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GEOTECHNICAL ENGINEERING STUDY
PROPOSED 7-ELEVEN STORE
SWC OF BRADLEY ROAD & LEGACY HILL DRIVE
COLORADO SPRINGS, COLORADO

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SUMMARY

1. The generalized subsurface profile encountered in the borings consisted of man-placed fill, underlain by native granular and clay overburden soils. Claystone bedrock was encountered below the overburden soils in five of the borings, beginning at depths of approximately 16 to 42 feet, and extending to the maximum 25 to 45-foot depths explored. The fill extended to depths ranging from approximately 13 to 19 feet (and to the maximum 11-foot depth explored in two borings), and consisted of a mixture of cohesive and granular soil types, including sandy lean clay (CL) and silty to clayey sand (SM, SC, SC-SM). The native overburden soils encountered consisted of granular and clay soil types, to include silty sand (SM), clayey sand (SC), and sandy lean clay (CL).

2. At the time of drilling, groundwater was encountered in two of the borings at depths of approximately 32 to 33 feet. When measurements were made 7 days after drilling, groundwater was measured in three borings at depths ranging from approximately 23.1 feet to 27.9 feet. Fluctuations in the water level may occur with time, particularly during wetter seasons and after precipitation events.

3. With proper site preparation, shallow spread footings with slab-on-grade floors or mat foundations should be feasible. Proper site preparation should include complete removal of existing non-engineered fills where present within the proposed structure footprints, down to the natural soils and replacement with properly compacted structural fill. New fill should extend down from the edge of foundations at a minimum 1:1 horizontal to vertical projection.

4. Drilled piers or helical piers end bearing in the native soils or bedrock at depth would also be a suitable foundation alternative, and would have an advantage over a shallow foundations in that overexcavation of the existing fill would not be required. The foundation recommendations included in the report include spread footing/mat foundations, drilled piers, and helical piers.

5. The existing fill encountered may be suitable for reuse as structural fill beneath foundations, floor slabs, and pavements. The existing fill was encountered throughout the site at depths of approximately 13 to 19 feet, with an average of about 16 feet. The “Site Grading and Earthwork” section of the report provides additional discussion, along with grading compaction criteria.

6. Pavement section alternatives based on the provided traffic volumes, on-site material properties, and the 7-Eleven pavement guidelines and standards are presented below:

LOCATION	Asphalt Over Aggregate Base Course (inches)	PCCP (inches)
Standard Duty	4.25 over 6.0	6.0
Heavy Duty	8.0 over 12.0	8.5

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 8.5 inches of PCCP. The PCCP thicknesses presented above are for unreinforced sections.

PURPOSE OF STUDY

This report presents the results of a geotechnical engineering study for the proposed 7-Eleven development to be located near the southwest corner of Bradley Road and Legacy Hill Drive in Colorado Springs, Colorado. The project site is shown on the attached Fig. 1. The was performed in accordance with our Proposal No. P-25-253R dated September 25, 2025, to develop recommendations for site grading, foundations, and pavements. This revision has been made to incorporate revised pavement section recommendations, based on updated traffic loading information that was provided to us on November 17, 2025.

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

PROPOSED CONSTRUCTION

We understand the convenience store building will be single-story, with a footprint area of approximately 4,800 sf as shown on Fig. 1. Framing is anticipated to utilize structural insulated panels (SIP's) resting on the foundation system. Underground storage tanks (UST's) are planned near the east portion of the site with associated product dispensing islands to be located between the proposed building and the UST's. The dispensing islands will be covered by overhead canopies. Areas away from the structures will be paved to provide access in and around the site for vehicles. A dumpster enclosure will be located near the northwest corner of the project site, and a 25-foot tall sign structure will be located at the northeast corner.

The final site grading plans were not provided at the time of the preparation of this report, and we are unaware of the planned cuts/fills to achieve the finished grades and finished floor elevations at the time of this report; however, we assume final grading will be near the existing ground surface elevations. If the proposed construction varies significantly from that described above or depicted herein, we should be notified to reevaluate the recommendations provided herein.

SITE CONDITIONS

The subject site is located at the southwest corner of the intersection of Legacy Hill Drive and Bradley Road as shown on Fig. 1. The property consists of a vacant lot with a small detention basin near the northeast corner of the site. The lot itself is raised above Bradley Road to the north and Legacy Hill Drive to the east by approximately up to 15 feet. Vacant lots are situated west and south of the subject site, with more vacant land east of Legacy Hill Drive. A residential neighborhood is located beyond the roadways, to the south and southeast of the property. The site is generally level beyond the slopes raising the site above the surrounding roadways. An approximately two-foot-tall soil berm borders the south edge of the site which then trends northeast toward the detention basin, transecting the eastern third of the subject site. The land east and south of the berm is generally vegetated with natural weeds and grass, while the majority of the site itself has been scarified and is without vegetation.

FIELD EXPLORATION

The field exploration of subsurface conditions consisted of drilling a total of seven borings at the approximate locations shown on Fig. 1. The borings were drilled on October 10th, 2025, and the locations were approximated by using a handheld GPS device. The boring logs and corresponding legend and notes are presented on Figs. 2 and 3, respectively.

The borings were drilled with continuous flight 4-inch diameter solid stem augers. The borings were logged by representatives of Kumar & Associates, Inc. Samples of the soils were taken with a 2-inch I.D. California sampler. The sampler was driven into the various strata with blows from a 140-pound hammer falling 30 inches. Penetration resistance values, when properly evaluated, provide an indication of the relative density or consistency of the soils. Depths at which the samples were taken and the penetration resistance values are shown on the boring logs.

LABORATORY TESTING

Samples obtained from the exploratory borings were visually classified in the laboratory by the project engineer and samples were selected for laboratory testing. Laboratory testing included index property tests such as in-situ moisture content and dry unit weight, grain size analysis, and Atterberg limits. Additional testing included soils moisture-density relationships (Proctor) and concentration of water soluble sulfates. The testing was conducted in general accordance with recognized test procedures, primarily those of the American Society for Testing of Materials (ASTM). Results of the laboratory testing program are shown on Figs. 2, and 4 thru 11, and are summarized on Table I.

SUBSURFACE CONDITIONS

Summary: The generalized subsurface profile encountered in the borings consisted of man-placed fill, underlain by native granular and clay overburden soils. Claystone bedrock was encountered below the overburden soils in five of the borings, beginning at depths of approximately 16 to 42 feet, and extending to the maximum 25 to 45-foot depths explored. The fill extended to depths ranging from approximately 13 to 19 feet (and to the maximum 11-foot depth explored in two borings), and consisted of a mixture of cohesive and granular soil types, including sandy lean clay (CL) and silty to clayey sand (SM, SC, SC-SM). The native overburden soils encountered consisted of granular and clay soil types, to include silty sand (SM), clayey sand (SC), and sandy lean clay (CL). The following subsurface descriptions are of a generalized nature to highlight the soil types encountered in the borings drilled for this study. The boring logs should be reviewed for more detailed information.

Existing Fill: Man-placed fill was encountered in each of the borings to depths ranging from approximately 13 to 19 feet (and to the maximum 11-foot depth explored in two borings), with an average of roughly 16 feet for those borings that extended beyond the fill. Review of available aerial photos suggest that site grading activities occurred in 2024. The fill consisted of a mixture of cohesive and granular soil types, including sandy lean clay (CL) and silty to clayey sand (SM, SC, SC-SM). The fill was slightly moist to moist, and mottled shades of tan, brown, and gray. Our study did not determine the exact lateral or vertical extent of the fill. Swell-consolidation test results presented on Figs. 4 and 5 indicate the tested samples of fill had a nil to low swell potential when wetted under a 1,000 psf surcharge. Standard Proctor (ASTM D698) test results presented on Fig. 11 indicate a maximum dry density of 115.1 pcf and an optimum moisture content of 10.2% for the tested sample of silty-clayey sand fill.

Native Granular Soils: The native granular soils encountered included silty sand (SM), with occasional layers clayey sand (SC). These soils were encountered below the fill in 3 of the 5 borings as shown on the boring logs, Fig. 2. The native granular soils were slightly moist to wet (below the groundwater level), and tan, brown, and gray in color. Sampler penetration blow counts indicate the granular soils ranged from loose to medium dense.

Native Clay Soils: Zones of sandy lean clay (CL) were encountered in two of the borings at depth, as shown on the boring logs, Fig. 2. The clay included occasional layers of clayey sand (SC). The clay soils were slightly moist to wet (below the groundwater level), and tan, brown, and gray. Sampler penetration blow counts indicate the clay soils are medium stiff to very hard.

Claystone Bedrock: Claystone bedrock was encountered below the overburden soils in Borings 1 thru 5, beginning at depths of approximately 16 to 42 feet, and extending to the maximum 25 to 45-foot depths explored. The bedrock was moist, and gray, tan, and brown in color. Sampler penetration blow counts indicate the bedrock is hard to very hard. Swell-consolidation test results presented on Figs. 4 thru 6 indicate the tested samples of bedrock had a low swell potential when wetted under a 1,000 psf surcharge.

Groundwater: At the time of drilling, groundwater was encountered in two of the borings at depths between approximately 32 and 33 feet. When measurements were made 7 days after drilling, groundwater was measured in three borings at depths ranging from approximately 23.1 feet to 27.9 feet. Fluctuations in the water level may occur with time, particularly during wetter seasons and after precipitation events. The borings were backfilled with auger cuttings upon completion of water level measurements.

GEOTECHNICAL ENGINEERING CONSIDERATIONS

As discussed previously, varied depths of existing fill was encountered throughout the site. Sampler penetration blow counts and measured in-place moisture and density values suggest zones of the existing fill are marginally compact. Foundations and floor slabs placed on uncontrolled fill can experience large total and differential movement resulting structural distress. For these reasons, it is our opinion that the fill should be considered unsuitable for support of the proposed structures.

With proper site preparation, shallow spread footings with slab-on-grade floors or mat foundations should be feasible. Proper site preparation should include complete removal of existing non-engineered fills where present within the proposed structure footprints, down to the natural soils and replacement with properly compacted structural fill. New fill should extend down from the edge of foundations at a minimum 1:1 horizontal to vertical projection. The intent of this recommendation is to provide a low risk of settlement beyond about 1-inch in magnitude.

Drilled piers or helical piers end bearing in the native soils or bedrock at depth would also be a suitable foundation alternative, and would have an advantage over a shallow foundations in that overexcavation of the existing fill would not be required. The foundation recommendations that follow include spread footing/mat foundations, drilled piers, and helical piers. If other foundation alternatives are preferred, we should be consulted.

The existing fill encountered may be suitable for reuse as structural fill beneath foundations, floor slabs, and pavements. The existing fill was encountered throughout the site at depths from approximately 13 to 19 feet, with an average of about 16 feet. The “Site Grading and Earthwork” section of the report provides additional discussion.

Provided the owner understands and accepts the potential for differential subgrade settlement and the resulting increased potential for distress, it is our opinion that partial overexcavation of the existing fill can be considered for pavement areas and areas of exterior flatwork, as these items can typically tolerate some movement and are more easily repaired than structures. At a minimum, we recommend overexcavation and replacement of the existing fill to a minimum depth of 2 feet in pavement and exterior flatwork areas. Increasing the depth overexcavation will reduce the risk and magnitude of potential movements.

FOUNDATION RECOMMENDATIONS

Spread Footings/Mats: The design and construction criteria presented below should be observed for a spread footing or mat foundation system. The construction details should be considered when preparing project documents.

1. We recommend that spread footing and mat foundations be supported on properly compacted structural fill. Any areas of existing fill or loose or soft material encountered within the foundation excavation should be removed and replaced with suitable nonexpansive fill. New structural fill should extend down from the edges of the foundations at a minimum 1 horizontal to 1 vertical projection.

2. Suitable structural fill should meet the material and placement requirements outlined in the "Site Grading and Earthwork" section of this report.

3. Spread footing or rigid mat foundations bearing on properly compacted nonexpansive fill should be designed for an allowable soil bearing pressure of 2,500 psf.

4. If the mat cannot be considered rigid, the soil pressure distribution should be computed using a method which models the soil-structure interaction, such as the beam on an elastic foundation procedure. A modulus of vertical subgrade reaction of 125 pci can be used for compacted on-site soils. As an alternate, a modulus of 250 pci can be used if the mat is supported on a minimum of 2 feet of Class 6 aggregate base course. These values are based on correlations to a 1 ft. x 1 ft. sample plate and should be decreased as appropriate to account for the actual foundation size and geometry.

5. The weight of the soil above the foundation may be used to resist uplift. We recommend a soil unit weight of 110 pcf be assumed for compacted fill placed over foundations for this purpose.

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

7. 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 30 inches below the exterior grade is typically used in this area.

8. Criteria for the lateral resistance of a spread footing placed on native granular materials or properly compacted structural fill is presented in the "Foundation Walls and Retaining Structures" section of this report.

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

10. Granular foundation soils should be densified with a smooth vibratory compactor prior to placement of formwork and reinforcing steel.

11. A representative of the geotechnical engineer should confirm proper subgrade preparations have been met prior to placing foundation formwork. Loose disturbed material should be removed from the foundation subgrade prior to placement of concrete. Placement of structural fill should be observed and tested by a representative of the geotechnical engineer. In addition, representatives of the geotechnical and/or structural engineer should check reinforcement placement immediately prior to concrete placement.

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

1. Piers should have a minimum length of 20 feet. The allowable end bearing pressure and allowable skin friction for piers bearing in the native soils or claystone are presented in the following table. Structural uplift can be resisted by using 75 percent of the allowable skin friction value plus an allowance for pier weight.

End Bearing Material	Allowable End Bearing Pressure (psf)	Allowable Skin Friction (psf)
Native Soils (min 20' deep)	5,000	150*
Claystone Bedrock (w/min embedment of 3 pier diameters or 5' into bedrock, whichever is greater)	25,000	2,000

*: The skin friction value presented should be neglected for the upper 6 feet of overburden soil.

2. Applicable to piers end bearing in claystone, piers should also be designed for a minimum dead load pressure of 5,000 psf calculated as the unfactored dead load applied to the pier cross sectional area. Our experience indicates application of dead load pressure is the most effective way to resist foundation movement due to swelling materials. However, if the minimum dead load requirement cannot be achieved and the piers are loaded as heavily as practicable, 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 rock around the upper portion 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. The parameters provided in the following table are intended to assist in the analysis of the lateral capacity of the pier using the LPile computer program. The values provided in the table are for use with that software application only and may not be appropriate for other usages.

MATERIAL	k_s (psi/in)	k_c (psi/in)	ϕ (degrees)	c (psi)	ε_{50}	γ_m / γ_b (pci)	Soil Model Type
Existing Fill	25	25	27	-	-	0.064	Sand (Reese)
Native Granular Soils	90	90	30	-	-	0.069/0.033	Sand (Reese)
Native Clay Soils	500	200	-	13	0.007	0.058/0.022	Stiff clay w/o water
Bedrock	2000	800	-	55	0.004	0.067	Stiff clay w/o water

k_s = modulus of subgrade reaction for static loading
k_c = modulus of subgrade reaction for cyclic loading
ϕ = angle of internal friction
c = undrained shear strength
ε_{50} = strain at 50% of peak strength
γ_m = moist unit weight
γ_b = buoyant unit weight

4. Piers should be reinforced their full length to resist an unfactored net tensile force from possible swelling soil of at least 30 kips. The recommended tensile force is for an 18-inch diameter pier and should be increased or reduced in proportion to the pier diameter for piers of different diameter.

5. A 4-inch void should be provided beneath the grade beams to concentrate pier loadings and to separate the expansive materials 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 any necessary pier caps.

6. Closely spaced piers may require appropriate reductions of the lateral and axial capacities. Reduction in lateral load capacity may be avoided by spacing the piers at least 5 “equivalent” pier diameters center to center in the direction parallel to pier loading. For axial loading, the piers should be spaced a minimum of 3 pier diameters center to center. Piers placed closer than that indicated above should be studied on an individual basis to determine the appropriate reduction in axial and lateral load design parameters.

   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 pier capacity. The modified p-y curve should then be reentered into the LPile software to calculate the pier deflection. The reduction in capacity for the leading pier, the pier leading the direction of movement of the group, is less than that for the trailing piers.

For pier spacing that is 5 or less diameters in the direction of loading, we recommend using the following p-multipliers to account for the effects of group interaction.

Center to Center Spacing in the direction of Loading	p-multipliers (p_m)		
	Row 1	Row 2	Row 3
3 diameters	0.80	0.40	0.30
5 diameters	1.00	0.85	0.70

P-multiplier values for other pier spacing values should be determined by interpolation. 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 use of a pier cap would prevent differential movement between the piers.

7. A minimum pier diameter of 18 inches is recommended to facilitate proper cleaning and observation of the pier hole. A maximum length to diameter ratio of 30:1 should also be maintained.

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

9. Based on the results of our experience with similar, properly constructed drilled pier foundations, we estimate pier movement will not exceed approximately 1 inch when end bearing in the native soils, and should not exceed about ½ inch when end bearing in bedrock.

10. Drilled pier shafts should be properly cleaned prior to the placement of concrete.

11. The presence of water and granular soils indicates the use of temporary casing or dewatering equipment in the pier holes may be required to address caving and/or water

infiltration. The requirements for casing and dewatering equipment 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 the tremie method is used.

12. The drilled shaft contractor should mobilize equipment of sufficient size and operating condition to achieve the required depths. A contractor with experience drilling in similar conditions should be used.

13. 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 piers.

14. Concrete should be placed in piers 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 the day of drilling will normally result in a requirement for additional penetration.

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

<u>Helical Piers</u>: The design and construction criteria presented below should be observed for a helical pier foundation system. The construction details should be considered when preparing project documents.

The axial design load of helical piers should be determined in general accordance with 2021 IBC, which states the allowable axial design load, Pa, should be determined as follows:

Pa= 0.5 Pu, where Pu (the ultimate load) is the least value of:

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

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

Based on consideration of bearing capacity theory and published correlations of boring penetration resistance values with ultimate bearing capacity, we recommend an ultimate bearing capacity for a helical pier embedded a minimum of 20 feet into the native soils of 15,000 psf. Helical bearing plates should be installed to a minimum depth of 20 feet. Depending on the size and quantity of helix plates designed, the pier lengths may need to extend to greater depths to achieve the capacity needed. Although greater capacities may be achievable when helixes are at or near the bedrock interface, and it is expected that minimal penetration into bedrock will be achievable.

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

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

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

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

SITE SEISMIC CRITERIA

Using estimated shear wave velocities for the subgrade materials encountered based on standard penetration testing, calculations for an assumed 100-foot profile indicate a design Site Class D per the International Building Code (IBC). Based on the subsurface profile and site seismicity, liquefaction is not a design consideration. Using an online database tool that references USGS maps, the following probabilistic ground motion values are reported for the project site address.

Intensity Measure Type	Intensity Measure Level 2% in 50 Years
MCE_R Ground Motion, 0.2s Period S_s	0.187 g
MCE_R Ground Motion, 1.0s Period S_1	0.056 g

FLOOR SLABS

Based on the subsurface conditions encountered, we believe light to moderately loaded slab-on-grade construction will be suitable for the proposed building, provided the existing fill is removed and replaced with properly compacted structural fill. This option makes most sense where a shallow foundation is utilized, as overexcavation of the fill would be simultaneously completed as part of the foundation preparation. As an alternative, where drilled piers or helical piers are utilized, the floor can be constructed as a structural floor above a well ventilated crawlspace or void form. The floor would be supported on grade beams and piers the same as the structure. With a structurally supported floor, overexcavation of the fill would not be necessary.

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

1. Existing fill encountered below the floor slab bearing elevation should be overexcavated and replaced with suitable structural fill per the depths and criteria presented in the "Site Grading and Earthwork" section of the report.

2. We recommend a modulus of subgrade reaction of 100 pci be used for design of slabs.

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

4. Interior non-bearing partitions resting on floor slabs should be provided with slip joints at the bottoms 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 which will allow at least 2 inches of vertical movement are recommended.

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

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

7. If moisture-sensitive floor coverings will be used, mitigation of moisture penetration into the slabs, such as by use of a vapor barrier, may be required. If an impervious vapor barrier membrane is used, special precautions will be required to reduce potential differential curing

problems which could cause the slabs to warp. Section 302.1R of the ACI Manual of Concrete Practice addresses this topic.

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

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.

FOUNDATION WALLS AND RETAINING STRUCTURES

Foundation walls and retaining structures which are laterally supported and can be expected to undergo only a moderate amount of deflection should be designed for a lateral earth pressure computed on the basis of an equivalent fluid unit weight of 65 pcf for backfill consisting of the on-site soils, or 50 pcf if imported CDOT Class I structural backfill is used. Cantilevered retaining structures which can be expected to deflect sufficiently to mobilize the full active earth pressure condition should be designed for a lateral earth pressure computed on the basis of an equivalent fluid unit weight of 55 pcf for backfill consisting of the on-site granular soils, or 40 pcf for CDOT Class I structural backfill.

All foundation and retaining structures should be designed for appropriate hydrostatic and surcharge pressures such as adjacent buildings, traffic, construction materials and equipment. 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 foundation wall or retaining structure.

The lateral resistance of a foundation or retaining wall footing placed on undisturbed native granular soils or properly compacted structural fill material will be a combination of the sliding resistance of the foundation on the foundation materials and passive earth pressure against the side of the footing. Resistance to sliding at the bottoms of the footings may be calculated based on an allowable coefficient of friction of 0.35. Passive pressure against the sides of the footings may be calculated using an allowable equivalent fluid unit weight of 190 pcf.

The onsite soils are suitable for use as wall backfill. Imported granular wall back fill, if used, should meet the requirements of a CDOT Class I structural backfill with less than 20% passing the No. 200 sieve. Proposed material should be approved by the geotechnical engineer prior to use.

The granular backfill behind foundation and retaining walls should be sloped from the base of the wall at an angle of at least 45 degrees from the vertical. Backfill should be placed in uniform lifts and compacted to the criteria presented in the “Site Grading and Earthwork” section of the report. Care should be taken not to overcompact the backfill since this could cause excessive lateral pressure on the walls. Some settlement of deep foundation wall backfills will occur even if the material is placed properly.

EXTERIOR FLATWORK

As discussed in the “Geotechnical Engineering Considerations” section, provided the owner understands and accepts the potential for differential subgrade settlement and the resulting increased potential for distress, partial overexcavation of the existing fill in exterior flatwork areas can be considered, as these items can typically tolerate movement and are more easily repaired. At a minimum we recommend overexcavation and replacement of the existing fill to a minimum depth of 2 feet in exterior flatwork areas. Increasing the depth overexcavation will reduce the risk and magnitude of potential movements. Fill placed for support of exterior flatwork should meet the material and compaction requirements for structural fill presented in the “Site Grading and Earthwork” section of this report.

It is extremely important that exterior flatwork and pavements be isolated from the building foundations and other structures. Careful design detailing is necessary at locations such as exterior landings and entry points.

Proper surface drainage measures as recommended in the “Surface Drainage” section of this report are also critical to reducing moisture or frost-related movement. Upward heave-related movement or settlement of fill of exterior flatwork adjacent to the buildings may result in adverse drainage conditions with runoff directed toward the structure. 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. Positive drainage and grades should be maintained throughout the life of the facility.

WATER SOLUBLE SULFATES

The concentration of water soluble sulfates measured in a sample of the on-site soils obtained from the borings was less than 0.01%. This concentration of water soluble sulfates represent

Class S0 severity exposures to sulfate attack on concrete exposed to these materials. The degree of attack is based on a range of Class S0, Class S1, Class S2, and Class S3 severity exposure as presented in ACI 201. Based on the laboratory data and our experience, we believe special sulfate resistant cement will not be required for concrete exposed to the on-site soils.

UNDERDRAIN SYSTEM

We recommend that an underdrain system be used for the crawlspace or void form space if a structural floor system is used. If void form is used, a minimum 12-inch void is recommended.

The underdrain system should consist of a drain extending along the perimeter of the building. The alignment of the drain system should preferably be just outside of the building perimeter, but far enough away that the drain doesn’t interfere with drilled pier foundation pier construction. The drains should consist of 4-inch diameter, rigid, perforated PVC pipe placed in the bottom of a trench and surrounded above the invert level with free-draining granular material. Free-draining gravel used in the drain system should contain less than 5% passing the No. 200 sieve, less than 30% passing the No. 4 sieve and have a maximum size of 2 inches. The free-draining backfill should extend up to the crawlspace (or base of void form) level. The perimeter drain cross section (including gravel) should be at least 12 inches in diameter and should be wrapped with filter fabric, Mirafi 140N or equivalent. The drain lines should be placed at least 1 foot below the surface of the crawl space (or base of void form) and graded to a sump pit or gravity discharge at a minimum slope of ½ percent.

SURFACE DRAINAGE

Proper surface drainage is very important for acceptable performance of the development 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. 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 buildings and other structures should be sloped to drain away from the foundations in all directions. We recommend a minimum slope of 6 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 the paved areas. These slopes may be changed as required for handicap access points in accordance with the Americans with Disabilities Act.

3. Ponding of water should not be allowed in backfill material or in a zone within 10 feet of foundations or foundation walls, whichever is greater.

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

5. Irrigated turf and other excessive landscape irrigation should be avoided within 10 feet of foundations.

PAVEMENT DESIGN

Subgrade Materials: Based on the results of the field exploration and laboratory testing programs, the pavement subgrade materials encountered at the site classify as A-4 and A-6 with group a index ranging from 0 to 4 in accordance with the American Association of State Highway and Transportation Officials (AASHTO) soil classification system. An R-value of 10 and a resilient modulus value of 3,562 psi were assumed for design of flexible pavements, and a corrected subgrade modulus of 100 pci was assumed for design of rigid pavements.

Design Traffic: Based on the anticipated traffic loading information provided in the 7-Eleven pavement guidelines and standards, the 18-kip equivalent single axle loading (ESAL) values used for the asphalt paved parking surfaces and automobile drive lanes (Standard-Duty) and areas accessed by tanker trucks (Heavy-Duty) are presented in the table below:

Pavement Alternative	Location	7-Eleven Required ESAL
Asphalt	Standard Duty	39,000
	Heavy Duty	2,250,000
Rigid	Standard Duty	100,000
	Heavy Duty	1,000,000

The Heavy-Duty pavement section should be constructed in locations of concentrated vehicular traffic movements. If the above-presented 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.

<u>Pavement Thickness Requirements</u>: Recommendations for a composite section of hot mix asphalt (HMA) over the aggregate base course (ABC), and a rigid Portland cement concrete pavements (PCCP) section are presented in this subsection. The pavement sections were determined in accordance with the 1993 AASHTO pavement design procedures using DARWin 3.01 pavement thickness design software.

For design purposes, the 7-Eleven standard design values presented in the table below were used to estimate the minimum pavement thickness alternatives:

Design Parameter	Design Value
Reliability	85%
Initial Serviceability	4.2 for Asphalt and 4.5 for Concrete
Terminal Serviceability	2.5
Overall Deviation	0.45 for Asphalt and 0.35 for Concrete
Concrete Compressive Strength	4,000 psi
Modulus of Rupture	630 psi

The following table presents the minimum pavement thickness alternatives for the project:

LOCATION	Asphalt Over Aggregate Base Course (inches)	PCCP (inches)
Standard Duty	4.25 over 6.0	6.0
Heavy Duty	8.0 over 12.0	8.5

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 8.5 inches of PCCP. The PCCP thicknesses presented above are for unreinforced sections.

<u>Subgrade Preparation</u>: As discussed in the "Geotechnical Engineering Considerations" section, provided the owner understands and accepts the potential for differential subgrade settlement and the resulting increased potential for distress, partial overexcavation of the existing fill in pavement areas can be considered, as these items can typically tolerate movement and are more easily repaired. At a minimum we recommend overexcavation and replacement of the existing fill to a minimum depth of 2 feet in exterior flatwork areas. Increasing the depth overexcavation will reduce the risk and magnitude of potential movements.

Fill placed for support of pavements should meet the material and compaction requirements for structural fill presented in the "Site Grading and Earthwork" section of this report. Prior to placement of fill or pavement materials, the entire subgrade area should be scarified to a depth of 12 inches, moisture conditioned as necessary, and compacted to the criteria discussed above.

Proof Roll: Before paving, the subgrade should be proof rolled with a heavily loaded, pneumatic-tired vehicle. The vehicle should have a gross vehicle weight of at least 50,000 pounds with a loaded single axle weight of 18,000 pounds and a tire pressure of 100 psi. Areas which deform excessively under heavy wheel loads are not stable and should be removed and replaced with suitable material to achieve a stable subgrade prior to paving or placement of base course.

Drainage: The collection and diversion of surface drainage away from paved areas is extremely important to the satisfactory performance of pavement. Drainage design should provide for the removal of water from paved areas and prevent the wetting of the subgrade soils.

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 HMA pavements should meet the material specifications for Class 6 ABC stated in the current CDOT *"Standard Specifications for Road and Bridge Construction"*.

2. *Hot Mix Asphalt:* Hot mix asphalt (HMA) materials and mix designs should meet the applicable requirements indicated in the current "Pike Peak Region Asphalt Paving Specifications" document. We recommend that the HMA used for this project be designed in accordance with the SuperPave gyratory mix design method. The mix should meet Grading S specifications with a SuperPave gyratory design revolution (*NDESIGN*) of 75. A mix meeting Grading SX specification can be used for the top lift wearing course, however, this is optional. The mix design(s) for the HMA should use a performance grade (PG) asphalt binder of PG 58-28 or PG 64-22. However, we recommend the PG 58-28 binder which tends to perform better under relatively low traffic volumes such as those anticipated at this site. Placement and compaction of HMA should follow the specifications provided in the Pike Peak Region specifications document.

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 one-third 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.

SITE GRADING AND EARTHWORK

Cut and Fill Slopes: Permanent cut and fill slopes should not be steeper than 3:1 (horizontal to vertical). Slopes will generally be stable at 2:1; however, 2:1 slopes will be prone to increased surface erosion and it will be difficult to maintain vegetation on them. The risk of slope instability will be significantly increased if seepage is encountered in cuts. If seepage is encountered in permanent excavations, an investigation should be conducted to determine if the seepage will adversely affect the cut stability.

Good surface drainage should be provided for all permanent cuts and fills to direct the surface runoff away from the slope faces. Permanent cut and fill slopes and other stripped areas should be protected against erosion by revegetation or other means. Fills should be benched into hillsides exceeding 4 horizontal to 1 vertical. Site grading should be planned to provide positive surface drainage away from all building and pavement areas.

No formal stability analyses were performed to evaluate the slopes recommended above. Published literature and our experience with similar cuts and fills indicate the recommended slopes should have adequate factors of safety. If a detailed stability analysis is required, we should be notified.

Temporary Excavations: We recommend temporary excavation slopes be constructed in accordance with OSHA regulations. In accordance with OSHA criteria, the on-site fill and native soils should be considered a Type C soil due to the variability of material properties. Temporary unretained excavations should have slopes no steeper than 1.5:1 (H:V) in Type C soils. A properly braced excavation or the use of a trench box should be used where the indicated unretained slopes cannot be accommodated. Flatter slopes will be required where groundwater seepage is encountered. OSHA regulations require that excavations greater than 20 feet in depth be designed by a professional engineer. If soils different from those indicated in this report are encountered, the OSHA soil type may vary and the required cut slopes may need to be adjusted. A contractor's competent person should make decisions regarding cut slopes.

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

<u>Excavation Considerations</u>: In our opinion, the overburden soils encountered in the exploratory borings drilled for this study can be excavated with heavy-duty construction equipment. Groundwater was encountered at depths of about 23 feet or greater in our borings, and it is not anticipated to be a design or construction consideration provided excavations do not extend deeper than about 23 feet. If groundwater is encountered during construction, we believe excavations can be dewatered using perimeter and/or lateral trenches combined with sumps. The trenches should be sloped to sumps where water can be pumped from the excavation. The bottom and sides of the excavation may become unstable, especially in the granular soils if the groundwater level is not lowered in advance of excavation and maintained at a sufficient depth below the bottom of the excavation. Dewatering must be maintained through the time period the excavation is open. The dewatering system should be properly designed, installed, and maintained by an experienced dewatering contractor.

<u>Underground Storage Tank Considerations</u>: Boring 4 was drilled in the vicinity of the proposed underground storage tanks, and encountered existing fill soils to a depth of approximately 17 feet, followed by native clay soils which extended to a depth of approximately 36 feet, and then claystone bedrock which extended to the 40-foot depth explored. Groundwater was encountered at a depth of 33 feet during drilling, and 27.9 feet when the boring was checked 7 days later.

We recommend that the tanks be supported on properly compacted structural fill. Any areas of existing fill or loose or soft material encountered at the bearing elevation should be removed and replaced with suitable nonexpansive fill. New structural fill should extend down from the edges of the foundations at a minimum 1 horizontal to 1 vertical projection. As an alternative to overexcavation, the tanks may be supported with drilled or helical piers, following the criteria presented in the "Foundation Recommendations" section of the report.

Our experience is that underground storage tanks are typically backfilled with granular bedding material to ease and speed up installation. We recommend that granular bedding material be compacted in accordance with the tank manufacturer's requirements or to at least 75% relative density (ASTM D 4253 and ASTM D 4254). Special care should be taken to provide adequate compaction below the haunches of the tank(s) using a concrete vibrator, vibratory plates, or other light compaction equipment as needed. In confined areas where compaction is difficult, placement of a cementitious flow fill around the tank(s) should be considered.

Subgrade Stabilization: Depending on the time of season that construction occurs, unstable subgrade may be encountered during subgrade preparations. Unstable soils may be stabilized by scarifying/ripping the subgrade and allowing it to dry, or by overexcavation and replacement of the subgrade with suitable, imported, angular, well-graded materials. Other alternatives include the use of Type 2 biaxial geogrid reinforcement in combination with a layer of Class 6 aggregate base course. It has been our experience that the use of a crushed concrete product meeting a Class 6 gradation can perform well when trying to achieve stabilization. Specific stabilization requirements should be evaluated at the time of construction. We are unable to accurately predict or quantify areas where unstable subgrade conditions may occur, however, we recommend this work activity, if required, be included as a line item in the bid schedule to avoid cost overruns.

Fill Material: The following recommended material and compaction requirements are presented for structural fills on the project site. A geotechnical engineer should evaluate the suitability of all proposed fill materials for the project prior to placement.

1. *Structural Fill*: With proper moisture conditioning, the on-site existing fill and native soils will be suitable for reuse as structural fill, including structural fill beneath foundations, floor slabs, exterior flatwork, and pavements. New fill should extend down from the edge of footings at a minimum 1:1 horizontal to vertical projection. Imported structural fill, if required, should consist of nonexpansive soil material having a maximum particle size of 2 inches, a maximum of 35% passing the No. 200 sieve, and a maximum plasticity index of 10.

2. *Pavement Areas*: Fill should consist of the onsite soils or similar imported soil which meets the minimum R-value used for the pavement design calculation (minimum R value of 10).

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*: All fill material should be 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.

5. *Overexcavation:* As discussed previously, we recommend overexcavation, moisture conditioning and recompaction of the existing fill where encountered below shallow foundations, slab-on-grade floors and underground storage tanks.

   Provided the owner understands and accepts the potential for differential subgrade settlement and the resulting increased potential for distress, partial overexcavation of the existing fill in pavement and exterior flatwork areas can be considered, as these items can typically tolerate movement and are more easily repaired. At a minimum we recommend overexcavation and replacement of the existing fill to a minimum depth of 2 feet in pavement and exterior flatwork areas. Increasing the depth overexcavation will reduce the risk and magnitude of potential movements.

6. *General Subgrade Preparation*: The ground surface shall be stripped of vegetation/organics prior to overexcavation and fill placement. Loose or unstable soils shall be removed, where present, in order to provide a stable platform prior to placement of fill.

7. *Lift Thickness*: The maximum allowable loose lift thickness should be limited to 8 inches.

<u>Compaction Requirements</u>: 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.

Area		Percentage of Standard Proctor Maximum Dry Density (ASTM D 698)	Percentage of Modified Proctor Maximum Dry Density (ASTM D 1557)
Building Pad/Foundation Subgrade	>10’	100%	-
	<10’	98%	
Floor Slab Subgrade	>10’	100%	-
	<10’	95%	
General Overlot Grading	>10’	100%	
	<10’	95%	
Foundation Backfill		95%	-
Beneath Pavement Areas/Exterior Flatwork/Utility Trenches		95%	-
Landscape and Other Misc. Overlot Fill Areas		95%	-
Aggregate Base Course		-	95%
Compaction should be achieved at a moisture content within +/- 2% of the optimum for granular soils, and between -1% and +3% for clay soils.			

Grading During Inclement Weather: If grading occurs during the time of year with freezing temperatures, fill should not contain frost or be placed on frozen ground. A loose lift or blanket of soil should be placed on prepared subgrade at the end of day to protect against frost, and the presence of frost should be checked at the beginning of each day. Any frost should be removed prior to placement of new fill. During periods of precipitation, excessive wetting of building pad and pavement subgrades should be avoided. Measures should be implemented to keep surface runoff from ponding in subgrade areas, to include usage of diversion berms, creation of temporary low areas where water can be directed and removed by pumping, or other methods as appropriate.

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 that Kumar & Associates, Inc. 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 exclusive use by the client for geotechnical related design and construction criteria 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, 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 that 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.

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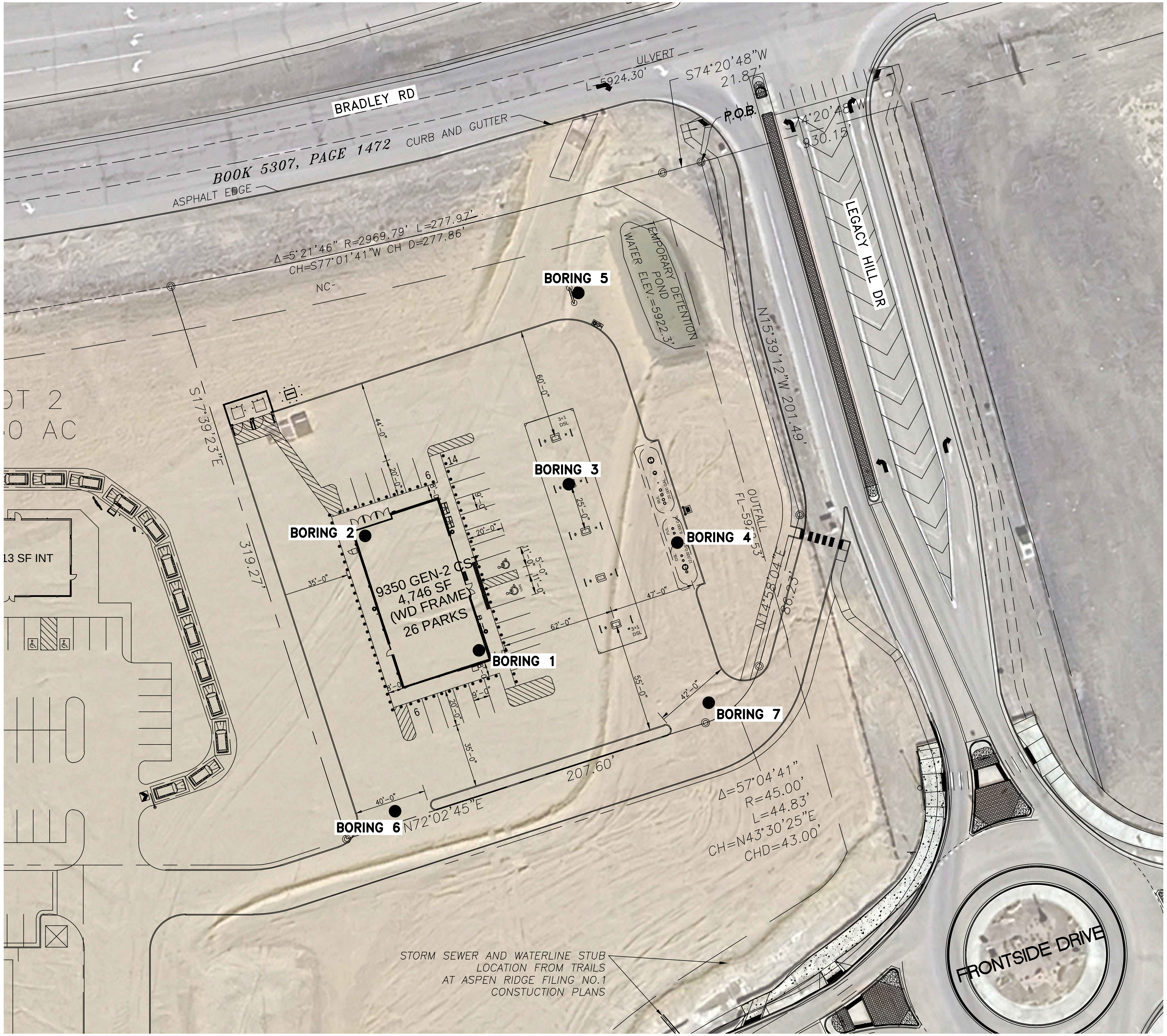

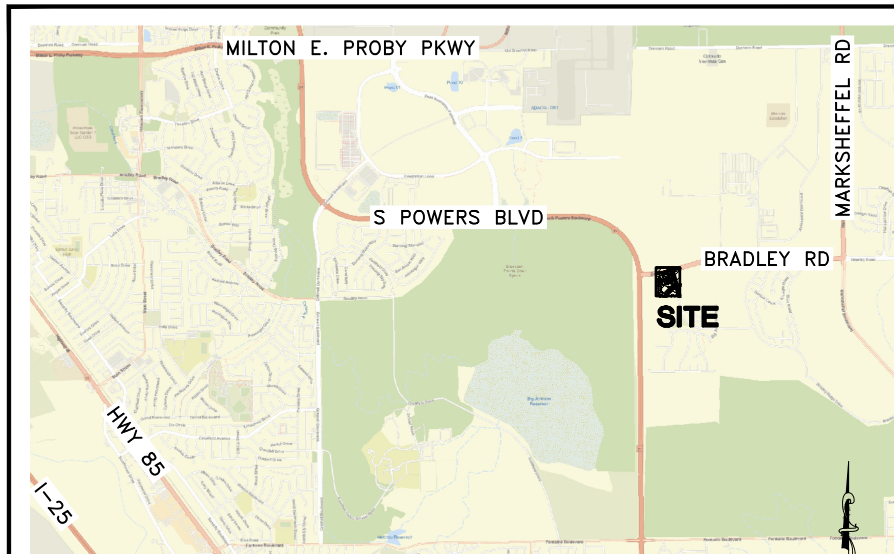

VICINITY MAP
NOT TO SCALE

25–2–231	Kumar & Associates		LOCATION OF EXPLORATORY BORINGS	Fig. 1

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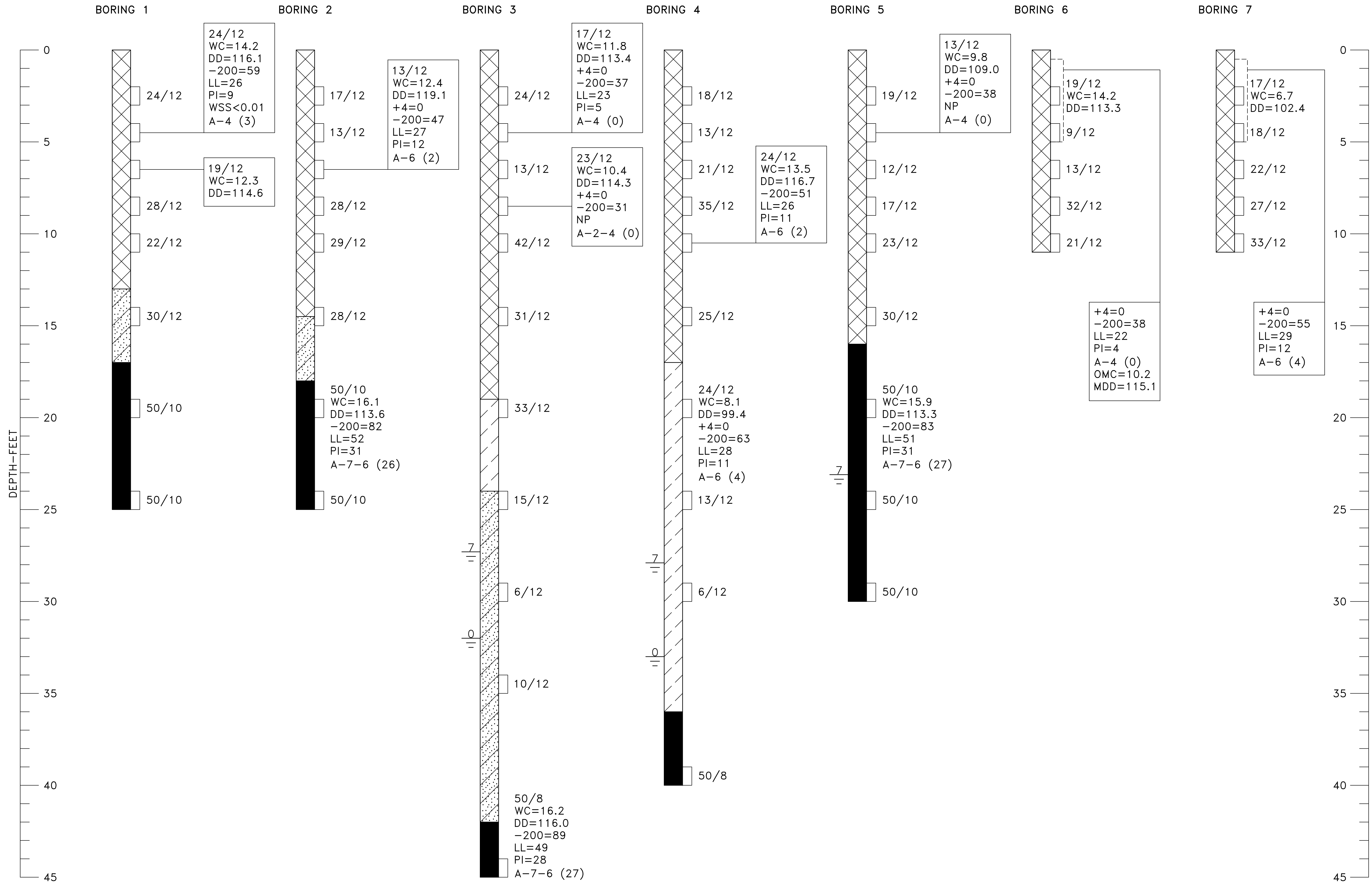

25–2–231	Kumar & Associates		LOGS OF EXPLORATORY BORINGS	Fig. 2

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LEGEND

FILL: MIXTURE OF SANDY LEAN CLAY (CL), CLAYEY SAND (SC), SILTY-CLAYEY SAND (SC-SM), AND SILTY SAND (SM), SLIGHTLY MOIST TO MOIST, MOTTLED SHADES OF TAN TO BROWN AND OCCASIONAL GRAY.

SILTY SAND (SM), WITH OCCASIONAL LAYERS OF CLAYEY SAND (SC), LOOSE TO MEDIUM DENSE, SLIGHTLY MOIST TO WET (BELOW GROUNDWATER), TAN, BROWN AND GRAY.

SANDY LEAN CLAY (CL), WITH OCCASIONAL LAYERS OF CLAYEY SAND (SC), MEDIUM STIFF TO HARD, SLIGHTLY MOIST TO WET (BELOW GROUNDWATER), TAN, BROWN AND GRAY.

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

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

DISTURBED BULK SAMPLE.

24/12 DRIVE SAMPLE BLOW COUNT. INDICATES THAT 24 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE THE SAMPLER 12 INCHES.

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

NOTES

1. THE EXPLORATORY BORINGS WERE DRILLED ON OCTOBER 10, 2025 WITH A 4-INCH-DIAMETER CONTINUOUS-FLIGHT POWER AUGER.

2. THE LOCATIONS OF THE EXPLORATORY BORINGS WERE MEASURED APPROXIMATELY BY HANDHELD GPS DEVICE AND SHOULD BE CONSIDERED ACCURATE ONLY TO THE DEGREE IMPLIED BY THE METHOD USED.

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

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

5. GROUNDWATER LEVELS SHOWN ON THE LOGS WERE MEASURED AT THE TIME AND UNDER CONDITIONS INDICATED. FLUCTUATIONS IN THE WATER LEVEL MAY OCCUR WITH TIME.

6. 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);
   NP = NON-PLASTIC (ASTM D4318);
   WSS = WATER SOLUBLE SULFATES (%) (CP-L 2103);
   A-4 (3) = AASHTO CLASSIFICATION (GROUP INDEX) (AASHTO M 145);
   OMC = OPTIMUM MOISTURE CONTENT (%) (ASTM D698);
   MDD = MAXIMUM DRY DENSITY (pcf) (ASTM D698).

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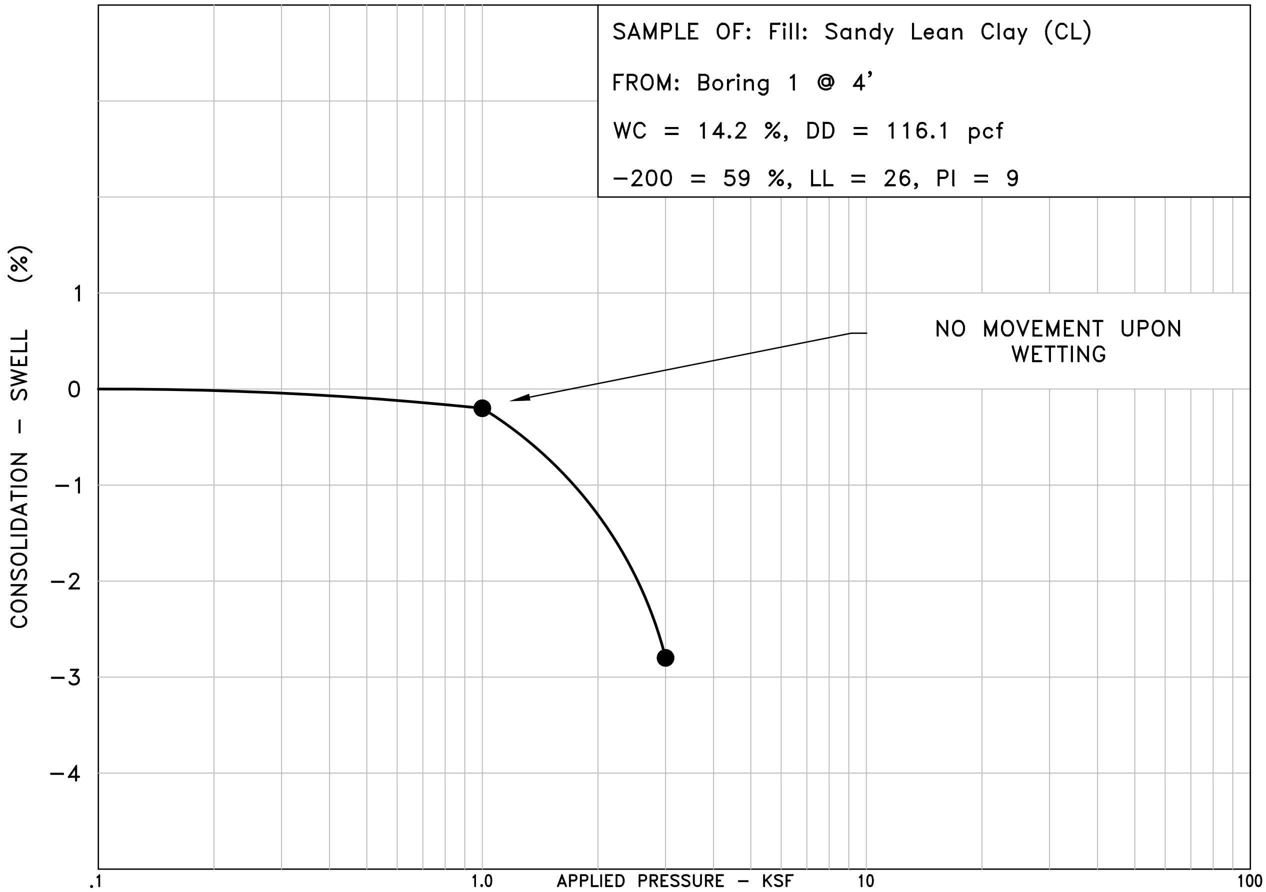

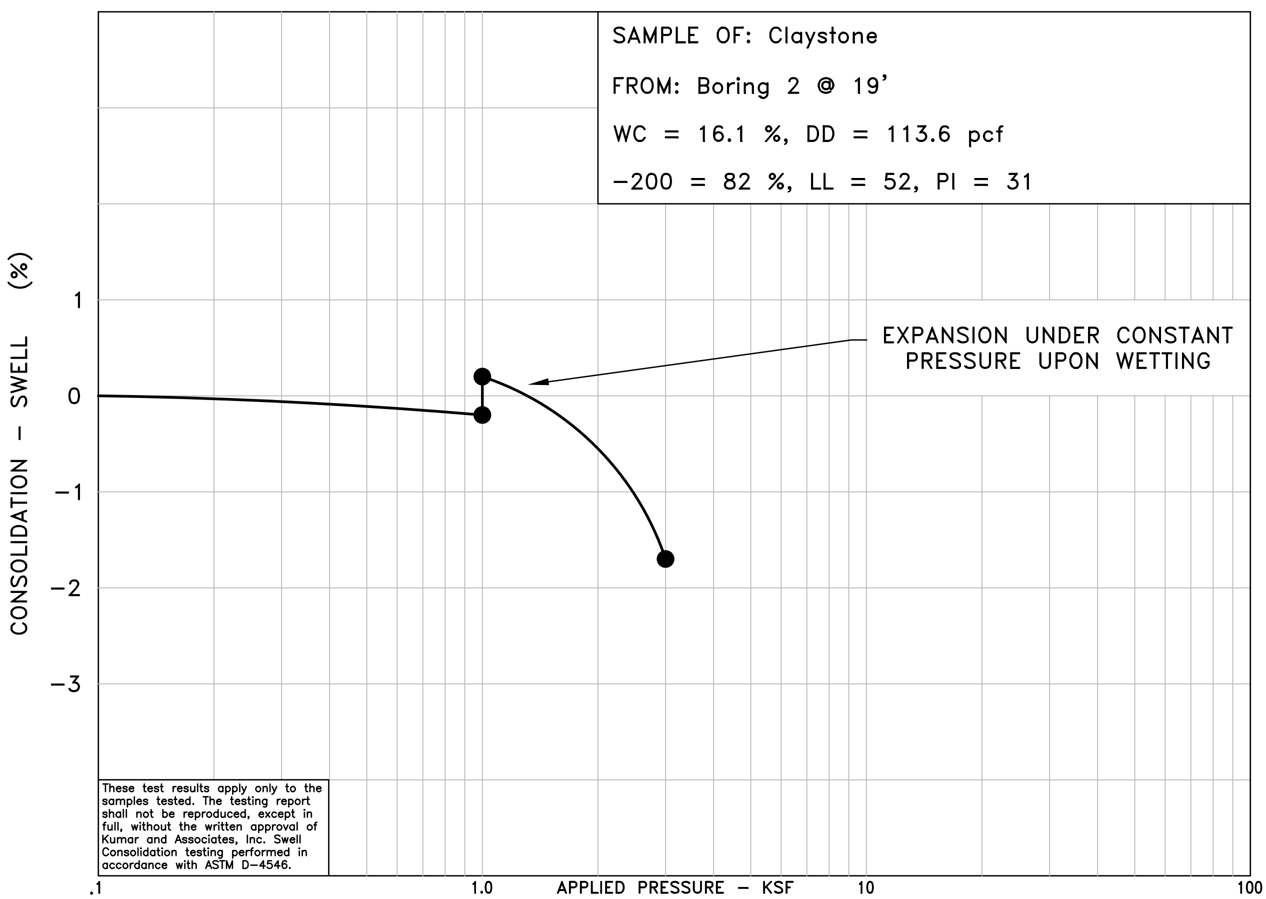

25-2-231	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULTS	Fig. 4

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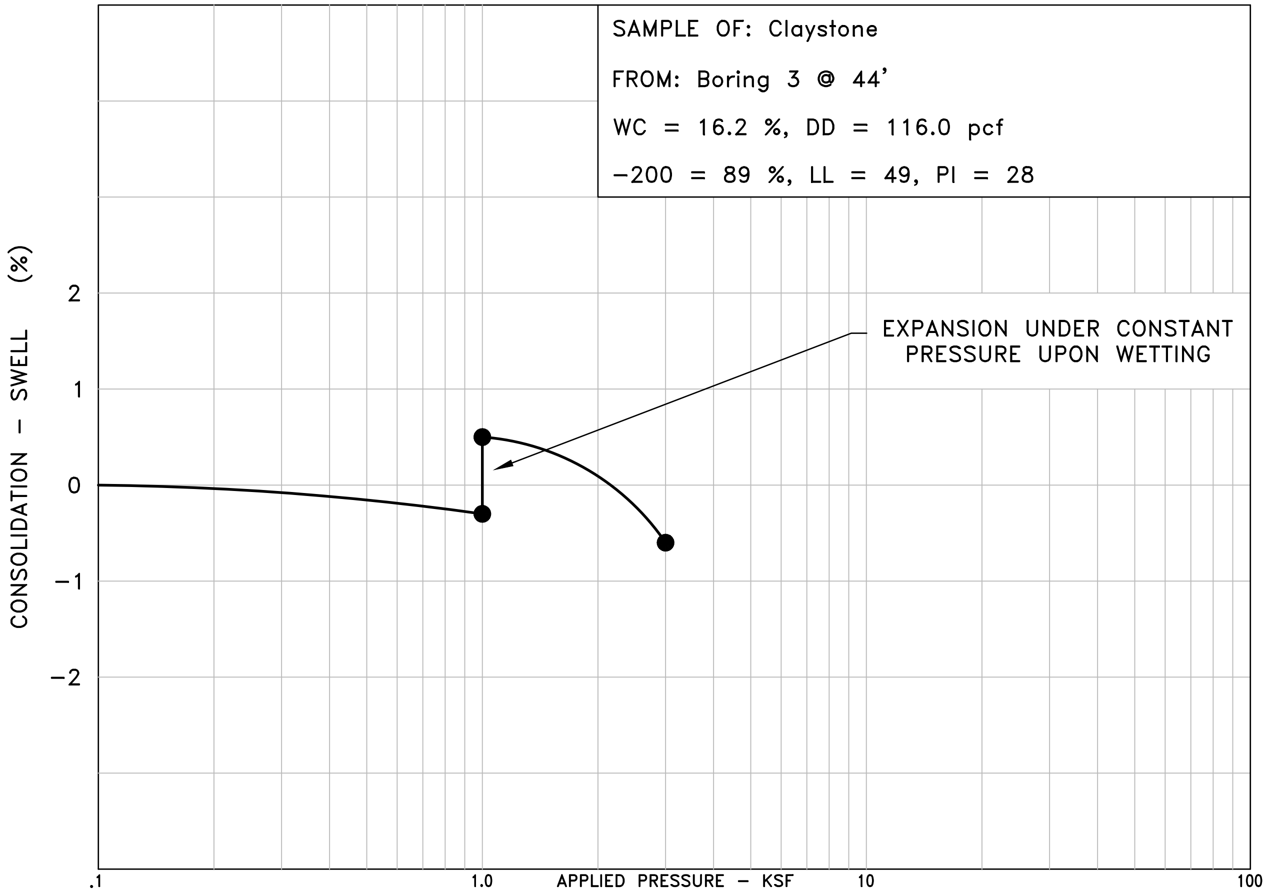

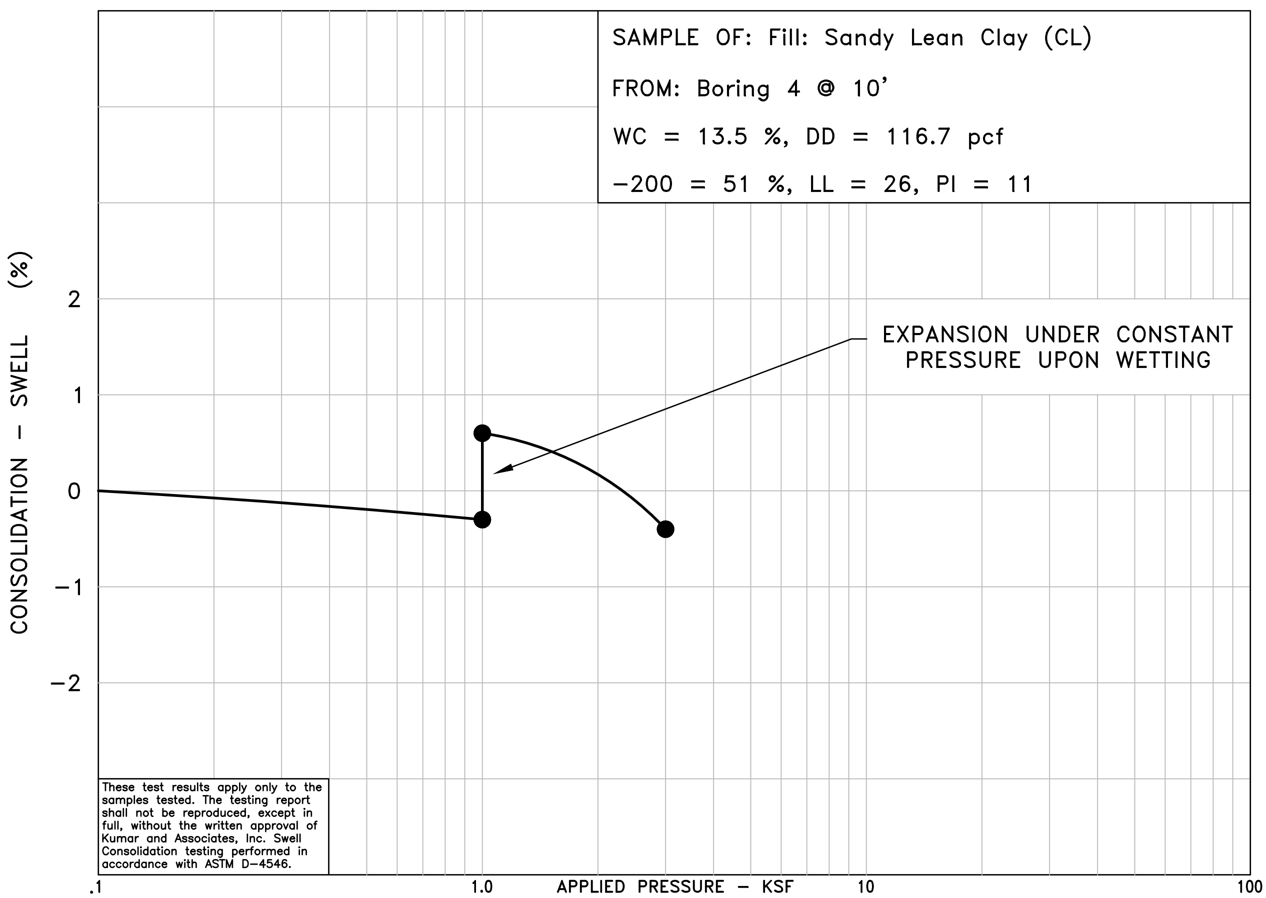

25−2−231	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULTS	Fig. 5

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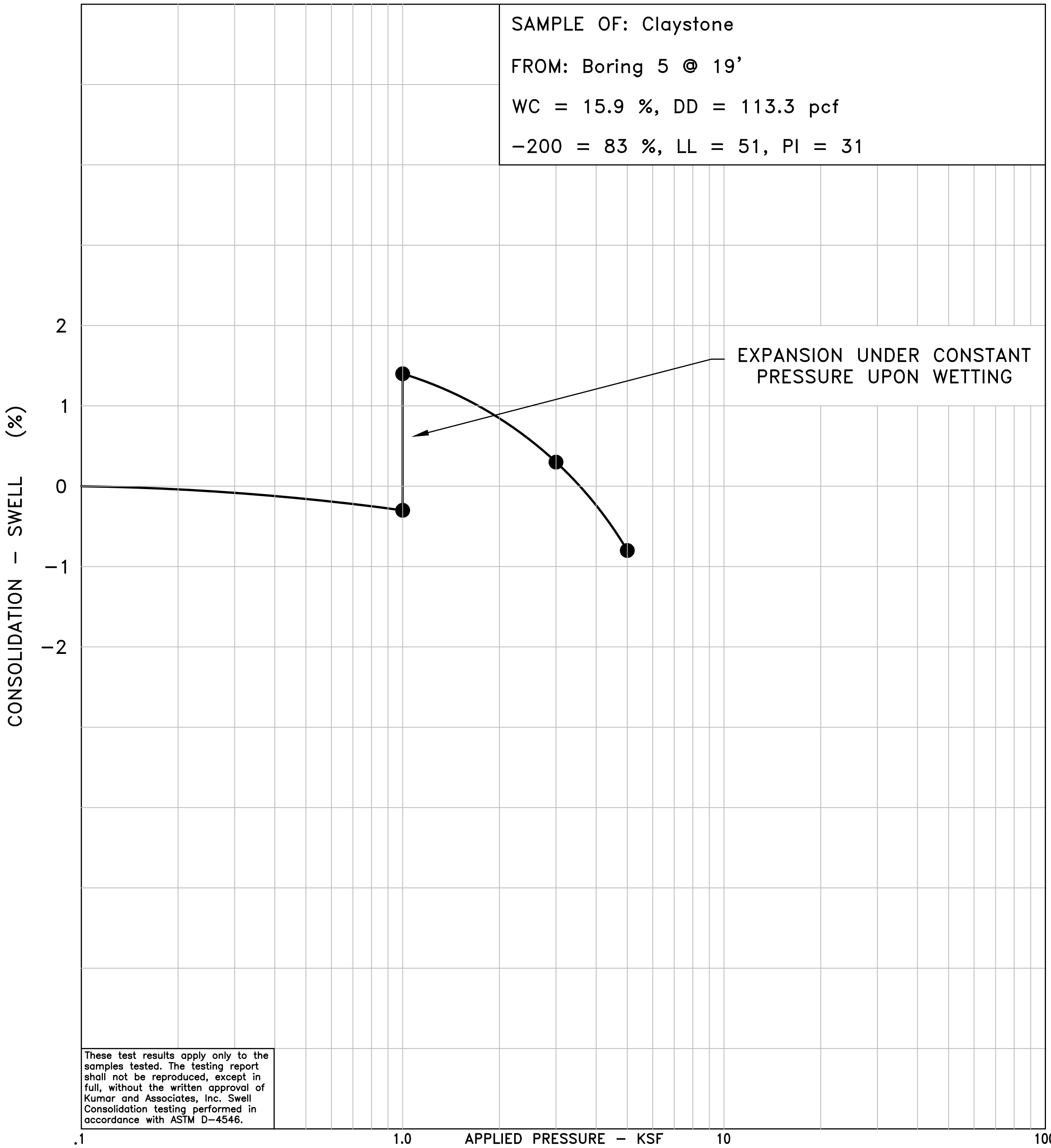

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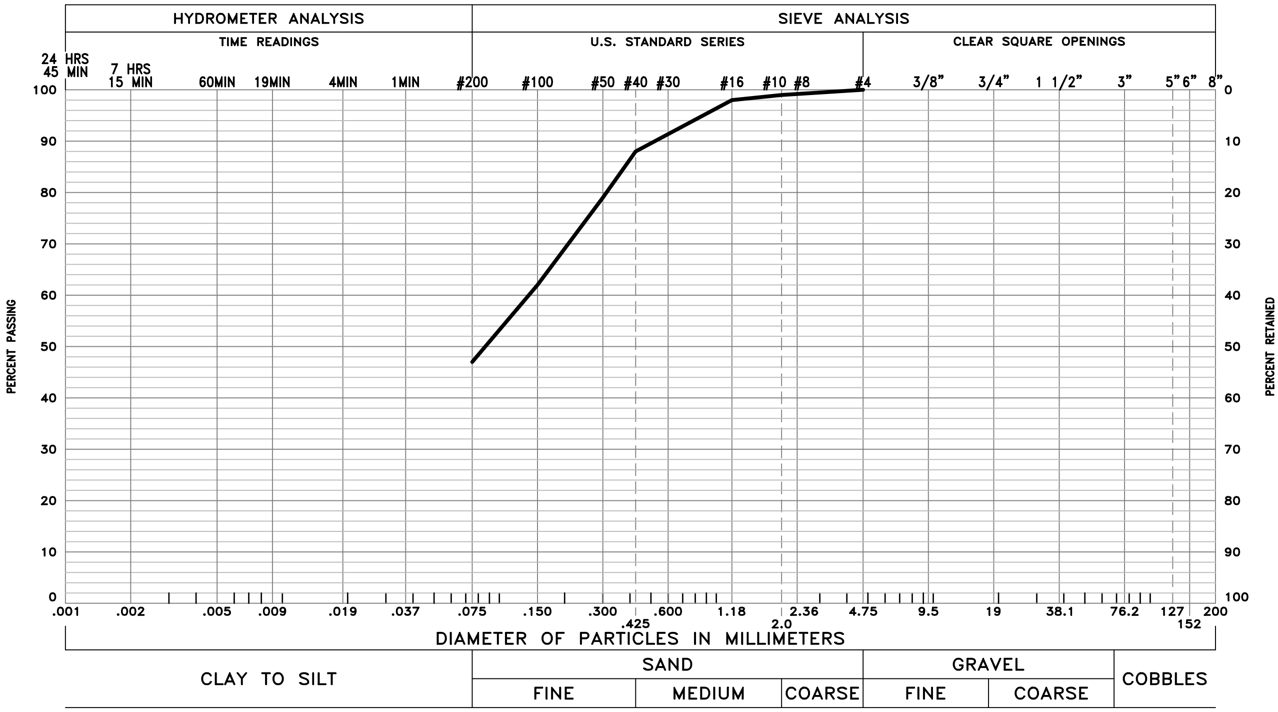
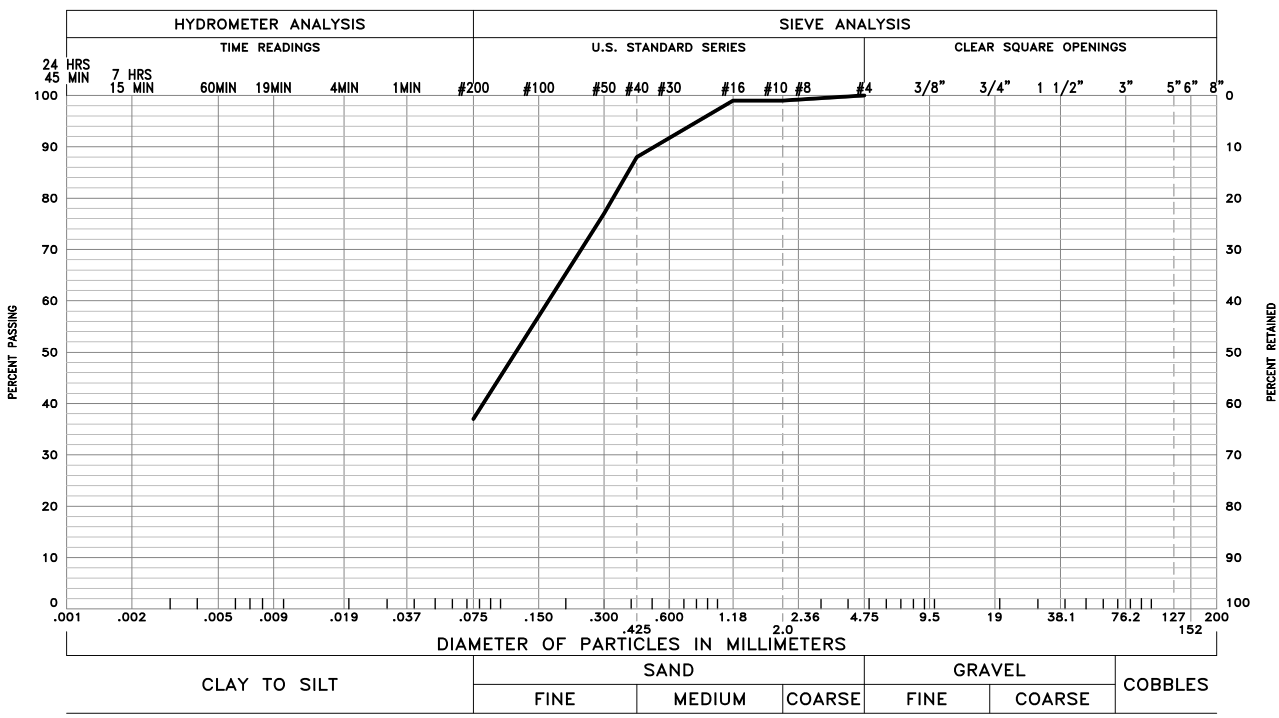
25-2-231	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULTS	Fig. 6



GRAVEL 0 % SAND 53 % SILT AND CLAY 47 %

LIQUID LIMIT 27 PLASTICITY INDEX 12

SAMPLE OF: Fill: Clayey Sand (SC) FROM: Boring 2 @ 6'



GRAVEL 0 % SAND 63 % SILT AND CLAY 37 %

LIQUID LIMIT 23 PLASTICITY INDEX 5

SAMPLE OF: Fill: Silty Clayey Sand (SC–SM) FROM: Boring 3 @ 4'

These test results apply only to the samples which were tested. The testing report shall not be reproduced, except in full, without the written approval of Kumar & Associates, Inc. Sieve analysis testing is performed in accordance with ASTM D6913, ASTM D7928, ASTM C136 and/or ASTM D1140.

25–2–231	Kumar & Associates	GRADATION TEST RESULTS	Fig. 7

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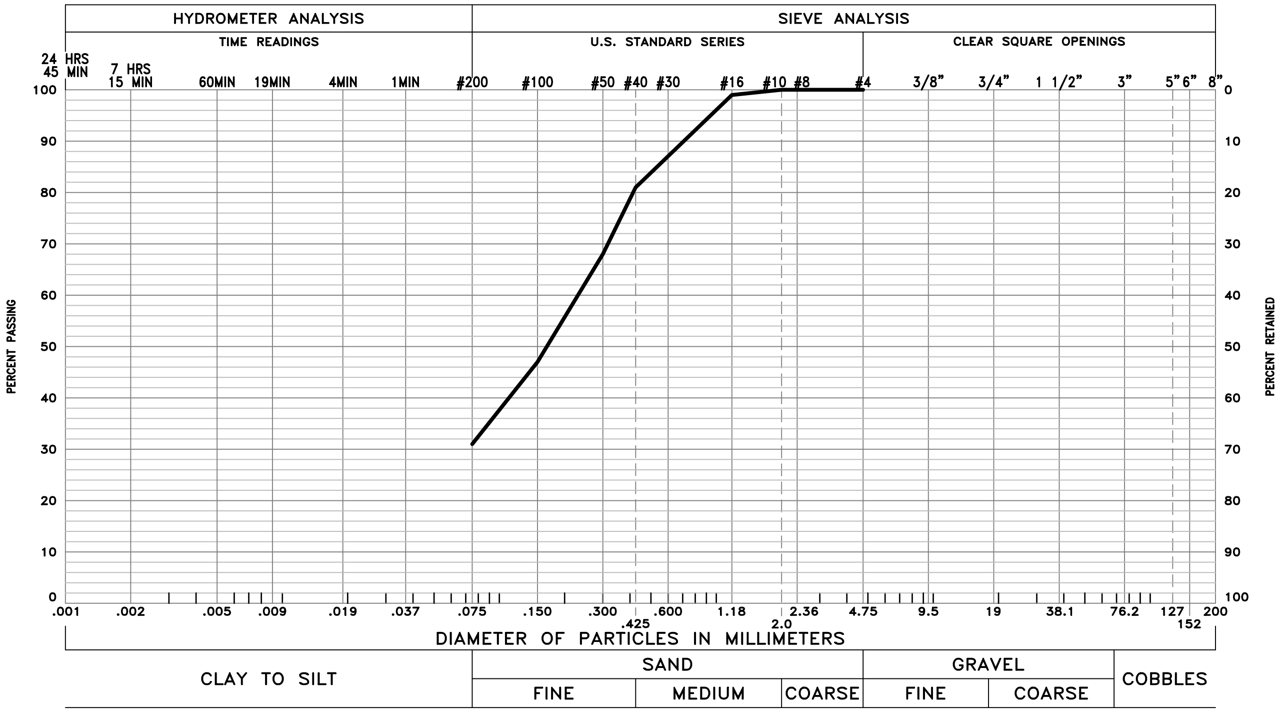
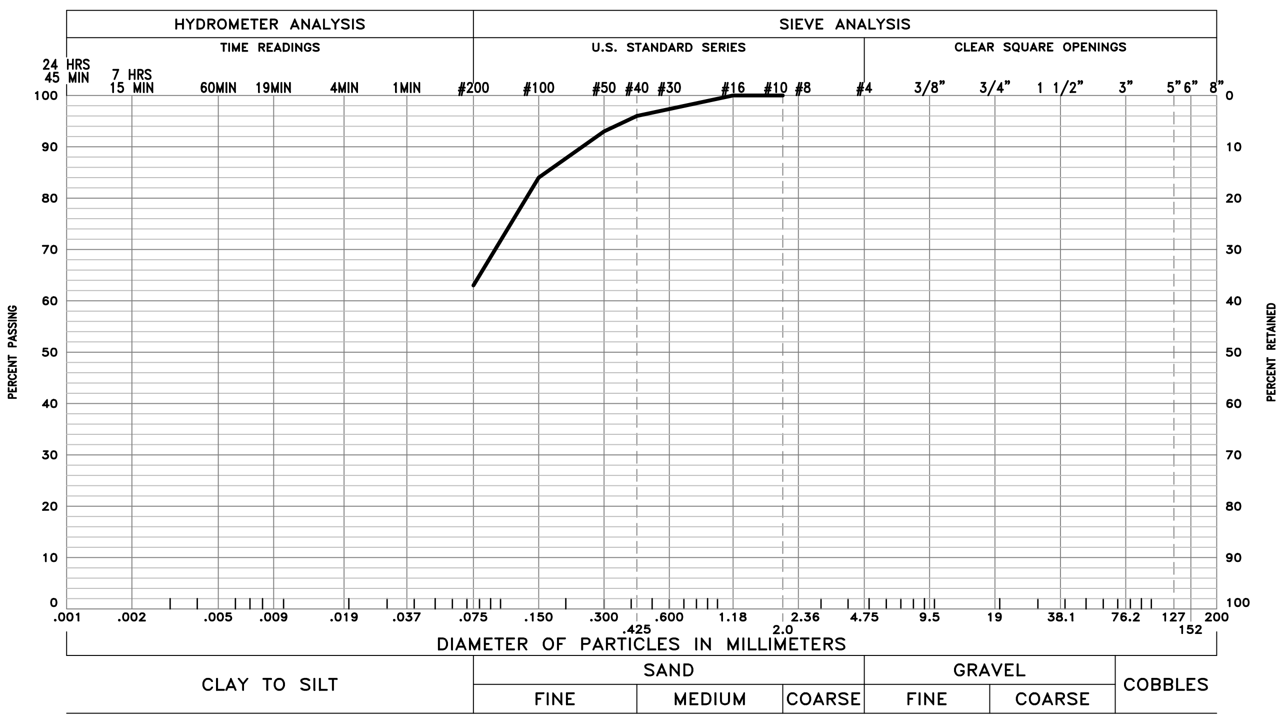


GRAVEL 0 % SAND 69 % SILT AND CLAY 31 %

LIQUID LIMIT – PLASTICITY INDEX NP

SAMPLE OF: Fill: Silty Sand (SM) FROM: Boring 3 @ 8'



GRAVEL 0 % SAND 37 % SILT AND CLAY 63 %

LIQUID LIMIT 28 PLASTICITY INDEX 11

SAMPLE OF: Sandy Lean Clay (CL) FROM: Boring 4 @ 19'

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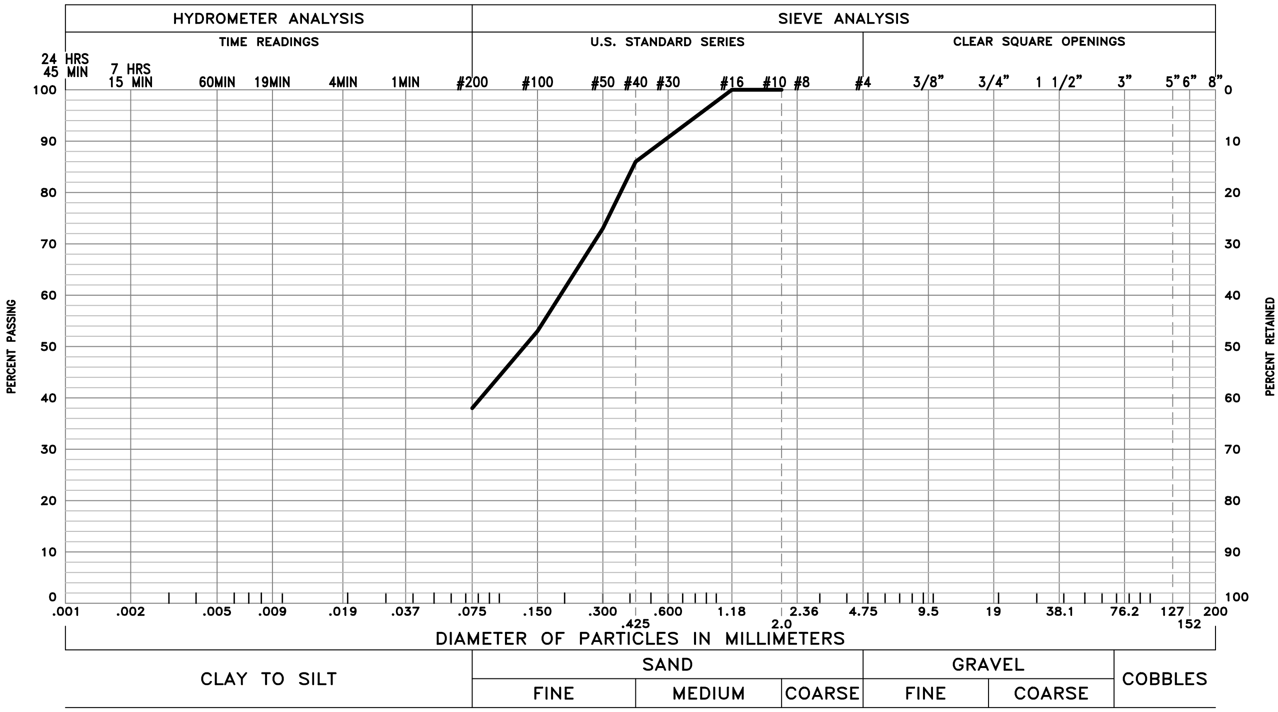
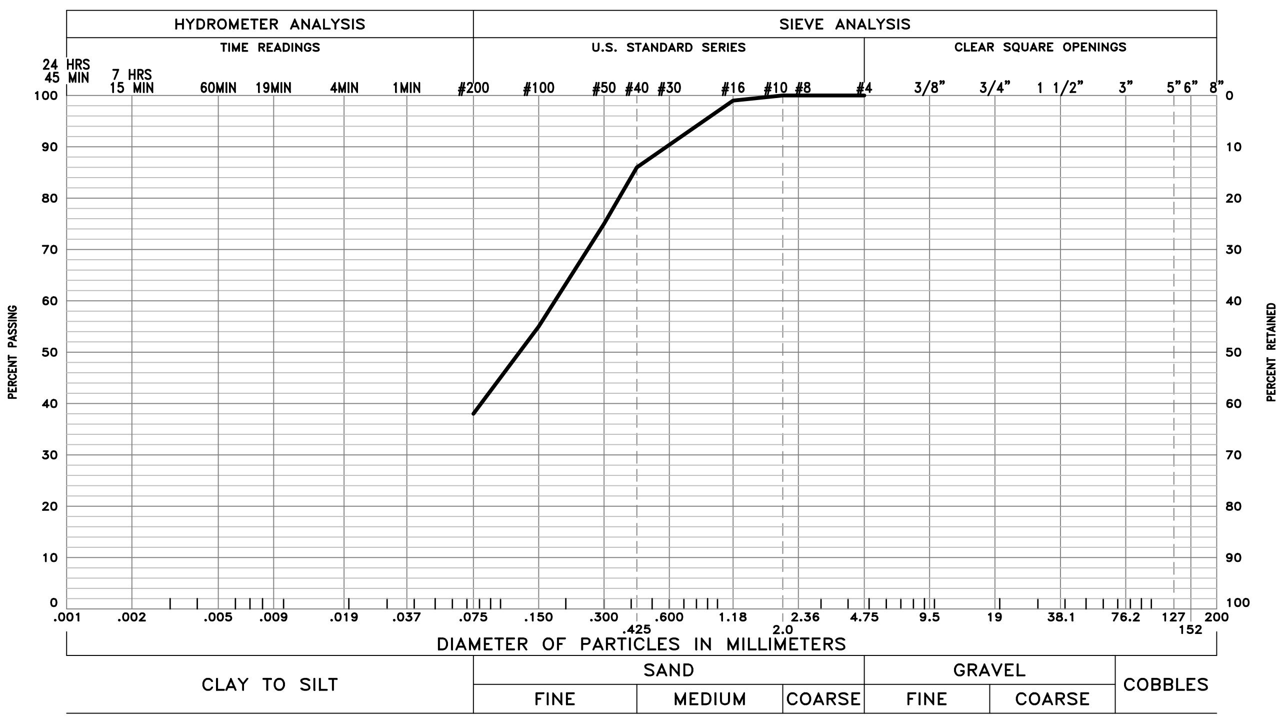


GRAVEL 0 % SAND 62 % SILT AND CLAY 38 %

LIQUID LIMIT – PLASTICITY INDEX NP

SAMPLE OF: Fill: Silty Sand (SM) FROM: Boring 5 @ 4'



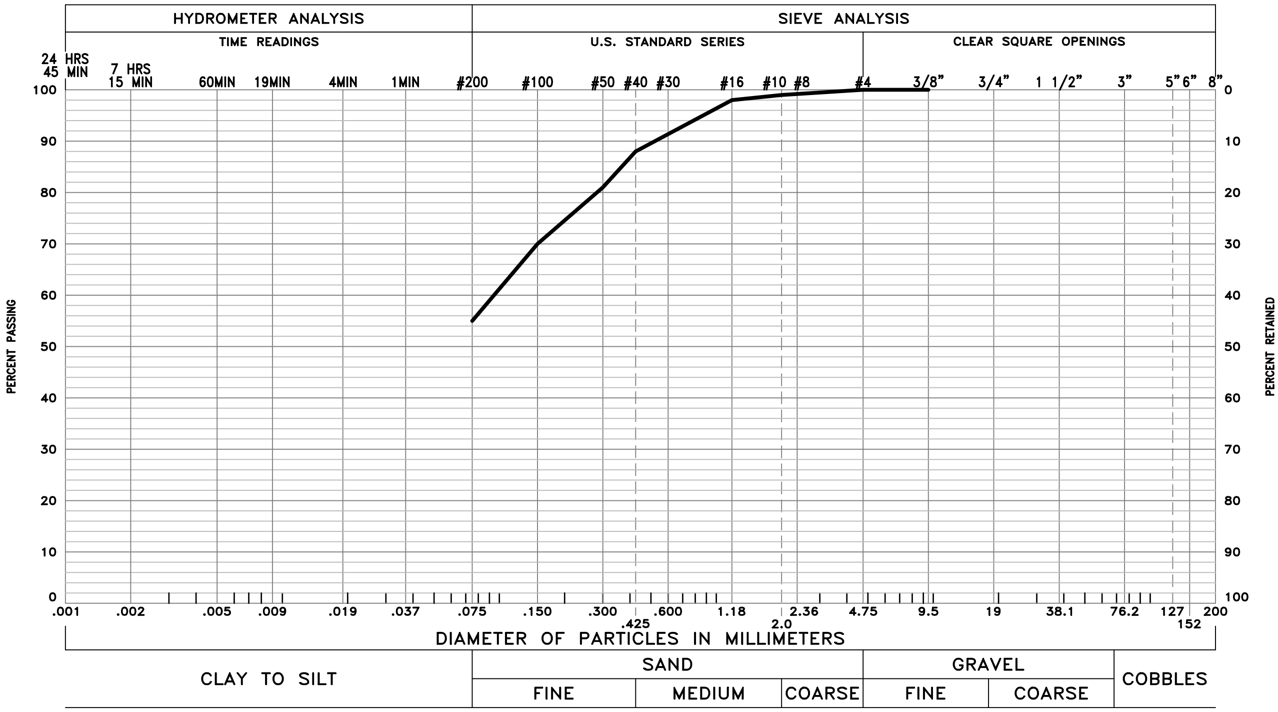
GRAVEL 0 % SAND 62 % SILT AND CLAY 38 %

LIQUID LIMIT 22 PLASTICITY INDEX 4

SAMPLE OF: Fill: Silty Clayey Sand (SC–SM) FROM: Boring 6 @ 6"–5'

These test results apply only to the samples which were tested. The testing report shall not be reproduced, except in full, without the written approval of Kumar & Associates, Inc. Sieve analysis testing is performed in accordance with ASTM D6913, ASTM D7928, ASTM C136 and/or ASTM D1140.

November 13, 2025 – 09:59am
V:\Projects\2025\25-2-231 7-11 Store\Drafting\252231-07 to 10.dwg



GRAVEL 0 % SAND 45 % SILT AND CLAY 55 %

LIQUID LIMIT 29 PLASTICITY INDEX 12

SAMPLE OF: Fill: Sandy Lean Clay (CL) FROM: Boring 7 @ 6"–5'

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November 13, 2025 – 09:59am
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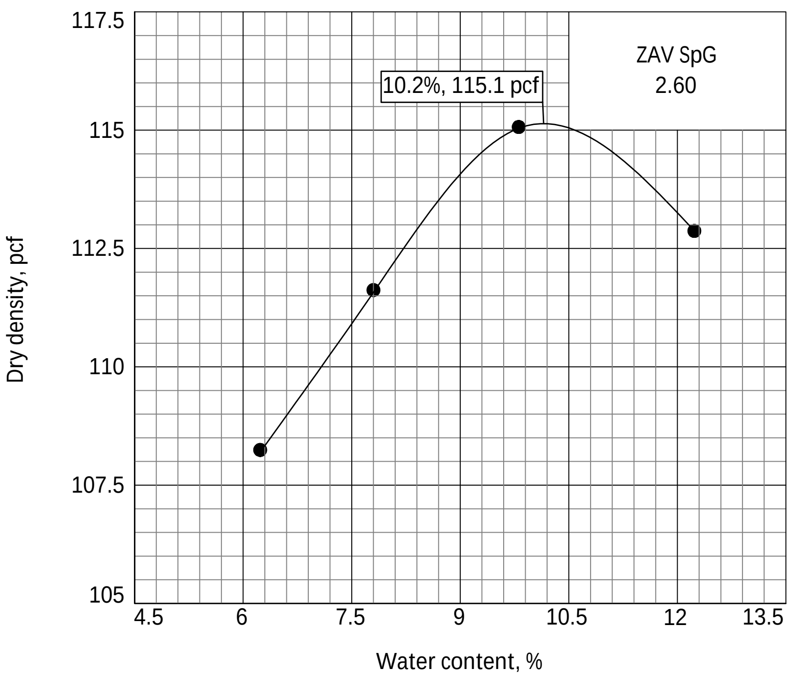
25–2–231	Kumar & Associates	GRADATION TEST RESULTS	Fig. 10

COMPACTION TEST REPORT



Curve No. 1

Preparation Method
Rammer: Wt. 5.5 lb. Drop 12 in.
Type
Layers: No. 3 Blows per 25
Mold Size 0.03333 cu. ft.
Test Performed on Material Passing #4 Sieve

%>#4 0 %<No.200 38
Atterberg (D 4318): LL 22 PI 4
NM (D 2216) Sp.G. (D 854) 2.60
USCS (D 2487) SC-SM
AASHTO (M 145) A-4(0)
Date: Sampled 10/14/25
Received 10/14/25
Tested 10/29/25
Tested By JEE

COMPACTION TESTING DATA
ASTM D 698-12 Method A Standard

	1	2	3	4	5	6
WM + WS	6062.1	6143.0	6234.3	6239.2		
WM	4320.1	4320.1	4320.1	4320.1		
WW + T #1	698.9	480.7	653.2	559.4		
WD + T #1	668.9	459.9	611.8	522.2		
TARE #1	188.7	193.7	190.3	218.5		
WW + T #2						
WD + T #2						
TARE #2						
MOIST.	6.2	7.8	9.8	12.2		
DRY DENS.	108.2	111.6	115.0	112.9		

SIEVE TEST RESULTS

Opening Size	% Passing	Specs.
#4	100	
#10	100	
#16	99	
#40	86	
#50	75	
#100	55	
#200	38	

TEST RESULTS

Maximum dry density = 115.1 pcf

Optimum moisture = 10.2 %

Material Description

Fill: Silty Clayey Sand (SC-SM)

Remarks:

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

Project No. 252-231 **Client:** Lasco Dev.

Project: 7-11 Bradley Rd.

○**Location: Boring 6 @ 6"-5'** **Sample Number: 1**

Checked by: DPC

Title: Senior Project Engineer

25-2-231	Kumar & Associates	MOISTURE-DENSITY RELATIONSHIPS	Fig. 11

November 13, 2025 - 09:59am
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Kumar and Associates, Inc.

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

Project No.: 25-2-231
Project Name: 7-11 Bradley Road
Date Sampled: 10/10/2025
Date Received: 10/14/2025

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 (Unified Soil Classification)
BORING	DEPTH (ft)				GRAVEL (%)	SAND (%)		LIQUID LIMIT	PLASTICITY INDEX			
1	4'	10/29/25	14.2	116.1			59	26	9	< 0.01	A-4 (3)	Fill: Sandy Lean Clay (CL)
	6'	10/29/25	12.3	114.6								Fill: Sandy Lean Clay (CL)
2	6'	10/29/25	12.4	119.1	0	53	47	27	12		A-6 (2)	Fill: Clayey Sand (SC)
	19'	10/29/25	16.1	113.6			82	52	31		A-7-6 (26)	Claystone
3	4'	10/29/25	11.8	113.4	0	63	37	23	5		A-4 (0)	Fill: Silty Clayey Sand (SC-SM)
	8'	10/29/25	10.4	114.3	0	69	31		NP		A-2-4 (0)	Fill: Silty Sand (SM)
	44'	10/29/25	16.2	116.0			89	49	28		A-7-6 (27)	Claystone
4	10'	10/29/25	13.5	116.7			51	26	11		A-6 (2)	Fill: Sandy Lean Clay (CL)
	19'	10/29/25	8.1	99.4	0	37	63	28	11		A-6 (4)	Sandy Lean Clay (CL)
5	4'	10/29/25	9.8	109.0	0	62	38		NP		A-4 (0)	Fill: Silty Sand (SM)
	19'	10/29/25	15.9	113.3			83	51	31		A-7-6 (27)	Claystone

Kumar and Associates, Inc.

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

Project No.: 25-2-231
Project Name: 7-11 Bradley Road
Date Sampled: 10/10/2025
Date Received: 10/14/2025

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 (Unified Soil Classification)
BORING	DEPTH (ft)				GRAVEL (%)	SAND (%)		LIQUID LIMIT	PLASTICITY INDEX			
6	6" - 5'	10/29/25	10.2*	115.1*	0	62	38	22	4		A-4 (0)	Fill: Silty Clayey Sand (SC-SM)
	2'	10/29/25	14.2	113.3								Fill: Silty Clayey Sand (SC-SM)
7	6" - 5'	10/29/25			0	45	55	29	12		A-6 (4)	Fill: Sandy Lean Clay (CL)
	2'	10/29/25	6.7	102.4								Fill: Sandy Lean Clay (CL)

* Optimum moisture content and maximum dry density, as determined by Standard Proctor (ASTM D698).