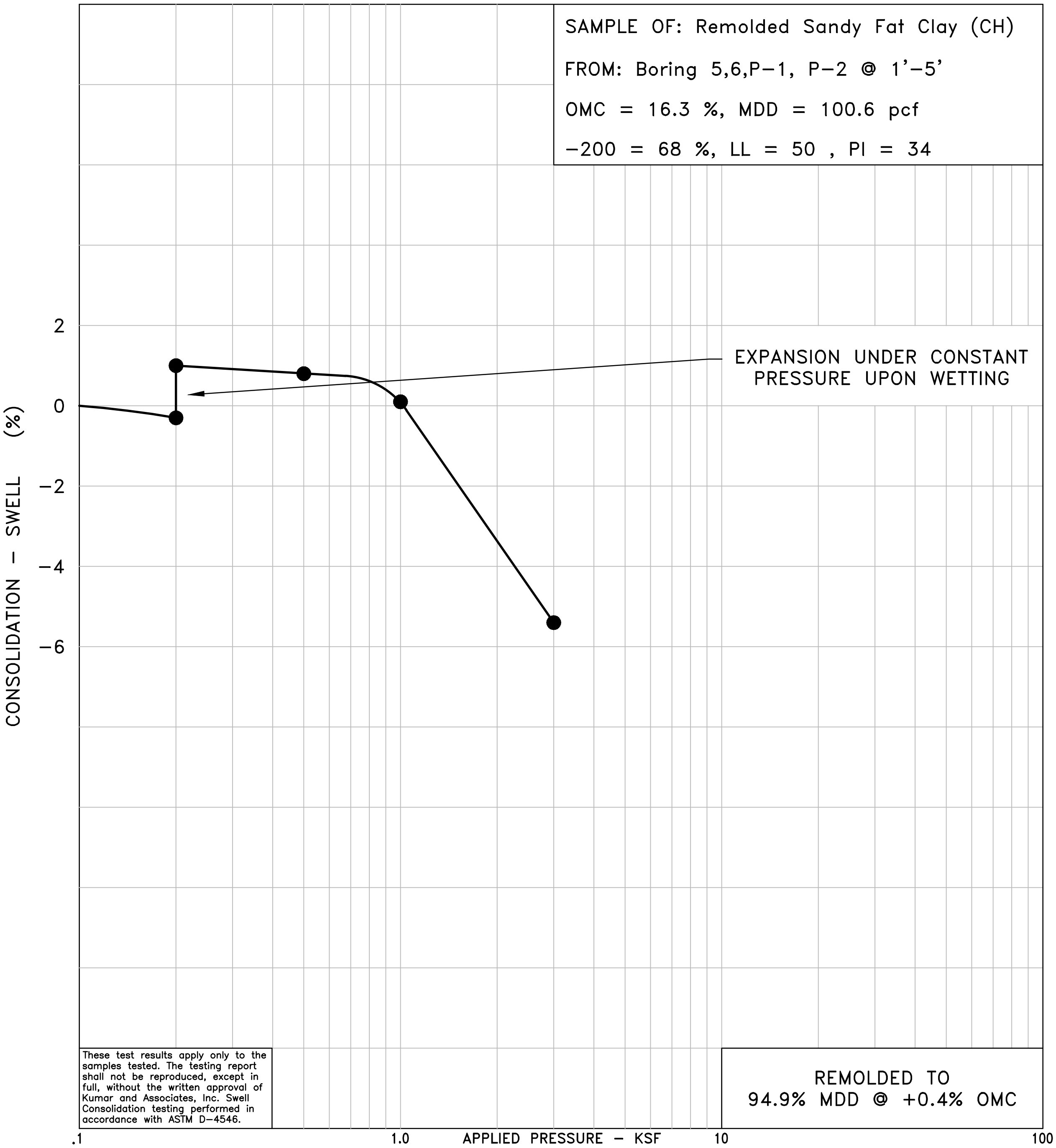
22−1−186	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULTS	Fig. 6



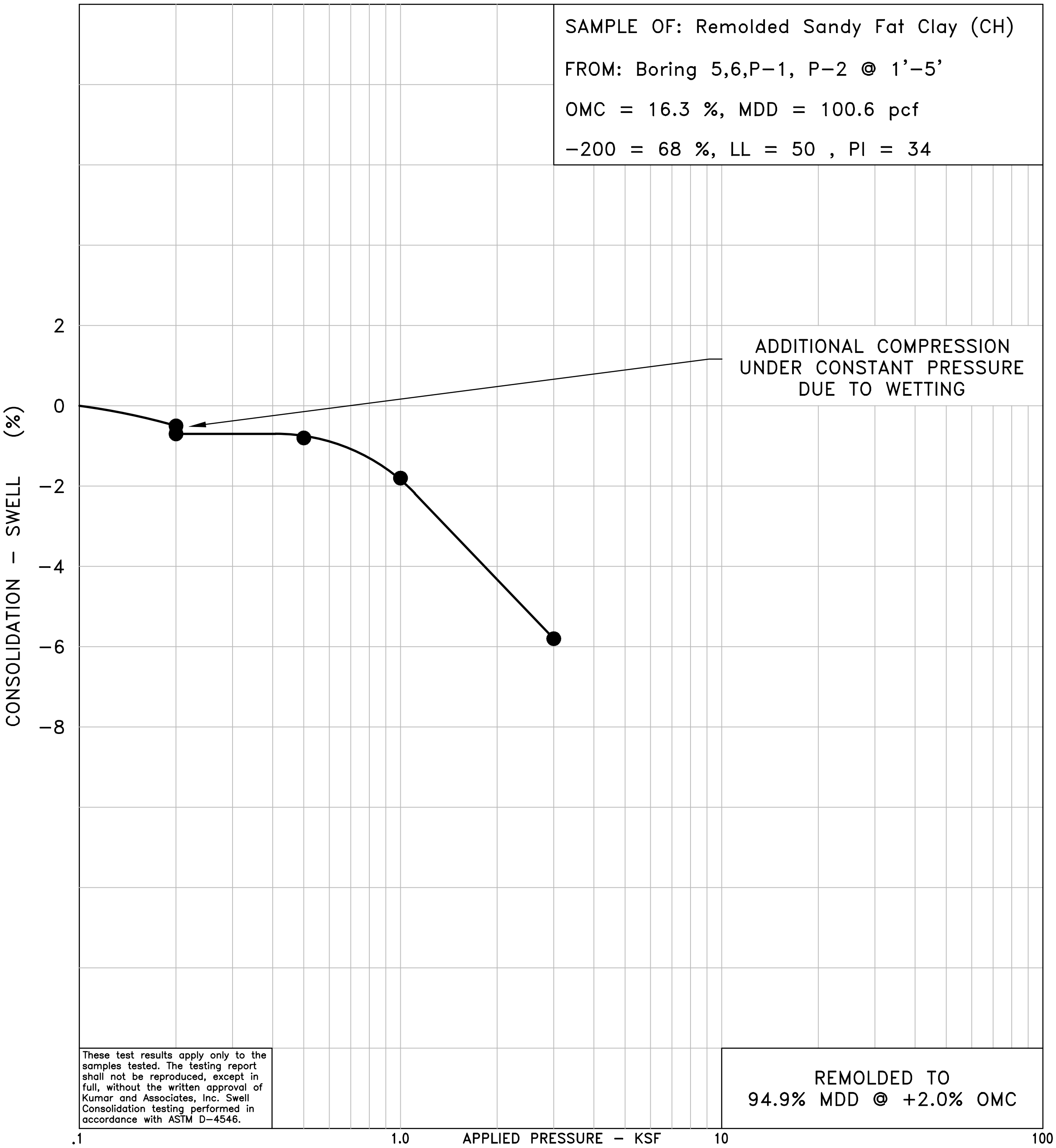
22-1-186	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULTS	Fig. 7



22-1-186	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULTS	Fig. 8



22-1-186	Kumar & Associates	REMOLDED SWELL-CONSOLIDATION TEST RESULTS	Fig. 9



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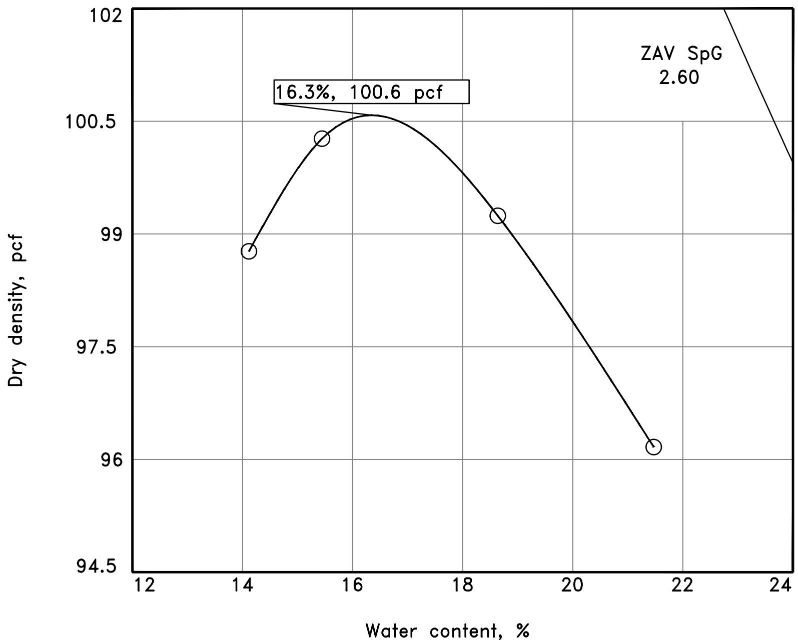
22-1-186	Kumar & Associates	REMOLDED SWELL-CONSOLIDATION TEST RESULTS	Fig. 10

COMPACTION TEST REPORT

Curve No. 1973



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	0
%<No.200	68
Atterberg (D 4318): LL	50
PI	34
NM (D 2216)	
Sp.G. (D 854)	2.6
USCS (D 2487)	CH
AASHTO (M 145)	
Date: Sampled	3-28-2022
Received	3-28-2022
Tested	4-6-2022
Tested By	KP

COMPACTION TESTING DATA
ASTM D 698-12 Method A Standard

	1	2	3	4	5	6
WM + WS	6277.0	6323.0	6353.0	6339.0		
WM	4573.0	4573.0	4573.0	4573.0		
WW + T #1	810.0	784.1	734.5	622.7		
WD + T #1	738.7	708.7	649.4	539.2		
TARE #1	233.6	220.5	192.8	150.3		
WW + T #2						
WD + T #2						
TARE #2						
MOIST.	14.1	15.4	18.6	21.5		
DRY DENS.	98.8	100.3	99.2	96.2		

SIEVE TEST RESULTS
ASTM D-422 ASTM D-1140

Opening Size	% Passing	Specs.

TEST RESULTS

Maximum dry density = 100.6 pcf

Optimum moisture = 16.3 %

Material Description

Sandy Fat Clay (CH)

Project No. 22-1-186 Client:

Project: Dove Valley Industrial

○ Location: P1, P2, B5, B6 Depth: 1'-5' Sample Number: 1973

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.

Checked by: DS

Title: Lab Manager

22-1-186	Kumar & Associates	MOISTURE-DENSITY RELATIONSHIPS	Fig. 11

TABLE I
SUMMARY OF LABORATORY TEST RESULTS

PROJECT NO.: 22-1-186
PROJECT NAME: Dove Valley Industrial
DATE SAMPLED: 3-28-2022
DATE RECEIVED: 3-28-2022

SAMPLE LOCATION		DATE TESTED	NATURAL MOISTURE CONTENT (%)	NATURAL DRY DENSITY (pcf)	PERCENT PASSING NO. 200 SIEVE	ATTERBERG LIMITS		WATER SOLUBLE SULFATES (%)	AASHTO CLASSIFICATION (group index)	SOIL OR BEDROCK TYPE
BORING	DEPTH (feet)					LIQUID LIMIT (%)	PLASTICITY INDEX (%)			
1	4	3-28-22	12.7	117.8	89	49	26	0.13	A-7-6 (25)	Lean Clay (CL)
2	1	3-28-22	11.0	105.5	77	40	25		A-6 (18)	Lean Clay with Sand (CL)
3	4	3-28-22	9.7	124.3	65	42	26		A-7-6 (14)	Sandy Lean Clay (CL)
3	14	3-28-22	21.2	105.5	77	61	39			Claystone Bedrock
4	1	3-28-22	13.6	117.8	82	54	38		A-7-6 (31)	Fat Clay with Sand (CH)
4	19	3-28-22	17.1	113.0	86	56	38	0.04		Claystone Bedrock
5	9	3-28-22	6.1	116.3	33	39	25			Clayey Sand (SC)
6	4	3-28-22	9.4	124.0	63	39	25		A-6 (13)	Sandy Lean Clay (CL)
P1	1	3-28-22	13.1	103.7	76	43	27		A-7-6 (19)	Lean Clay with Sand (CL)
P2	4	3-28-22	12.4	117.6	72	48	34		A-7-6 (23)	Lean Clay with Sand (CL)
P1, P2, B5, B6	1-5	3-28-22	16.3*	100.6*	68	50	34		A-7-6 (21)	Sandy Fat Clay (CH)

*Maximum Dry Density and Optimum Moisture Content ASTM D698