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GEOTECHNICAL ENGINEERING STUDY
AND PAVEMENT THICKNESS DESIGN
RIDGEGATE SENIOR HOUSING DEVELOPMENT
RIDGEGATE PARKWAY ¾-MILE WEST OF MERIDIAN VILLAGE PARKWAY
LONE TREE, COLORADO

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SUMMARY

1. A field exploration program consisting of drilling six (6) exploratory borings was conducted to obtain information on subsurface conditions at the approximate locations shown on Fig. 1. Four borings (Borings 1 through 4) were drilled in the footprint area of the proposed buildings, and two borings (P-1 and P-2) for the proposed pavement areas. The subsurface conditions encountered in the borings generally consisted of a thin veneer of topsoil. Beneath the topsoil, natural soils were encountered in two borings, and the remaining three borings encountered about 1 to 6.5 feet of man-placed fill extending to natural soils. Bedrock was encountered in building borings beneath natural soils, at depths ranging from about 9 to 17.5 feet and the borings were terminated in the bedrock at depths of about 25 feet or 30 feet. The pavement borings were terminated in natural soil at depths of about 10 feet.

   Groundwater was not encountered in the borings during drilling or when measured 5 days later. The borings were backfilled subsequent to these measurements.

2. Based on the information obtained from the exploratory borings and the results of the laboratory testing, the most appropriate foundation system is to support the proposed building on a deep foundation system consisting of straight shaft piers drilled into unweathered bedrock. Piers should be designed for an allowable end-bearing pressure of 15,000 psf, bedrock side shear (skin friction) of 10% of end bearing, and a minimum dead load pressure of 15,000 psf.

   As an alternative to a deep foundation, a shallow foundation system is considered feasible for the support of the proposed building when placed on properly compacted structural fill extending to undisturbed natural soils. Footings placed on a prepared subgrade may be designed for a net allowable soil-bearing pressure of 2,500 psf with a minimum dead load of 800 psf. The allowable bearing pressure may be increased by one-third for transient loads.

3. Slab-on-grade floors are considered feasible for the proposed construction. Slab-on-grade floors and movement-sensitive flatwork should be placed on at least 5 feet of structural fill extending to undisturbed natural soil or bedrock.

4. Pavement section alternatives based on the anticipated traffic volume, on-site material properties, and local industry standards of practice are presented below:

LOCATION	Full Depth Asphalt Pavement (inches)	Asphalt Over Aggregate Base Course (inches)	PCCP (inches)
Automobile Parking	6.0	4.0 over 7.0	6.0
Drive Lanes	7.0	5.0 over 7.0	7.0

PURPOSE AND SCOPE OF STUDY

This report presents the results of a geotechnical engineering study and pavement thickness design for the proposed Ridgegate Affordable Housing, Ridgegate Parkway ¾-mile West of Meridian Village Parkway in Lone Tree, Colorado. The project site is shown on Fig. 1. The study was conducted to characterize the general site subsurface conditions and provide geotechnical engineering recommendations to be used for design purposes. The study was conducted in general accordance with the scope of work in our Proposal No. P-23-226 to Koelbel and Company, dated February 16, 2023.

A field exploration program consisting of exploratory borings was conducted to obtain information on general subsurface conditions. Samples of the on-site existing fill, natural soils, and bedrock materials obtained from the exploratory borings were tested in the laboratory to determine their swell-consolidation, classification, and general engineering characteristics. The results of the field exploration and laboratory testing programs were used to evaluate site geotechnical considerations and develop geotechnical engineering recommendations.

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

PROPOSED DEVELOPMENT

Based on the information provided, we understand Phase I construction will consist of an L-shaped building with an approximate structure footprint of 29,250 sf. The building will be four stories over a one-story parking podium. Phase II construction will occur to the east, on the east side of an asphalt paved surface parking lot. The building for Phase II construction will have a structure footprint of approximately 15,500 sf. with 14 tuck-under vehicle stalls.

We assume the building will be constructed using a combination of masonry structure and steel frame construction. Foundation loads are expected to be relatively light to moderately heavy, consistent with this type of construction.

Site grading with range from a cut of 9 feet for the southwest building corner to fills of about 5 feet from the northeast corner. Site grading for future Phase 2 of the development with range from a one-foot cut at the southern end of the building to fills of up to 7 feet for the north end of the building.

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

SITE CONDITIONS

The site is an approximately 7.5 acres parcel consisting of a vacant lot. The overall site is bounded on the west and east by vacant lots, on the north by westbound of Ridgegate Parkway, and on the south by eastbound of Ridgegate Parkway. Badger Gulch Park and a natural drainage are located to the east with additional vacant land to the west. Based on available topographic information, the site slopes down from west to east, with about 16 feet of relief across the proposed building footprint.

SUBSURFACE CONDITIONS

Field Exploration: The subsurface conditions were explored by drilling six (6) exploratory borings at the approximate locations shown on Fig. 1. The borings were advanced through the overburden soils into underlying bedrock, where present, using 4-inch-diameter, continuous-flight augers and were logged by a representative of K+A. Samples of the existing fill, natural soils, and bedrock materials were obtained using a 2-inch-I.D. California-liner sampler driven into the subsurface materials with blows from a 140-pound hammer falling 30 inches. The sampling procedure is similar to the standard penetration test described by ASTM International (ASTM) test procedure D1586. The California-liner sampler is used locally to obtain relatively undisturbed soil and bedrock samples. Penetration resistance values (blow counts) indicate the relative density or consistency of the soils and bedrock.

Depths at which samples were taken and the associated blow counts are shown on the boring logs on Fig. 2. A legend and notes associated with the graphic logs describing the soils and bedrock materials encountered are presented on Fig. 3.

Subsurface Conditions: The subsurface conditions encountered in the borings generally consisted of a thin veneer of topsoil. Beneath the topsoil, natural soils were encountered in Borings 1 and 4, and the remaining three borings encountered about 1 to 6.5 feet of man-placed fill extending to natural soils at depths ranging from about 1 foot to 6.5 feet. Bedrock was encountered in building borings beneath natural soils at depths ranging from about 9 to 17.5 feet, and the borings were terminated in the bedrock at depths of about 25 feet or 30 feet. The pavement borings were terminated in natural soil at depths of about 10 feet.

The existing fill materials consisted of lean to fat clay with a variable fine- to coarse-grained sand fraction and were moist to very moist and brown to dark brown to gray. The vertical and horizontal limits of the fill were not determined as part of this study.

The natural soil varied between cohesive and granular soils. The cohesive soils generally consisted of lean clay with a variable fine- to medium-grained sand fraction, and were moist and brown to dark brown and calcareous in places. The granular soils generally consisted of silty, clayey sand which was fine- to coarse-grained with variable gravel content and cobbles, and were slightly moist to moist and white to orange to brown. Boring 2 encountered poorly-graded sand with which was fine- to coarse-grained variable gravel content and cobbles, and was moist and light brown to brown. Based on blow counts, the natural clay soil was stiff to hard, and the granular soils generally were medium dense to dense.

The bedrock consisted of claystone which possessed high plasticity and was very moist and was orange to brown to black. Based on blow counts, the claystone bedrock was firm to hard and isolated weathered zone.

Groundwater Conditions: Groundwater was not encountered in the borings during drilling. The building borings were left open to measure stabilized groundwater, and the pavement borings were backfilled after the completion of drilling. Stabilized groundwater was not encountered in the building borings when measured 5 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, and swell-consolidation. 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 6, and summarized in Table I.

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

Swell-Consolidation: Swell-consolidation tests were conducted on samples of the existing clay fill, natural clay soil, and claystone bedrock to determine the swell and 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 inundated with water, and the change in sample height when deformation ceased was measured with a dial gauge. The samples were then loaded incrementally to maximum surcharge pressures of 3,000 to 10,000 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 6 as plots of the curve of the final strain at each increment of pressure against the log of the pressure. Based on the results of the swell-consolidation test, the clay fill, natural clay soil, and claystone bedrock samples generally exhibited low swell potentials, and one sample of claystone bedrock exhibited nil movement upon loading when wetted primarily due to the high natural moisture.

GEOTECHNICAL ENGINEERING CONSIDERATIONS

Presence of Expansive Soil and Bedrock: Fat clay was encountered in the on-site existing fill. Claystone bedrock was also encountered within 9 feet of the surface. Fat clay and claystone is problematic in that the elevated moisture contents of the clay/claystone will make it difficult to stabilize. Although the samples of fat clay fill tested exhibited low swell potential, it will become more expansive if dried during construction and placed below structures. Consequently, clay/claystone should not be re-used beneath the shallow foundations and slabs-on-grade. The contractor should be aware that clay/claystone may become unstable and difficult to place at the upper end of the recommended moisture content range and should not be used as fill below building structures and should be replaced by structural fill.

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

On-site Existing Fill: As indicated previously, up to 6.5 feet of man-placed fill was encountered in our borings. Although not indicated in our borings, deeper fills may be present across the site and should be anticipated. Without documentation regarding placement and compaction testing, the fill should be considered non-engineered and unsuitable in its current condition for support of foundation elements and slabs-on-grade due to the presence of fat clay and the potential for excessive heave and/or settlement.

Expansive Soil Considerations: As previously mentioned, expansive sub-soils, consisting of on-site clay fill, natural clay soils, and claystone bedrock, were encountered at the site. Accordingly, subgrade preparation below shallow foundations, floor slabs, movement-sensitive flatwork, and pavements should be performed in accordance with the recommendations presented herein.

Shallow foundations, soil-supported slabs, and pavements placed on or near expansive soils and bedrock will be prone to heave from post-construction increases in the moisture content of these materials, potentially resulting in a movement more than normally accepted tolerances and associated structural distress. The safest foundation system ordinarily would be to support the buildings on 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 the uplift forces from expansive soils and bedrock.

Floor slabs present a problem where expansive materials are present near floor slab elevation because a sufficient dead load cannot be imposed on them to resist the uplift generated when the materials are wetted and expanded. 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. Given the size of the building, structural floor systems 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 overexcavation 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 sections of this report. The use of shallow foundations and slab-on-grade floors will require significant overexcavation beneath the foundation and slab subgrade elevation and backfilling with a zone of moisture-treated and compacted fill.

The acceptable performance of shallow foundations and a slab-on-grade floor system will rely on minimizing water infiltration into the underlying expansive soils by providing good surface drainage and by using prudent landscaping and irrigation practices. In choosing shallow foundations and a slab-on-grade floor system, the owner should understand and accept the risk of potential distress resulting from some foundation and slab movement due to ground heave, even though mitigation measures are used to reduce that risk.

<u>Potential Heave-Related Movement</u>: The risk of ground heave beneath a building can be reduced to a certain degree by providing a zone of non- to low-swelling, relatively impervious, moisture-conditioned fill directly beneath foundations and floor slabs. Heave estimate calculations can be useful in evaluating the relative effectiveness of varying the thickness and material composition of this prepared fill zone. However, such calculations cannot address the uncertainty in the potential depth, and degree of wetting that may occur beneath the buildings or the likelihood of variable swell potential across the site.

We have performed calculations for a range of scenarios of the depth of wetting and overexcavation and backfill combinations to demonstrate the potential for ground heave if the expansive materials beneath the buildings should be thoroughly wetted to a significant depth, including below the base of a prepared fill zone. The following table presents estimates of potential heave based on the results of swell-consolidation tests assuming different thicknesses of moisture-treated fills with low swell potential using test and analysis methods generally accepted along the Colorado Front Range. Both depth of wetting and depth of prepared fill were considered as variables in the analysis.

Sub-Slab Fill Alternative	Ground Heave in Inches		
	10 feet of wetting	15 feet of wetting	20 feet of wetting
No moisture treatment	3.2	4.1	5.0
5 feet of Non-expansive Structural Fill	1.1	2.1	3.0
8 feet of On-site Treated Soil – 1.0% Swell at 200 psf	1.2	2.1	3.0

The calculations performed to generate the above table assumed 20 feet of expansive soils and bedrock. Since up to 15 feet of granular soils exists in portions of the site, the above calculations may be conservative in nature. Additionally, the calculations were performed for floor slabs which generally exert low to moderate surcharge pressures on the underlying soils. Accordingly, the potential post-construction heave-related movement for shallow foundations should generally be less than what is shown in the above table.

The heave estimate calculations demonstrate removing and replacing the expansive on-site fill and natural clay soils back in place beneath the proposed building footprint with moisture-treated, low-swelling on-site or imported non-expansive fill materials will result in a reduction in potential ground heave. The on-site material should not contain claystone or fat clay. The criteria for non-expansive to low-swelling fill materials are provided in the "Site Grading and Earthwork" section of this report.

Our experience indicates that soil-supported slabs on a large majority of sites with generally similar subsurface conditions do not experience extreme moisture increases to significant depth provided good surface drainage is designed, constructed, and maintained, and good irrigation practices are followed. The risk of post-construction moisture increase could be further reduced by eliminating the landscape requiring a lot of irrigation within about 20 feet of the building. Wetting can also occur as a result of unforeseeable influences such as plumbing leaks or breaks or, in some cases, off-site influences depending on geologic conditions.

Considering the above discussion, we believe shallow foundations and a slab-on-grade floor system may be considered for the project, provided the potential for some movement due to ground heave and associated possible distress is recognized by the owner. The intent of our recommendations for foundations and slab-on-grade floors is to provide for conditions where there is a good chance that ground heave beneath the building will not exceed amounts acceptable to the owner. The recommendations should result in heave movements that do not exceed 1.5 inches and are unlikely to exceed 2 inches unless extreme wetting is allowed. Barring unforeseen events, we do not believe significant wetting is likely to occur if the surface drainage and irrigation recommendations presented in this report are followed.

<u>Foundations</u>: The most appropriate foundation system would be to support the structure on drilled piers bearing in the bedrock. Drilled piers extending into bedrock are the most common deep foundation system that provides a cost advantage over other deep foundation systems.

A shallow foundation system may also be considered feasible. However, a shallow foundation would be more susceptible to post-construction movements resulting from moisture changes in the soils and would require a zone of structural fill below footings. Recommendations for a shallow foundation system can be provided upon request.

FOUNDATION RECOMMENDATIONS

Drilled Pier Foundations: Based on the data from the field exploration and laboratory testing properties, straight-shaft piers drilled into the bedrock may be used to support the proposed structure.

1. Piers should be designed for an allowable end bearing pressure of 15,000 psf and a skin friction of 1,500 psf for the portion of the pier in bedrock. Uplift due to structural loadings on the caissons 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 15,000 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. 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 given above acts in the direction to resist uplift caused by swelling soil 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 three pier diameters or 8 feet into the bedrock, whichever is greater. Piers should also have a minimum 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 overburden soils of 50 tcf and a modulus of horizontal subgrade reaction of 250 tcf in the bedrock. The modulus value given is for a long one-foot wide pier and must be corrected for pier size.

5. Piers should be reinforced at their full length to resist an unfactored net tensile force from swelling soil pressure of at least 45 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 is 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.

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

Material	c (psf)	Ø	γ_T	k_s	k_c	ϵ_{50}	Soil Type
On-site Clayey Overburden Soils	500	0	125	500	200	0.007	1
On-site Clayey Sand Overburden	0	32	125	90	90	-	2
Bedrock	8,000	0	135	2,000	800	0.004	1

c Cohesion intercept (pounds per square foot)
φ Angle of internal friction (degrees)
γ 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. Stiff clay without free water (Reese)
2. Sand

7. A 4-inch void should be provided beneath the grade beams to concentrate pier loadings and to separate the expansive soil from the grade beams. 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 void should also be provided beneath necessary pier caps.

8. The minimum spacing requirements between piers should be three diameters from center to center. At this spacing, no reduction in axial or horizontal soil modulus values is required. Piers grouped less than three diameters from center to center should be studied on an individual basis to determine the appropriate reductions in both lateral and axial capacity.

9. The pier length-to-diameter ratio should not exceed 30.

10. 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.

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

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

13. Although the groundwater was not encountered in the exploratory borings, 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 using an approved tremie method.

14. Pier excavations in natural alluvial soils are likely to encounter cobbles and boulders. The presence of natural cobbles and boulders may also require casing as well as using coring tools.

15. The drilled shaft contractor should mobilize equipment of sufficient size and operating condition to achieve the required bedrock penetration.

16. Care should be taken that the pier shafts are not oversized at the top. Mushroomed pier tops can reduce the effective dead load pressure on the caissons.

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

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

Spread Footings: If the increased risks associated with shallow foundation movements are accepted by the owner, the design and construction criteria presented below should be observed for a spread footing foundation system. The construction details should be considered when preparing project documents.

1. Spread footings placed on a minimum of 3 feet of properly moisture conditioned and compacted structural fill extending to natural soils should be designed for an allowable soil bearing pressure of 2,500 psf. The footings should also be designed for a minimum dead load pressure of 800 psf. The 3 feet of structural fill should extend a minimum of 3 feet outside of footing limits.

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

3. 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.

4. The lateral resistance of a spread footing placed on properly compacted structural fill material 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 0.32. Passive pressure against the sides of the footings can be calculated using an equivalent fluid unit weight of 200 pcf. These lateral resistance values are working values.

   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.

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

6. 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 moisture conditioning and compaction of the exposed subgrade soils or removal of soft subgrade soils and replacement with new compacted structural fill.

7. A representative of the geotechnical engineer should observe all footing excavations, observe and test compaction, and evaluate the suitability of all fill materials prior to concrete placement.

SITE SEISMIC CRITERIA

The general soil profile across the site after construction will generally consist of relatively stiff/medium dense to hard/dense overburden soils extending to relatively weathered to very hard bedrock at depths of about 9 to 17.5 feet below the finished ground surface. Overburden consisting of new fill and/or existing overburden soils, and weathered to hard bedrock, will generally classify as Site Class C in accordance with the International Building Code (IBC).

Accordingly, we recommend IBC Site Class C be used for design in accordance with the International Building Code (IBC). Considering the subsurface profile and depth of groundwater, liquefaction is not a design consideration.

FLOOR SLABS

Floor slabs present a problem where expansive materials are present near floor slab elevation because sufficient dead load cannot be imposed on them to resist the uplift pressure generated when the materials are wetted and expand. We believe slab-on-ground construction may be used, provided the risk of distress resulting from slab movement is accepted by the owner. To reduce potential floor slab movement associated with the expansive nature of the subsoils, we recommend floor slabs be placed on a minimum of 5 feet of properly moisture-conditioned and compacted structural fill. The replaced material should consist of non-expansive material meeting the criteria presented in the Site Grading section of this report. It is also very important to provide the recommended isolation between the structure and the slab-on-grade floors to reduce damage in the event that slab movement occurs.

The following measures should be taken to reduce the damage that could result from movement should the underslab materials be subjected to moisture changes.

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

2. Interior nonbearing 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, 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 won't damage the partition wall. If slab bearing masonry block partitions are constructed, the slip joints will have to be placed at the tops of the walls. If slip joints are provided at the tops of walls and the floors move, it is likely the partition walls will show signs of distress, such as cracking. An alternative, if masonry block walls or other walls without slip joints at the bottoms are required, is to find them on grade beams and piers and to construct the slabs independently of the foundation. If slab-bearing partition walls are required, distress may be reduced by connecting the partition walls to the exterior walls using slip channels. Floor slabs should not extend beneath exterior doors or over foundation grade beams unless saw cut at the beam after construction.

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

4. If moisture-sensitive floor coverings will be used, mitigation of moisture penetration into the slabs, such as using a vapor barrier, may be required. If an impervious vapor barrier membrane is used, special precautions will be required to prevent differential curing problems, which could cause the slabs to warp. This topic is addressed by ACI 302.1R.

5. 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 expansive materials are subjected to moisture increases. However, the precautions should reduce the damage if such movement occurs.

EXTERIOR FLATWORK

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

It is extremely important exterior flatwork and pavements are isolated from the building foundations. Many problems associated with expansive soils are related to ineffective isolation between pavements and exterior slabs and foundation-supported components of structures. 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, the upward movement of exterior flatwork may restrict the movement of outward swinging doors. Site grading and drainage design should consider those possibilities, particularly at entryways.

UNDERDRAIN SYSTEM

We understand there will be no lower level constructed for the building. However, if areas of the lower slab level are below the adjacent exterior grade, we recommend an underdrain system be constructed at the base of the over-excavation and replacement fill zone to prevent the development of perched water in the fill. An underdrain system should consist of drain lines extending along the perimeter of the over-excavated zone in the areas where the slab is recessed. The alignment of the drain system should preferably be just outside of the structure perimeter. The drains should consist of a 4-inch diameter, rigid, perforated PVC pipe placed in trenches excavated to a depth of at least 6 inches below the base of the over-excavated zone.

The drainpipes 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 unless a filter geotextile is used on the slab subgrade and around underdrain trenches; in that case, 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. We are available to assist in the underdrain system design.

The base of the over-excavation should be graded to slope toward the drain lines with a minimum slope of 0.5%. The overall underdrain pipe system should be sloped at a minimum slope of 0.5% to an overall site sub-drain collection system or to a sump or sumps where water can be removed by pumping or gravity drainage. Sumps should be provided with alarms and/or redundant pumps in the event the pumping equipment malfunctions. In addition, the drain lines should be provided with appropriately spaced cleanouts for maintenance and inspection, which we recommend being performed on a routine basis. An over-designed sump and pump capacity are desirable in the event subsurface water conditions change. We also believe that standby pump capacity and standby generators should be provided in the event of pump or energy failure.

SURFACE DRAINAGE

Proper surface drainage is very important for the 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 building should be a relatively impervious on-site soil or be covered by impervious flatwork or a pavement structure.

4. Exterior backfill should be adjusted to 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 GRADING AND EARTHWORK

<u>Site Preparation</u>: Existing fill materials should be completely removed beneath shallow foundations, slabs-on-grade, pavement, and movement-sensitive flatwork and replaced with structural fill as recommended herein. Prepared subgrade for areas considered movement-sensitive such as floor slabs and exterior flatwork adjacent to the building, should consist of the removal of 5 feet of non-expansive fill, and the subgrade below new structural fill should be scarified to depths of 12 inches, moisture-conditioned and compacted in accordance with the following criteria for new fills. Subgrade preparation for areas not considered movement-sensitive should be completed in accordance with the flexible pavement subgrade preparations presented in the “Pavement Thickness Design” section of this report.

<u>Temporary Excavations</u>: 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. All existing fill should classify as OSHA Type C soils. The natural clay soils and claystone bedrock should classify as OSHA Type B soils. Some of the bedrock may classify as Type A soils depending on the level of fracturing. If localized perched water or seepage is encountered, much flatter side slopes than those allowed by OSHA, or temporary shoring may be required.

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

Fill Material Specifications: The following material specifications are presented for fills on the project site. We believe the onsite fill materials are not suitable for reuse as structural fill due to the presence of fat clay. The natural soils are suitable for reuse when used according to the specifications outlined below.

1. *Structural Fill*: Structural fill may consist of natural on-site soils that are moisture-conditioned and compacted, as recommended herein. Imported structural fill material, if necessary, should have a maximum of 50 percent passing the No. 200 sieve, a maximum liquid limit of 30, and a maximum plasticity index of 15. Imported fill materials not meeting the above liquid limit and plasticity index criteria may be acceptable provided the maximum percentage passing the No. 200 sieve specified above and the swell criteria outlined in Item 4 below are satisfied.

   Existing fat clay fill and claystone bedrock should not be used as structural fill placed in the building footprint.

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, claystone, and other deleterious substances, and should not contain rocks or lumps having a diameter of more than 4 inches. A fill material should be considered non-expansive if the swell potential under a 200 psf surcharge pressure does not exceed 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% of the optimum moisture content for granular materials and between optimum and 3% 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 Shallow 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.

WATER-SOLUBLE SULFATES

The concentrations of water-soluble sulfates measured in representative samples of the man-placed fill and the natural soils obtained from the site borings were below detectible limits. These concentrations of water-soluble sulfates represent a Class S0 severity of exposure to sulfate attack on concrete exposed to the native clay soils. The degree of attack is based on a range of Class S0 (not applicable), Class S1 (moderate), Class S2 (severe), and Class S3 (very severe) severity of exposure, as presented in ACI 201.2R-16.

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

PAVEMENT THICKNESS DESIGN

A pavement section is a layered system designed to distribute concentrated traffic loads to the subgrade. The 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, A-7-5, and A-7-6 soils and isolated zone of A-1-a soils with group indices between 0 and 41 in accordance with the American Association of State Highway and Transportation Officials (AASHTO) classification. Soils classified as A-6, A-7-5, and A-7-6 are generally considered to provide poor subgrade support. Soils classified as A-1-a are generally considered to provide excellent subgrade support. For design purposes, a resilient modulus value of 3,025 psi was selected for flexible pavements, and a modulus of subgrade reaction of 75 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 109,500 was assumed for drive and fire lane areas, trash collection points, and where turning points are concentrated (Heavy-Duty). The values are selected 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.

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

Pavement Thickness Requirements: Recommendations for a full-depth asphalt section, a composite section of hot mix asphalt (HMA) over aggregate base course (ABC), and for a rigid Portland cement concrete pavements (PCCP) section are presented in the table below. The pavement sections were determined in accordance with the 1993 AASHTO pavement design procedures. For design purposes, a design reliability of 80% was assumed for all pavement areas. The following table presents the minimum pavement thickness alternatives for the project:

LOCATION	Full Depth Asphalt Pavement (inches)	Asphalt Over Aggregate Base Course (inches)	PCCP (inches)
Automobile Parking	6.0	4.0 over 7.0	6.0
Drive Lanes	7.0	5.0 over 7.0	7.0

Dumpster pads and any areas of the pavement that will be subjected to concentrated truck turning movements should be paved using a minimum section consisting of 7.0 inches of PCCP. The PCCP thicknesses presented above are for unreinforced sections.

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

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

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

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

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

Subgrade Preparation: Prior to placing the pavement section, the entire subgrade area should be thoroughly scarified and well mixed to a minimum depth of 12 inches, adjusted to moisture content within 0 to +3 percentage points of optimum, for cohesive soils and within 2 percentage points of optimum for granular soils, and compacted to at least 95% of the standard Proctor maximum dry density. The pavement subgrade should be proofrolled with a heavily loaded pneumatic-tired vehicle. Pavement design procedures assume a stable subgrade. Areas which deform excessively under heavy wheel loads are not stable and should be removed and replaced to achieve a stable subgrade prior to paving. Areas of existing fill may also require deeper removal and replacement if they are either unstable or not well compacted.

Pavement design procedures assume a stable subgrade. Prior to placing the pavement section, the pavement subgrade should be proof-rolled with a heavily loaded pneumatic-tired vehicle with a tire pressure of at least 100 psi capable of applying a minimum load of 18-kips per axle. Areas that deform excessively under heavy wheel loads are not stable and should be removed and replaced to achieve a stable subgrade prior to paving. Areas where excessive deflection occurs should be ripped, scarified, wetted, or dried if necessary and re-compacted to the required moisture and density specifications.

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

DESIGN AND CONSTRUCTION SUPPORT SERVICES

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

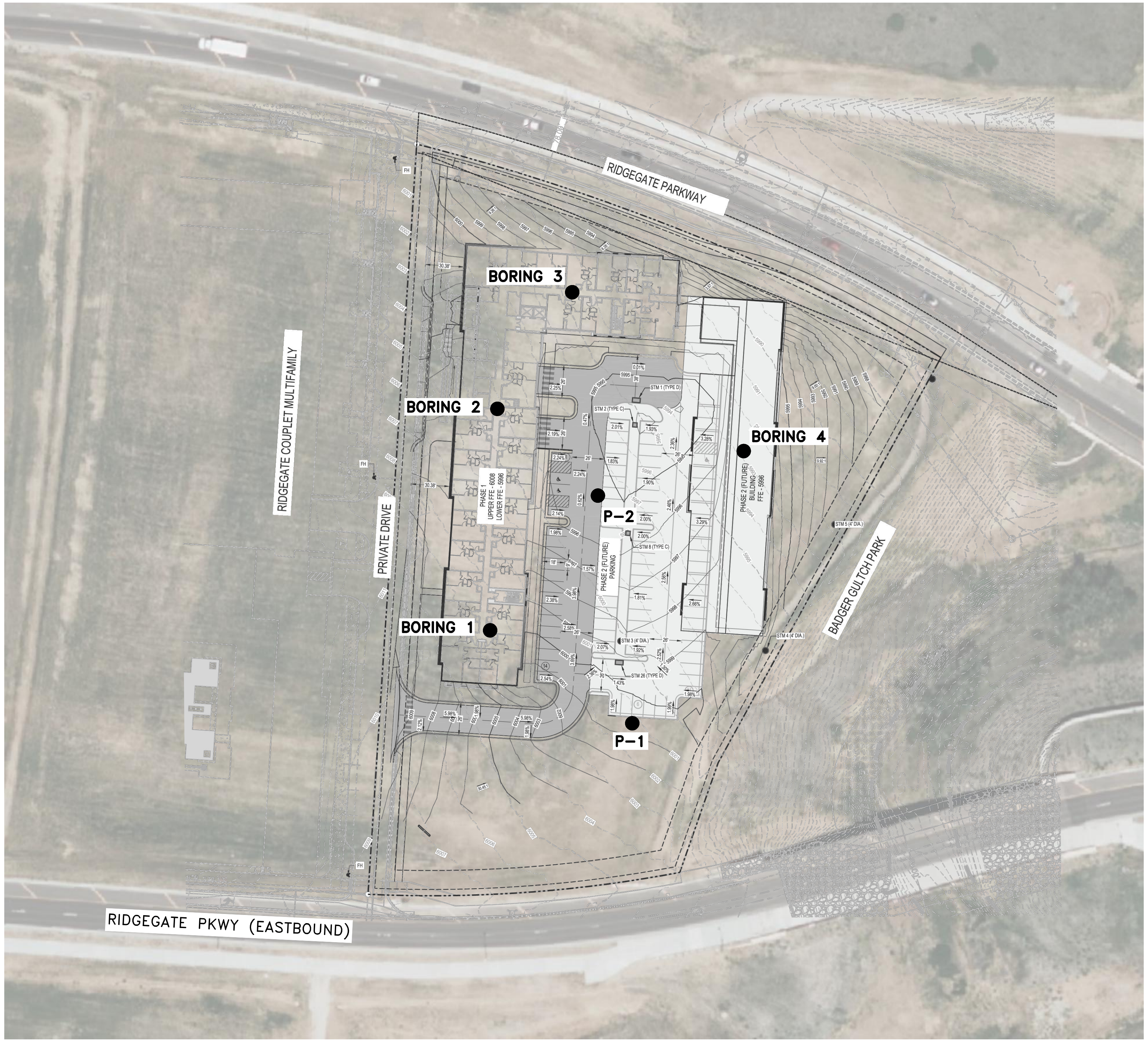
We recommend that K+A be retained to provide construction observation and testing services to document that the intent of this report and the requirements of the plans and specifications are being followed during construction. This will allow us to identify possible variations in subsurface conditions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

LIMITATIONS

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 that a re-evaluation of the recommendations presented in this report can be made. K+A is not responsible for liability associated with interpretation of subsurface data by others.

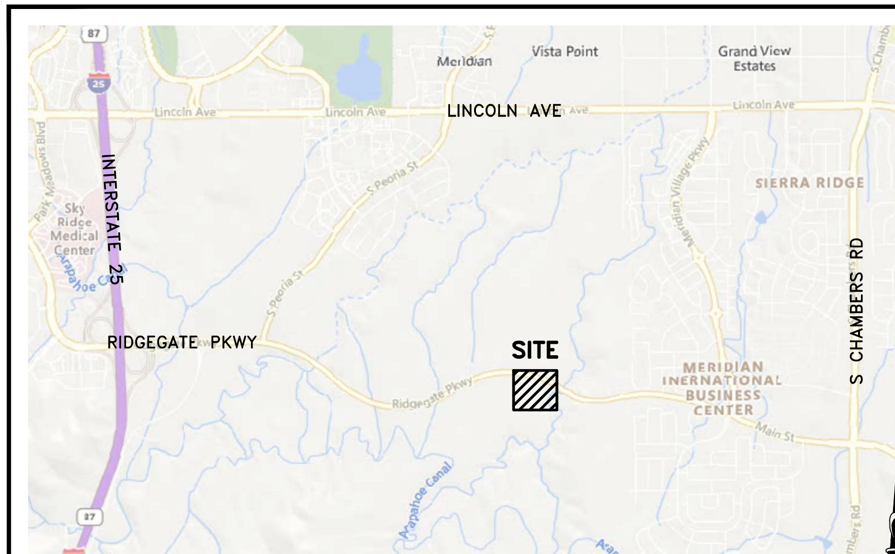
Swelling soils occur on this site. Such soils are stable at their natural moisture content but will undergo high volume changes with changes in moisture content. The extent and amount of perched water beneath the building site as a result of area irrigation and inadequate surface drainage is difficult,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

KMH/as
Rev. by: JAN
Enclosures
cc: file



50 0 50 100

APPROXIMATE SCALE-FEET



VICINITY MAP
NOT TO SCALE

23-1-198	Kumar & Associates	RIDGEGATE SENIOR HOUSING, LONE TREE, COLORADO	LOCATION OF EXPLORATORY BORINGS	Fig. 1

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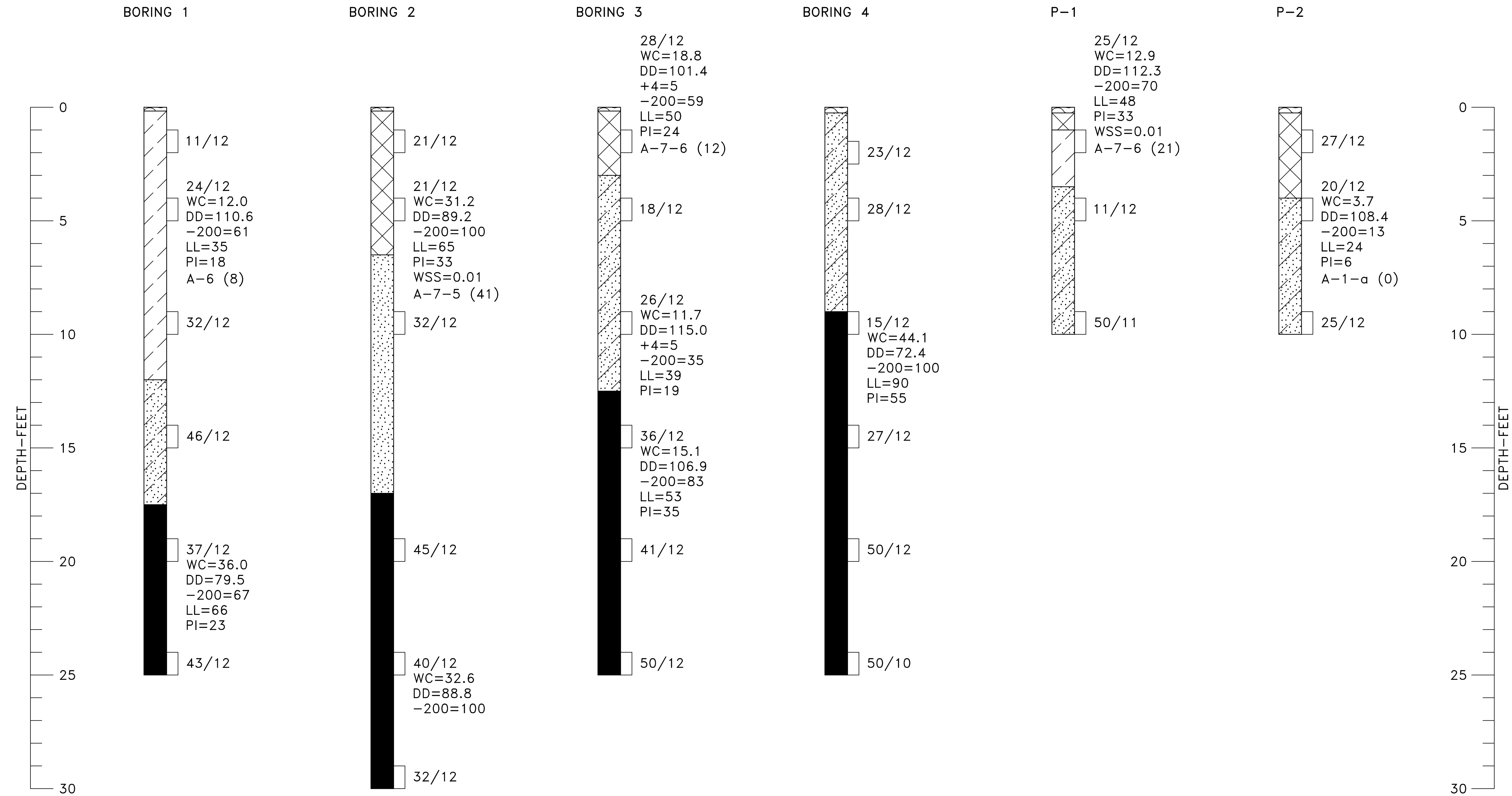


23-1-198	Kumar & Associates	RIDGEGATE SENIOR HOUSING, LONE TREE, COLORADO	LOGS OF EXPLORATORY BORINGS	Fig. 2

LEGEND

TOPSOIL.

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

LEAN CLAY (CL) WITH VARIABLE FINE- TO MEDIUM-GRAINED SAND FRACTION, STIFF TO HARD, MOIST, BROWN TO DARK BROWN, CALCAREOUS IN PLACES.

CLAYEY SAND (SC), FINE- TO COARSE-GRAINED WITH VARIABLE GRAVEL CONTENT AND OCCASIONAL COBBLES, DENSE, MOIST, BROWN TO ORANGE.

POORLY-GRADED SAND (SP), FINE- TO COARSE-GRAINED WITH VARIABLE GRAVEL CONTENT AND OCCASIONAL COBBLES, DENSE, MOIST, LIGHT BROWN TO BROWN.

SILTY, CLAYEY SAND (SC-SM), FINE- TO COARSE-GRAINED WITH VARIABLE GRAVEL CONTENT, MEDIUM DENSE TO DENSE, SLIGHTLY MOIST TO MOIST, WHITE TO ORANGE TO LIGHT BROWN.

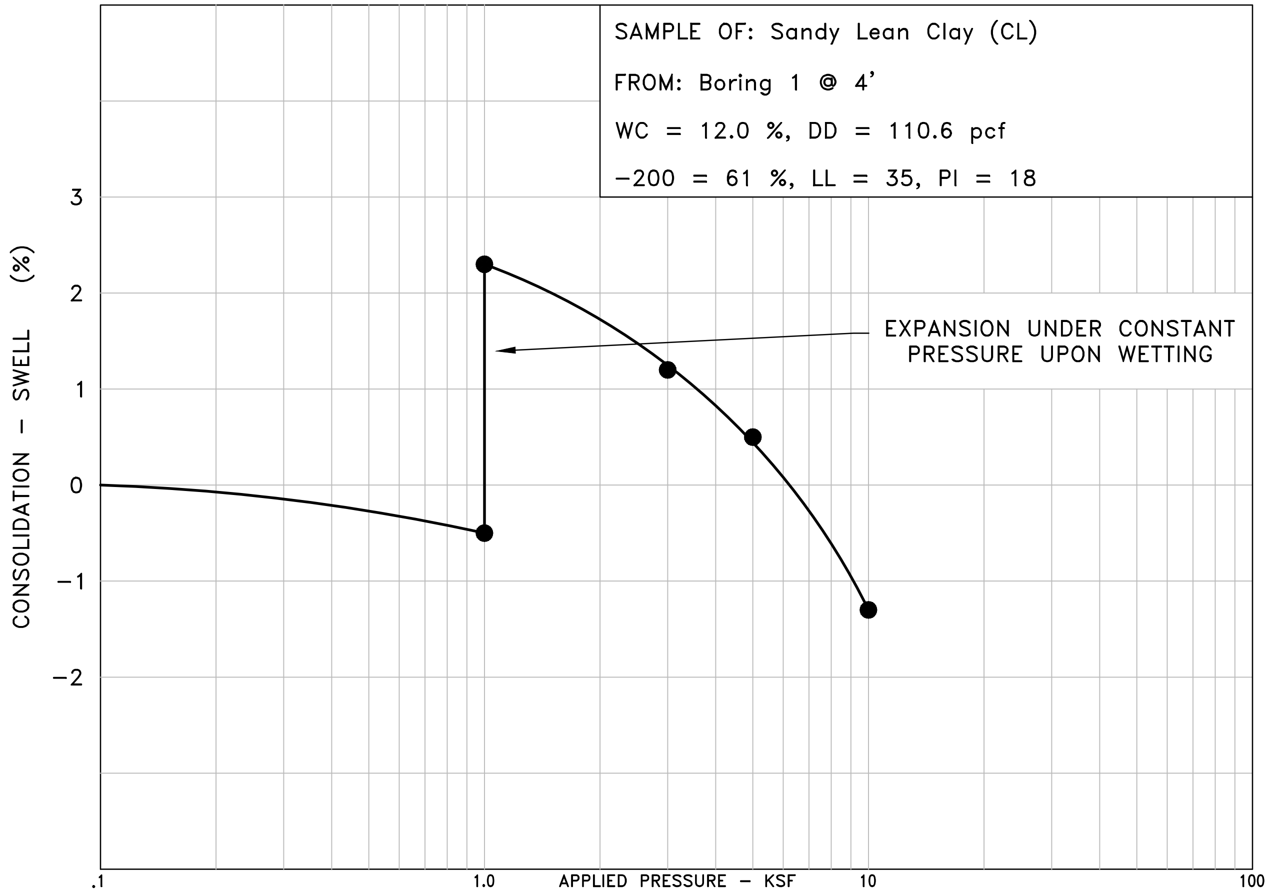
CLAYSTONE BEDROCK, HIGH PLASTICITY, FIRM TO HARD AND ISOLATED WEATHERED ZONE, VERY MOIST, ORANGE TO BROWN TO BLACK.

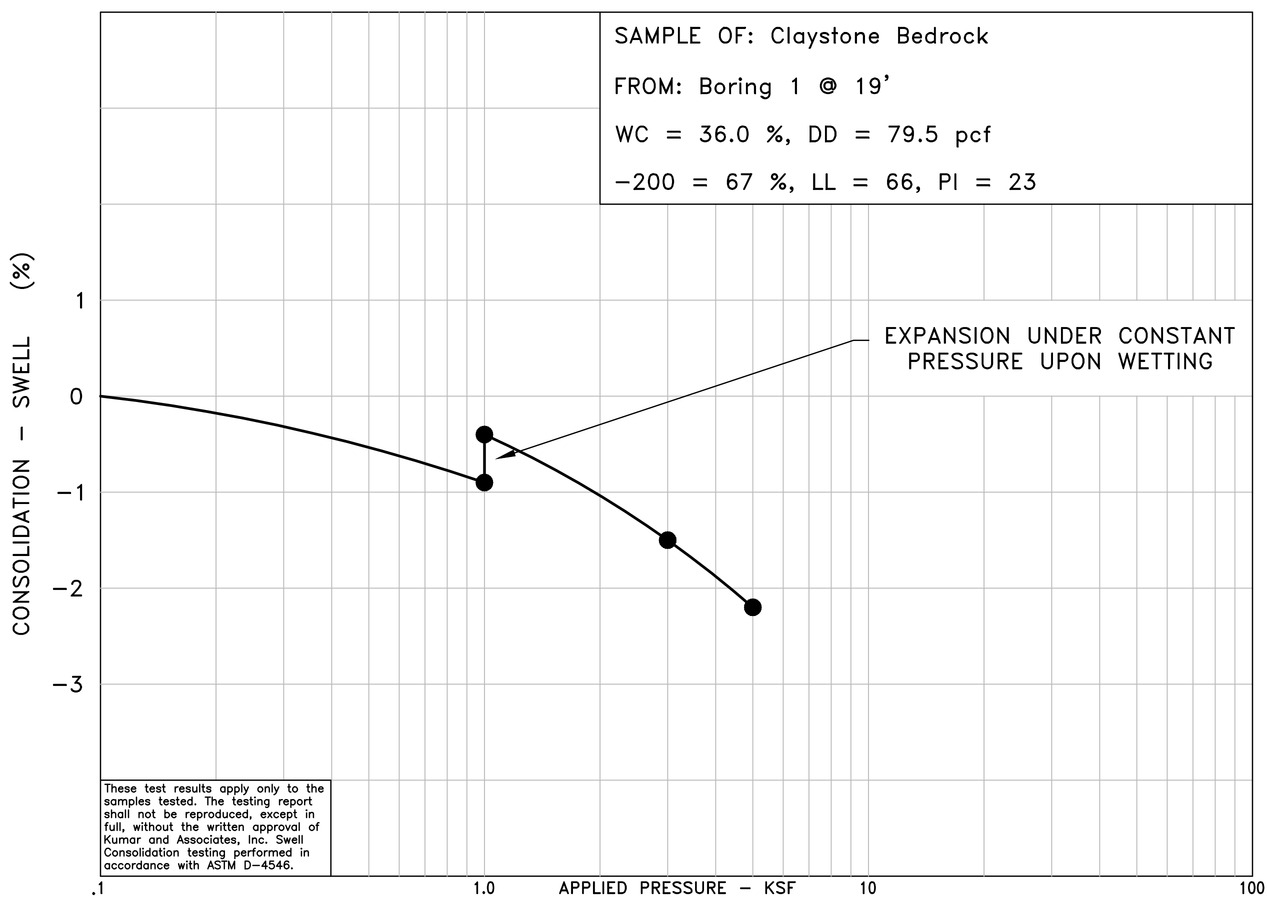
DRIVE SAMPLE, 2-INCH I.D. CALIFORNIA LINER SAMPLE.

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

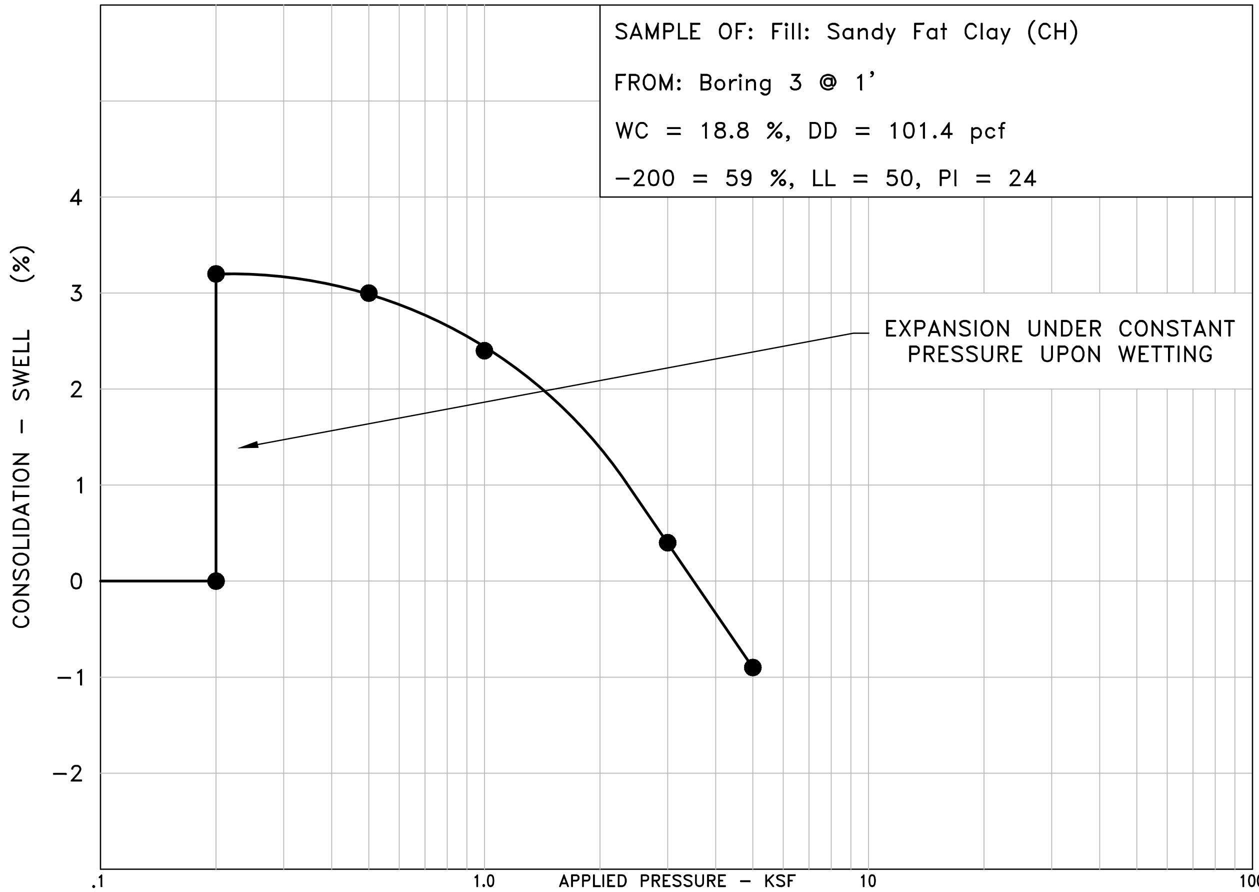
NOTES

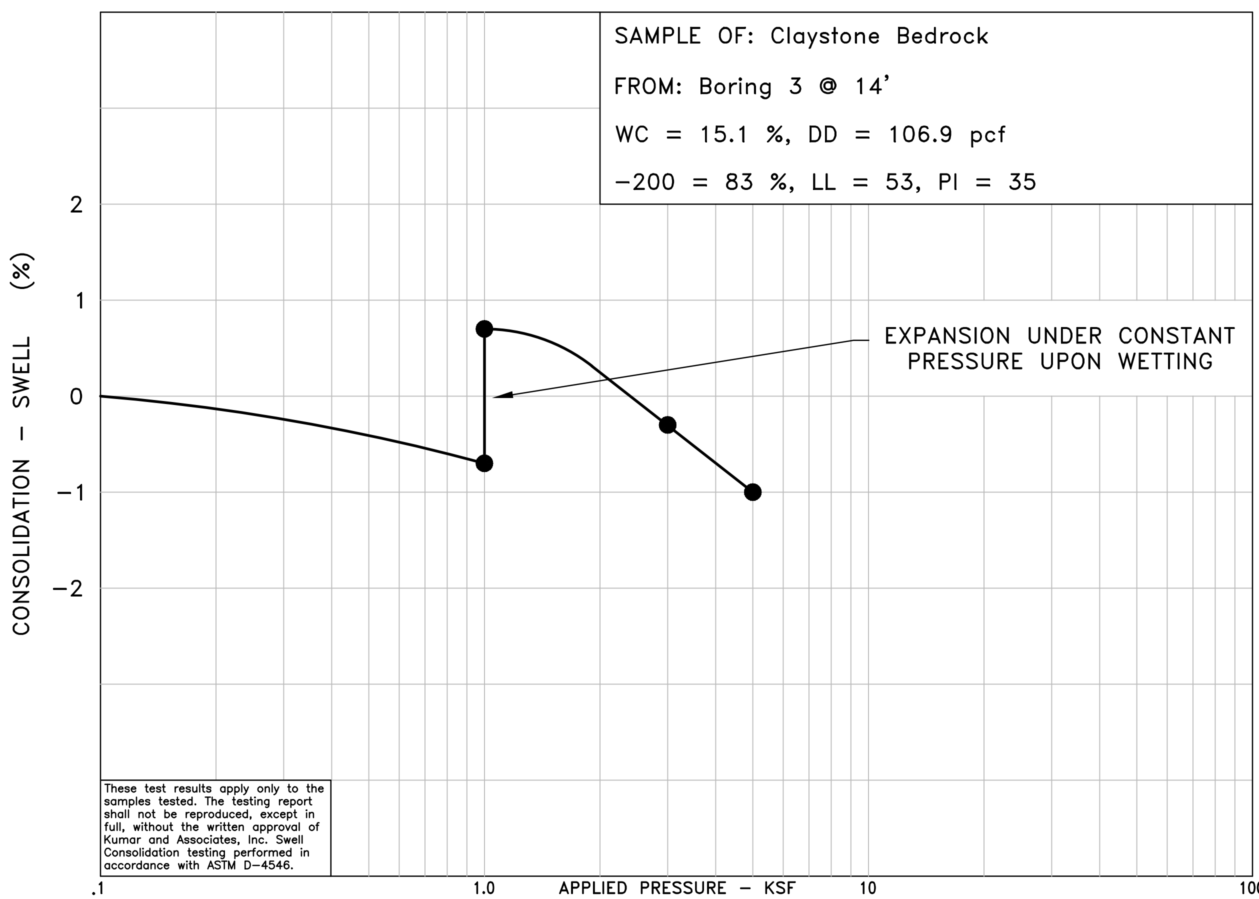
1. THE EXPLORATORY BORINGS WERE DRILLED ON MARCH 10, 2023 WITH A 4-INCH-DIAMETER CONTINUOUS-FLIGHT POWER AUGER.
2. THE LOCATIONS OF THE EXPLORATORY BORINGS WERE LOCATED BY GPS COORDINATES OBTAINED FROM GOOGLE EARTH AND LOCATED IN THE FIELD WITH A HANDHELD GPS UNIT.
3. THE ELEVATIONS OF THE EXPLORATORY BORINGS WERE NOT MEASURED AND THE LOGS OF THE EXPLORATORY BORINGS ARE PLOTTED TO DEPTH.
4. THE EXPLORATORY BORING LOCATIONS SHOULD BE CONSIDERED ACCURATE ONLY TO THE DEGREE IMPLIED BY THE METHOD USED.
5. THE LINES BETWEEN MATERIALS SHOWN ON THE EXPLORATORY BORING LOGS REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN MATERIAL TYPES AND THE TRANSITIONS MAY BE GRADUAL.
6. GROUNDWATER WAS NOT ENCOUNTERED IN THE BORINGS AT THE TIME OF DRILLING OR WHEN CHECKED 5 DAYS LATER.
7. LABORATORY TEST RESULTS:
   WC = WATER CONTENT (%) (ASTM D2216);
   DD = DRY DENSITY (pcf) (ASTM D2216);
   +4 = PERCENTAGE RETAINED ON NO. 4 SIEVE (ASTM D6913);
   -200 = PERCENTAGE PASSING NO. 200 SIEVE (ASTM D1140);
   LL = LIQUID LIMIT (ASTM D4318);
   PI = PLASTICITY INDEX (ASTM D4318);
   WSS = WATER SOLUBLE SULFATES (%) (CP-L 2103);
   A-6 (8) = AASHTO CLASSIFICATION (GROUP INDEX) (AASHTO M 145).



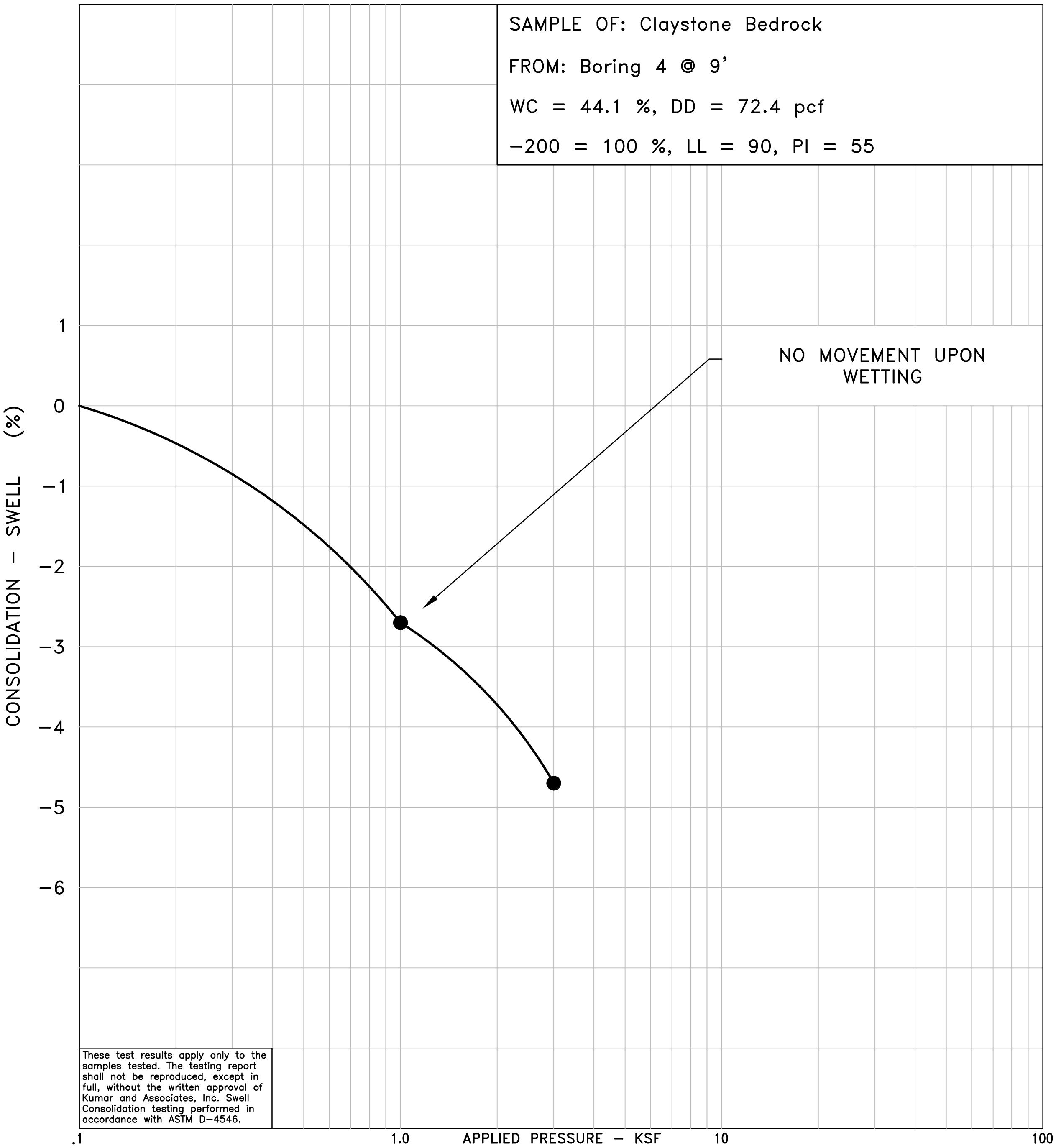


23-1-198	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULTS	Fig. 4





23−1−198	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULTS	Fig. 5



23-1-198	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULT	Fig. 6

TABLE I
SUMMARY OF LABORATORY TEST RESULTS

PROJECT NO.: 23-1-198
PROJECT NAME: Ridgegate Senior Housing - Ridgegate Parkway West of Meridian Village Parkway, Lone Tree, Colorado
DATE SAMPLED: 3/10/2023
DATE RECEIVED: 3/13/2023

SAMPLE LOCATION		DATE TESTED	NATURAL MOISTURE CONTENT (%)	NATURAL DRY DENSITY (pcf)	GRADATION		PERCENT PASSING NO. 200 SIEVE	ATTERBERG LIMITS		WATER SOLUBLE SULFATES (%)	AASHTO CLASSIFICATION (group index)	SOIL OR BEDROCK TYPE
BORING	DEPTH (feet)				GRAVEL (%)	SAND (%)		LIQUID LIMIT (%)	PLASTICITY INDEX (%)			
1	4	3/14/2023	12.0	110.6			61	35	18		A-6 (8)	Sandy Lean Clay (CL)
1	19	3/14/2023	36.0	79.5			67	66	23			Claystone Bedrock
2	4	3/14/2023	31.2	89.2			100	65	33	0.01	A-7-5 (41)	Fill: Fat Clay (CH)
2	24	3/14/2023	32.6	88.8			100					Claystone Bedrock
3	1	3/14/2023	18.8	101.4	5	64	59	50	24		A-7-6 (12)	Fill: Sandy Fat Clay (CH)
3	9	3/14/2023	11.7	115.0	5	60	35	39	19			Clayey Sand (SC)
3	14	3/14/2023	15.1	106.9			83	53	35			Claystone Bedrock
4	9	3/14/2023	44.1	72.4			100	90	55			Claystone Bedrock
P-1	1	3/14/2023	12.9	112.3			70	48	33	0.01	A-7-6 (21)	Sandy Lean Clay (CL)
P-2	4	3/14/2023	3.7	108.4			13	24	6		A-1-a (0)	Silty, Clayey Sand (SC-SM)